ATTACHMENT 6 ; (Response to items 3 and 4 of Attachment 2) PI2320

PILGRIM NUCLEAR GENERATING STATION - UNIT 1

STATISTICAL ANALYSIS OF BOUNDARY STRENGTHS FOR MASONRY WALLS FROM FIELD TEST DATA

Prepared For

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1 INTRODUCTION

This report describes a series of statistical analyses performed by Computech Engineering Services, Inc. (CES) for Bechtel Power Corporation (BPC). The purpose of the analyses is to evaluate the boundary strengths of the masonry walls at Pilgrim Nuclear Generating Station, Unit 1.

In the latter part of 1981 and early 1982, field inspection of the anchorage conditions at the boundaries of the masonry walls at Pilgrim was undertaken. This work was performed under the direction of Cