PGE Portland General Electric Company

Donald J. Broehl Assistant Vice President

November 21, 1979

Trojan Nuclear Plant
Docket 50-344
License NPF-1

Mr. R. H. Engelken, Director
U.S. Nuclear Regulatory Commission
Region V
Suite 202, Walnut Creek Plaza
1990 N. California Blvd.
Walnut Creek, CA 94596

Dear Sir:

As indicated in my letter of November 19, 1979 transmitting Supplement 1 to Licensee Event Report 79-15, enclosed are the following attachments for that Supplement:

Attachment 1: "Sample Calculations"

Attachment 5: "Tension and Shear Transfer"

Sincerely,

Donald J. Broehl

DJB/LWE/4sa6A19
Enclosures

c: Mr. A. Schwencer, Chief
Operating Reactors Branch #1
Division of Operating Reactors
U. S. Nuclear Regulatory Commission

Mr. Lynn Frank, Director
State of Oregon
Department of Energy

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ATTACHMENT 5

TENSION AND SHEAR TRANSFER

1. Introduction

The significant loads normal to the Trojan walls are due to piping reactions resulting from thermal and out-of-plane seismic inertia loads. The piping loads cause local tensile bond stresses at the concrete block - concrete core or concrete block - mortar interface. The out-of-plane seismic inertia loads cause shear stresses along these interfaces. The important parameters in considering monolithic action of the wall are the tensile bond and vertical shear transfer between the block and concrete or between the block and mortar in double wythe construction. Bond strength is the strength of the system joining the block to concrete or another block, and may be controlled by the strength of the block, the strength of the concrete or mortar, or the adhesive quality of the concrete or mortar to the block. The quality of construction used on Trojan contributes to the monolithic action.

2. Quality of Construction

Masonry blocks for Trojan were designed with one of the normal two interior cells open through its shell end or head face, thereby limiting the potential for a void or plane of weakness. The blocks were manufactured under quality controls which limited linear shrinkage to 0.05 percent and controlled the density of the units to between 130 and 135 pounds per cubic foot. After shipment to the Plant site, masonry block units were stacked under cover off the ground and protected from damage by construction activities. The contract specification which

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was followed specified that mortar overhangs and droppings be removed from cells, interior faces and foundations where fill was to be placed. Cleanout openings of sufficient size to allow flushing away of such materials were provided. Where accessible, the space between wythes of masonry was cleaned with the use of water spray. Unfinished work exposed to the weather was protected by covering the top of the wall.

Concrete core placement was accomplished from above by gravity flow from the discharge of concrete pump hoses. The placement procedure which was followed provided that individual lifts not exceed 8 ft. Standard 2-in. vibrators were used to limit voids in the core concrete ("Pencil" electric vibrators were used for block cell fill consolidation). Additionally, wall core volume was computed and checked against batched-pumped quantities. Materials and installation were covered by the Trojan Quality Assurance Program.

3. Tension and Shear Transfer Between Masonry and Concrete

The UBC does not address explicitly the tensile bond due to a direct tension load for a wall constructed using grouted unit masonry as a permanent form for concrete placed between the wythes. Based on our evaluation and considering the quality of the construction of these walls, we believe that the tensile bond and shear transfer between concrete and masonry block thus developed will each average on the order of 200 psi. To confirm the validity of the tensile bond value we judged to exist, Bechtel undertook an extensive literature search and also interviewed a number of experts in the masonry field.

The literature search included the "Proceedings of the North American Masonry Conference", August 1978 at the University of Colorado, a compilation of recent papers concerning masonry. Two of the papers contained in that reference are considered to be relevant in the evaluation of this type of interface.

The paper, "Behavior of Concrete Masonry Under Biaxial Stresses", by Hegemier, Nunn, and Arya of the University of California at San Diego (UCSD) gives some results of the NSF research program at UCSD. In describing the general approach to their theoretical and experimental studies the authors state, "the experimental effort is intended to define material behavior and the behavior of typical connections in concrete masonry structures. The analytical phase involves the translation of observed experimental data into viable mathematical models." Their paper provides interaction curves for principal stresses for concrete and for initial macrocracking for masonry. These curves indicate negligible interaction for biaxial principal tensile stresses for concrete and masonry.

"Grout-Block Bond Strength in Concrete Masonry", by Nunn, Miller, and Hegemier gives results of tensile core tests of the block-grout interface in a cast wall. The tensile values varied from 0 psi where the material surfaces were poor to over 300 psi where a good surface was used. Specifically, the low bond strengths were attributed by the authors to powder on the surface of the blocks, lack of vibration, and lack of admixtures. Good construction and quality control contributed to values of bond in the higher range. The mean tensile values for various types of grout varied from 83 to 155 psi, and the standard deviations varied from 76 to 98.

Several other publications were also reviewed in detail. The following is an illustration of a publication that we considered to be relevant.

The book, "Reinforced Brick Masonry and Lateral Force Design", by Plummer and Blume, contains test results on the tensile bond between mortar and brick, as well as values for the bond between grout and brick. On Page 16, the authors conclude, "Results of the mortar specimen tests were very erratic and inconclusive; however, the results obtained from the grout specimens were quite consistent and indicate higher tensile bond strengths than are generally reported for mortared specimens." These bond tensile stresses varied from 31 to 184 psi depending on the type of brick and type of grout.

A telephone survey was also made to assist in the evaluation. The Army Corps of Engineers, Waterway Experiment Station, TVA, PCA, Masonry Institute, Authors of Masonry Handbook, Master Builders Co., U.C. Berkeley, UCSD, a consultant for the Clay, Brick and Tile Institute, and the University of Clemson were all consulted. The information obtained by this survey included results of tests of high strength concrete to concrete where the bond was found to be about 5 percent of the compressive strength of the concrete. Also, shear-bond push-out strengths on cell fills have been found to be in the range of 200 to over 600 psi for standard block with 2700 to 3100 psi compressive grout strength and 3500 to 4000 psi net block strength. Several of the experts suggested that tensile bond is between 5 and 10 percent of the ultimate strength of the material being considered. One recommendation was to use a value of 50 psi for working stress design and 150 psi for ultimate strength design for the tension between concrete and block.

The available data gathered in the literature search, although not always directly bearing on our application, along with the telephone discussions, gave confidence that our expected ultimate strengths were realistic. We therefore conclude that monolithic action for the loads imposed will indeed exist at the concrete to block interface.

Nevertheless, the information obtained from the literature search and the survey of experts does not provide a conclusive basis for selection of a value for the tensile bond at Trojan for composite walls. Therefore, a testing program was conducted to confirm the accuracy of our judgment with respect to the tension capacity at the block-concrete core interface at Trojan. The tests have been completed, and an analysis of the results is in progress.

The maximum tension developed at the interface of the masonry unit necessary to qualify the entire wall thickness to act monolithically to support the piping restraint loads in the Trojan Plant is on the order of 25 psi. This is much less than the strength demonstrated by the in-situ tests of the composite walls and thus the testing confirms the validity of our assumption of 50 psi as a criteria for tensile and shear bond strength in the evaluation of these walls.

4. <u>Tension and Shear Transfer Between Masonry and Mortar</u>

Bechtel also reviewed the UBC and other sources for information with respect to the question of tensile bond and shear transfer at the masonry block - mortar interface. The UBC addresses flexural tension strength across a

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vertical mortar joint, but does not give values for such strength. One paper from "Proceedings of the North American Masonry Conference" also addresses tensile strength. "Structural Properties of Block Concrete", by Holm of Solite Corporation, contains test data showing the tensile strength of concrete block to be in excess of 10 percent of the compressive strength. Based on the formula of $0.75 \times 6.7 (f'm)^{1/2}$ derived from the test the tensile strength for 2000 psi block which is the minimum value at Trojan was calculated to be 225 psi.

The UBC does give a value of 12 psi for flexural tension for mortar in a bed or horizontal joint. It is important to note that the block used on Trojan is an open end concrete masonry unit. About half of the block joint length at one end was filled with grout. Thus, the structural joint is not mortar but grout bonded to the adjacent block.

The evaluation described in LER 79-15 has not relied on the tensile bond in mortared double wythes reinforced conrete block walls. The criteria outlined in Attachment 4 do rely on shear bond between wythes to the extent that the walls act monolithically in the event of seismic excitation to withstand its own out-of-plane inertial loading.

Although the UBC does not explicitly address the question of shear stress in a vertical surface between wythes, that stress must occur along with horizontal bending unless multiple wythe construction was prohibited. Since this is not the case, we believe that 12 psi allowable stress in Table 24-B, multipled by 1.5 for factored loads, can be appropriately used for vertical as well as horizontal surfaces, providing good mortar contact exists.

We believe there are good qualitative arguments for considering VQ/It shear strength while ignoring tensile strength at the present time.

1. Aggregate interlock is effective in providing shear resistance. Confinement for normal forces is provided by tension ties at 4-ft centers each way.

2. Shear resistance does not require continuous bonding between wythes thus permitting mortar gap or local weakness. There is no tendency for a local failure to propagate along the wythes as is the case for tensile bond.

The following two papers given at the North American Masonry Conference are also of interest.

"Effect of Grouting on the Strength Characteristics of Concrete Block Masonry", by Hamid, Drysdale, and Heiderbracht of McMaster University, gives the results of compression tests on grouted masonry where the compressive stresses varied from 1500 psi to 2300 psi. Some of the failure modes were by tensile splitting. Since the compression stresses in the Trojan walls are well below 1500 psi, this failure mode is not expected.

The paper, "Masonry Panels: Review, Present Use and Design", contains recommended design stress values for high bond mortars. For tensile-flexural design, 30 percent of the ultimate test tensile strength is recommended. This illustrates that, with good material, very high values can be achieved.

In summary, we believe there is ample justification to conclude that shear and tensile bond do exist between masonry units and mortar provided that there is good mortar contact. Hence, we consider it appropriate to include mobilization of all wythes in determing inertial forces and global bending resistance.

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ATTACHMENT 1

SAMPLE CALCULATIONS

Introduction

The sample calculations are presented in two sections. Section A illustrates the evaluation of a typical double wythe reinforced masonry wall using the criteria in Attachment 4.

Prior to the complete development of the criteria in Attachment 4, certain more conservative rules were used to evaluate the double masonry wythe walls in order to permit early implementation of modifications. Section B illustrates the method used.

Any further evaluation of double wythe masonry walls will use the method in Section A.

Section A

For the purpose of illustration, a 14 inch thick wall, consisting of wythes of nominal thickness of 8 and 6 inches respectively, spanning vertically between reinforced concrete slabs between elevations 75'-6" and 63'-6" in the Fuel Building is considered. The loads on the example wall due to pipe restraint will be progressively increased to explain the various limiting situations as described in LER 79-15. For tensile pipe load, with the conservative criterion of neglecting tension bond between wythes, the 6 inch wythe section is investigated. The wall is allowed to act as a monolithic unit for the purpose of withstanding its own out-of-plane inertia loading. As discussed in Attachment 5, aggregate interlock and less stringent requirements for mortar bond than for tensile loads facilitate this behavior. However, in the following simplified approach, the 6 inch wythe is considered detached for the tensile load and its response characteristics is accordingly assessed.

1.0 Material properties

$f_m' = 2,000$ psi

$f_y = 40,000$ psi

$f_s = 20,000$ psi

$f_m = 0.33 \times 2000 = 660$ psi

$v_m = 50$ psi

$E_m = 1000\, f_m' = 2 \times 10^6$ psi

$E_s = 30 \times 10^6$ psi

$$n = \frac{E_s}{E_m} = 15$$

$$r = \frac{f_s}{f_m} = \frac{20,000}{660}$$

$$= 30.3$$

$$k_{balanced} = \frac{n}{n+r}$$

$$= 0.33$$

2.0 Transverse bending capacity of Wall

2.1 Wall of 14" nominal thickness

For the purpose of this calculation, the 8" wythe section is considered in tension. The moment capacity obtained when the 6" wythe is in tension will be larger. However, for this illustration the same moment capacity is conservatively assumed for both the cases.

$t = 13\frac{5}{8} = 13.625"$

$d = 13.625 - \frac{1}{2}(7.625) = 9.8125"$

$p = \frac{0.1}{12 \times 9.8125} = .00085$ (based on #4 @ 24" o.c.)

$np = 15 \times .00085 = .0127$

$k = \sqrt{(np)^2 + 2np} - np$
$= 0.1474 < k_{balanced}$

.·. steel controls

S = Bending capacity using working stress design approach

$= 0.10 \times 20 \times 9.8125 \left(1 - \frac{0.1474}{3}\right) \times \frac{1000}{12}$

$= 1555$ lbs-ft /ft

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M_c = Bending capacity for factored load
= 1.5 S
= 2333 lbs-ft /ft

2.2 Wythe of 6" nominal thickness

$t = 5\frac{5}{8}" = 5.625"$

$d = \frac{1}{2}(5.625) = 2.8125"$

$p = \frac{A_s}{bd} = \frac{0.10}{12 \times 2.8125} = .00296$ (based on #4 @ 24" o.c.)

$np = 15 \times .00296 = .044$

$k = \sqrt{(np)^2 + 2np} - np$
$= 0.257 < k_{balanced}$
$\therefore$ steel controls

S_6 = Bending capacity using working stress design approach
$= 0.10 \times 20 \times 2.8125 \left(1 - \frac{0.257}{3}\right) \times \frac{1000}{12}$

= 430 lbs-ft /ft

M_6 = Bending capacity for factored load
= 1.5 S
= 640 lbs-ft /ft

3.0 Transverse bending due to wall's inertia load

3.1 Wall of 14" nominal thickness

3.1.1 Moment of Inertia

The moment of inertia of the wall is taken as

$$I_a = \frac{1}{2}(I_g + I_c)$$

Reference: BC-TOP-9A, Rev. 2
Section 4

Where:

I_g = moment of inertia of gross cross section about its centroid (neglecting steel area)

I_c = moment of inertia of the cracked section

$$I_g = \frac{12 \times 13.625^3}{12}$$

$$= 2529 \text{ in}^4$$

$$I_c = \left(\frac{12 \times 1.446^3}{3}\right) + (15)(0.1)[(9.8125)(1-0.1474)]^2$$

$$= 117 \text{ in}^4$$

$$I_a = \frac{1}{2}(2529 + 117)$$

$$= 1323 \text{ in}^4$$

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3.1.2 Natural frequency of vibration

The wall's natural frequency, f, is dependent on its end conditions. The 14" thick double-wythe reinforced concrete block walls span vertically between the floor slabs which are generally 24" thick reinforced concrete with the wall's vertical reinforcement fully anchored into the slab. The bending stiffness of the restraining slabs, with the conservative assumption of slab span equal to the vertical span of the wall, is approximately 8 to 10 times larger than even the uncracked stiffness of the wall. The wall, therefore, will be assumed to be fixed at its ends. The frequency, f is calculated for a strip 12" wide:

$$f = \frac{7.12}{2L^2}\sqrt{\frac{EIg}{A\gamma}}$$

Reference: Vibration Problems in Engineering by Timcshenko and Young 3rd Edition - Page 337.

where:

L = vertical span of wall = 132 inches
$E = 2x(10)^6$ psi
$I = 1323$ in^4
$g = 386.4$ in/Sec^2
$A = 13.625x12 = 163.5$ inches^2
$\gamma = (138) \div (1728)$ lbs/inches^3

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$$= \frac{7.12}{2(132)^2}\sqrt{\frac{(2.0)\times(10.)^6 \times 1323 \times 386.4 \times 1728}{163.5 \times 138}}$$

$= 57.2$ cps

3.1.3 Walls inertia load and transverse bending

Acceleration at elv. 75'-6" for $\beta = 5\%$ and $f = 57.2$ cps $= 0.36g$

Out-of-plane inertia load

$$w = (138) \times \frac{13.625}{12} \times 0.36$$

$= 56.4$ lbs/ft^2

$$M_I = 56.4 \times \frac{11^2}{12}$$

$= 569$ lbs-ft /ft

3.1.4 Wall's reserve moment capacity for pipe restraint load

$$(M_R)_{14} = M_c - M_I$$
$= 2333 - 569$
$= 1764$ lbs-ft /ft

3.2 Wythe of 6" nominal thickness

3.2.1 Wall's inertia load and transverse bending

Acceleration at elv. 75'-6" for β = 5% and f = 23.6 cps
= 0.36g (See Subsection 3.1.3 of this Section)

The out-of-plane inertia load

$$w_6 = (138) \times \frac{5.625}{12} \times 0.36 = 23 \text{ lbs/ft}^2$$

Transverse bending moment due to wall's inertia

$$M_I = 23 \times \frac{11^2}{12} = 234 \text{ lbs-ft/ft}$$

3.2.2 Wythe's reserve moment capacity for pipe restraint load

$$\begin{aligned}(M_R)_6 &= M_6 - M_I \\ &= 640-234 \\ &= 406 \text{ lbs - ft/ft}\end{aligned}$$

4.0 Wall evaluation to resist pipe load

The pipe restraint load resulting from seismic and thermal with appropriate load factors is taken as a normal load to the wall for the purpose of this illustration. The pipe restraint loads which are supported by brackets to cause bending in the wall are treated in a similar fashion and are not therefore, dealt with in this example. As explained before, for the purpose of this illustration, the pipe restraint load is progressively increased to cover various situations as described in the LER.

P_i = Factored pipe restraint load normal to the wall for case (i) resulting from combined seismic and thermal conditions. The normal load is considered to be resisted by an equivalent width of the nominal wall equal to six times thickness of one wythe (UBC 1967-Section 2418 (c)), which for a 6 inch wythe is 2.8 feet. Also, the restraint is assumed to be attached to the wall on the 6 inch wythe side.

M_p = Transverse bending moment in the wall due to the load P. The load P is considered in this example to act at one-third span to create maximum wall moment in the fixed end condition.

Case 1 $\quad P = 300$ lbs.

$\quad = 107$ lbs/ft. width of wall

$$M_p = \frac{4PL}{27} = \frac{4 \times 107 \times 11}{27} = 175 \text{ lbs-ft}$$

If P is tensile, $(M_R)_6 > M_p$ (406 lbs-ft > 175 lbs-ft)

Hence, the wall is considered adequate and no modification is necessary.

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Case 2 $P = 900$ lbs

$= 321$ lbs/ft. width of wall

$$M_p = \frac{4 \times 300 \times 11}{27}$$

$= 524$ lbs-ft

• For tensile P, $(M_R)_6 < M_P$ (406 lbs-ft < 524 lbs-ft)

Hence, in this case thru-bolting would be done to mobilize the entire wall by activating both the wythes to act together.

$P = 900$ lbs

$$= \frac{900 \times 12}{6 \times 13.625} = 132 \text{ lbs/ft width of wall}$$

$$M_p = \frac{4 \times 132 \times 11}{27} = 215 \text{ lbs/ft}$$

$(M_R)_{14} > M_P$ (1764 lbs-ft > 215 lbs-ft)

Case 3 $P = 7300$

$$= \frac{7300 \times 12}{6 \times 13.625}$$

$= 1071$ lbs/ft width of wall

$$M_P = \frac{4 \times 1071 \times 11}{27}$$

$= 1746$ lbs-ft

or, $M_P = (M_R)_{14}$ (1746 lbs-ft = 1764 lbs-ft)

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Check horizontal shear

Maximum shear,

$$V = \frac{Pa^2}{L^3}(a + 3b) + \frac{wL}{2}$$

where: $a = \frac{2L}{3}$

$b = \frac{L}{3}$

$$\text{or, } V = \frac{20}{27}P + \frac{11}{2}w$$

$$= 1112 \text{ lbs}$$

$$v = \frac{VQ}{It}$$

$$= \frac{1112 \times 12 \times 62}{1323 \times 12}$$

$$= 0.88 \text{ psi} < 1.5 \times 12 = 18 \text{ psi}$$

Case 4 $P > 7300$ lbs

In this case the normal load is more than the wall resistance. Therefore, the wall will be off-loaded by suitable support alteration.

5.0 In-plane shear load on wall combined with pipe load

Let, length of wall = 20 ft.
in-plane shear force = 100 kips

$v = \frac{V}{jLt}$ Reference: UBC 1967-Section 2417

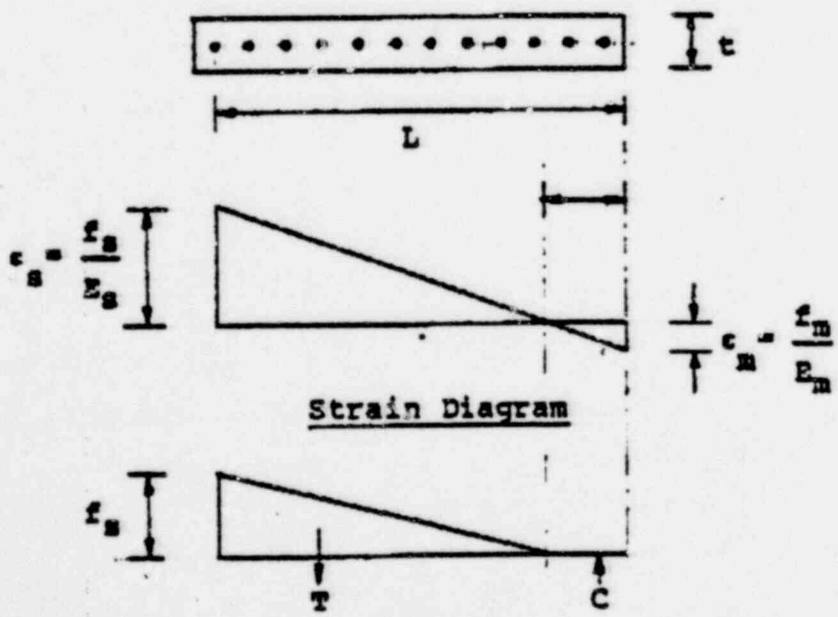
$= \frac{100 \times 1000 \times 3}{2 \times 20 \times 12 \times 13.625}$

$= 44.4$ psi $< 1.5 \times 50 = 75$ psi

5.1 Global in-plane wall bending
Considering the wall fixed at top and bottom

$M = \frac{100 \times 11}{2}$

$= 550$ k-ft



Strain Diagram

Stress Diagram

a_s = area of steel (in^2/in)

$= \frac{0.2}{12}$

$= 0.0167\ in^2/in$

$n = 15$

$t = 13.625$ inches

$T = 1/2\ f_s \times a_s \times L\ (1-k)$

$C = 1/2\ kL \times t \times f_m$

$T = C$

or, $f_m = \frac{f_s a_s}{t} (\frac{1}{k} - 1)$ ... (1)

From the strain diagram,

$$\frac{f_s}{Es} \times \frac{E_m}{fm} = \frac{L-kL}{t} (\frac{1}{k} - 1)$$

Substituting the value of f_m from equation (1) and $\frac{E_s}{E_m} = n$

$$(\frac{1}{k} - 1)^2 = \frac{nt}{a_s}$$

$$(\frac{1}{k} - 1)^2 = \frac{15 \times 13.625}{.0167}$$

$$= 12{,}240$$

or, $k = .0088$

$$M = \frac{1}{2} f_s a_s \; L(1-k) \times \frac{2L}{3}$$

$$\text{or, } f_s = \frac{3M}{a_s L^2 \; (1-k)}$$

$$= \frac{3 \times 550 \times 12}{0.0167 \times (20 \times 12)^2 \times 0.9912}$$

$$= 20.8 \;\; \text{kips/in}^2$$

This is the stress in the reinforcing steel at the farthest end of the tension zone. The steel stress decreases linearly up to the neutral axis where the stress is zero.

Having thus obtained the maximum steel stress in the reinforcing steel at a particular wall location, the steel stress due to the pipe restraint loads, considering individual wythes of the wall, would be directly added to it to determine the final stress. The resulting stress should be less than 1.5 x 20 = 30 ksi. If the stress exceeds this magnitude, the entire wall would be mobilized by thru bolting, and the stress in the rebar due to transverse bending for the pipe load would be recalculated. If the final steel stress was still larger than the allowable limit, the wall would be off-loaded and alternate support arrangement implemented.

Section B

The principal differences in the calculation procedure in this Section from that in Section A are as follows:

- Wall's capacity for factored loads is taken as 1.33 times UBC working stress capacity (in Section A capacity is taken 1.5 times UBC working stress capacity).

- The concentrated pipe restraint load is considered to be resisted by a width of wall (or wythe) equal to (b + 2t) where, b = base plate width and
  t = thickness of wall (or wythe)
  (in Section A the resisting width is taken as 6t).

1.0 Material properties

Same as Section A

2.0 Transverse bending capacity of wall

2.1 Wythe of 8" nominal thickness

$$t = 7\tfrac{5}{8}" = 7.625"$$

$$d = \tfrac{1}{2}(7.625) = 3.8125"$$

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$$p = \frac{0.1}{12 \times 3.8125} = .0022 \text{ (based on \#4 @ 24" o.c)}$$

$$np = 15 \times .0022 = .0328$$

$$k = \sqrt{(np)^2 + 2np} - np$$
$$= 0.2254 < k_{balanced}$$

$\therefore$ steel controls

S_s = Bending capacity using working stress design approach

$$= 0.10 \times 20 \times 3.8125 \left(1 - \frac{0.2254}{3}\right) \times \frac{1000}{12}$$

$$= 588 \text{ lbs-ft /ft}$$

M_s = Bending capacity for factored load

$$= 1.33\ S$$
$$= 782 \text{ lbs-ft /ft}$$

2.2 Wythe of 6" nominal thickness

$$t = 5\frac{5}{8}" = 5.625"$$

$$d = \frac{1}{2}(5.625) = 2.8125"$$

$$p = \frac{A_s}{bd} = \frac{0.10}{12 \times 2.8125} = .00296 \text{ (based on \#4 @24" o.c.)}$$

$$np = 15 \times .00296 = .044$$

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$$k = \sqrt{(np)^2 + 2np} - np$$
$$= 0.257 < k_{balanced}$$
$\therefore$ steel controls

S_6 = Bending capacity using working stress design approach

$$= 0.10 \times 20 \times 2.8125 \left(1 - \frac{0.257}{3}\right) \times \frac{1000}{12}$$

$$= 430 \text{ lbs-ft /ft}$$

M_6 = Bending capacity for factored load

$$= 1.33\ S$$
$$= 572 \text{ lbs-ft /ft}$$

3.0 Load Capacities

3.1 Punching

Allowable stress $= 2\sqrt{f_m'} \times 1.33$ Reference:
$= 120$ psi UBC-1967, Section 2612(h)

Since the UBC does not explicitly specify any value for block, the formula given for concrete is used with the value f_m' for grouted masonry replacing f_c' for concrete. As can be seen below, the capacity thus obtained is much greater than the applied load.

Taking a base plate of smallest dimension 6" x 6" The resisting force P_v is:

$$P_v = 4(b + d)d \times 0.12$$

where $b = 6"$

d = effective depth of 14" wall

$= 13.625 - 1/2\ (7.625)$

$= 9.81"$

$$\therefore P_v = 4(6 + 9.81) \times 9.81 \times 0.12$$

$= 77.4$ kips

3.2 Pullout

This was based on the strength of one wythe only

$\therefore d = 1/2\ (5.625) = 2.81"$

The pullout capacity P_t is

$$P_t = 4(6+\frac{d}{2})\ \frac{d}{2} \times 0.12$$

$= 4(6+2.81) \times 2.81 \times 0.12$

$= 11.9$ kips

4.0 Transverse bending due to wall's inertia load

The following calculations are based upon considering the wythes acting separately in bending. This condition was postulated to occur due to tensile load of the pipe restraints causing tensile delamination.

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4.1 Moment of inertia

$$I_a = \frac{1}{2}(I_g + I_c)$$

Where:

I_g = moment of inertia of gross cross section about its centroid (neglecting steel area) of a 14 in. block wall.

I_c = Moment of inertia of the cracked section

I_a = 1323 in^4 (See Subsection 3.1.1 of Section A)

4.2 Natural frequency of vibration of wall

f = 57.2 cps (see Subsection 3.1.2 of Section A)

4.3 Wall's inertia load and Transverse bending

Acceleration at elv. 75'-6" for β = 5% and f = 57.2 cps

= 0.36g Reference: Floor Spectra for β = 5%

The out-of-plane inertia load

(a) for 8" wythe

$$w_8 = (138) \times \frac{7.625}{12} \times 0.36 = 31.6 \text{ lbs/ft}^2$$

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(b) for 6" wythe

$$w_6 = (138) \times \frac{5.625}{12} \times 0.36 = 23.3 \text{ lbs/ft}^2$$

Transverse bending moment due to wall's inertia

$$M_8 = 31.6 \times \frac{11^2}{12} = 319 \text{ lbs-ft /ft}$$

$$M_6 = 23.3 \times \frac{11^2}{12} = 235 \text{ lbs-ft /ft}$$

5.0 Wall capacity for loads other than its own inertia load

Reserve moment capacity of wall, M_R, to resist other loads, such as from pipe restraint, is calculated as the difference between each wythe's capacity, as calculated in Subsection 2.0 of this Section and the moment produced due to each wythe's inertia load.

For 8" wythe,

$$(M_R)_8 = 782-319$$
$$= 463 \text{ lbs-ft /ft}$$

For 6" wythe

$$(M_R)_6 = 572-235$$
$$= 337 \text{ lbs-ft /ft}$$

Reserve moment capacity of wall under a compressive load

$$(M_R)_{14} = (M_R)_8 + (M_R)_6 = 463 + 337 = 800 \text{ lbs ft/ft}$$

Reserve moment capacity of wall under a tensile load

$$(M_R)_6 = 337 \text{ lbs-ft/ft}$$

The pipe restraint load resulting from seismic and thermal with appropriate load factors is taken as a normal load to the wall for the purpose of this illustration. The pipe restraint loads which are supported by brackets to cause bending in the wall are treated in a similar fashion and are not therefore, dealt with in this example. As explained before, for the purpose of this illustration, the pipe restraint load is progressively increased to cover various situations as described in LER 79-15.

P_i = Factored pipe restraint load normal to the wall for case (i) resulting from combined seismic and thermal conditions. The normal load is considered to be resisted by an equivalent width of the wall equal to (b + 2t)

where:

b = side dimension of base plate, in.

t = thickness or wall
= 13.625" when load is compressive
= 5.625" when load is tensile

M_p = Transverse bending moment in the wall due to the load P. The load P is considered in this example to act at one-third span to create maximum wall moment in the fixed end condition.

Case 1 P = 200 lbs.

If P is compressive,

$$P = \frac{200 \times 12}{6 + (2 \times 13.625)} = 72 \text{ lbs/ft width of wall}$$

$$M_p = \frac{4PL}{27} = \frac{4 \times 72 \times 11}{27} = 118 \text{ lbs-ft}$$

$$(M_R)_{14} > M_p \quad (800 \text{ lbs-ft} > 118 \text{ lbs-ft})$$

If P is tensile,

$$P = \frac{200 \times 12}{6 + (2 \times 5.625)} = 139 \text{ lbs/ft width of wall}$$

$$M_p = \frac{4PL}{27} = \frac{4 \times 139 \times 11}{27} = 227 \text{ lbs-ft}$$

$$(M_R)_6 > M_p \quad (337 \text{ lbs-ft} > 227 \text{ lbs-ft})$$

Hence, the wall is considered adequate and no modification is necessary.

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Case 2 $P = 400$ lbs

If P is compressive,

$$P = \frac{400 \times 12}{6 + (2 \times 13.625)} = 144 \text{ lbs/ft width of wall}$$

$$M_P = \frac{4\ PL}{27} = \frac{4 \times 144 \times 11}{27} = 235 \text{ lbs-ft}$$

$$(M_R)_{14} > M_P \quad (800 \text{ lbs-ft} > 235 \text{ lbs-ft})$$

Wall is adequate in compression

If P is tensile

$$P = \frac{400 \times 12}{6 + (2 \times 5.625)} = 278 \text{ lbs /ft width of wall}$$

$$M_P = \frac{4 \times 278 \times 11}{27} = 453 \text{ lbs-ft}$$

$$(M_R)_6 < M_P \quad (337 \text{ lbs-ft} < 453 \text{ lbs-ft})$$

The 6" wythe is not adequate under tensile load, therefore, thru-bolts should be used to engage both wythes.

Case 3 $P = 1400$ lbs

$$P = \frac{1400 \times 12}{6 + (2 \times 13.625)} = 505 \text{ lbs/ft width of wall}$$

$$M_P = \frac{4\ PL}{27} = \frac{4 \times 505 \times 11}{27} = 823 \text{ lbs-ft}$$

$$(M_R)_{14} < M_P \quad (800 \text{ lbs-ft} < 823 \text{ lbs-ft})$$

The wall is not adequate, the pipe support needs modification to unload the wall.

In-plane shear and bending have not yet been evaluated since the majority of the 14 inch walls are below elevation 45' where no differential deflection exists, hence no in-plane forces. Those walls above elevation 45' will be reviewed by methods in Section A of this Attachment prior to plant startup.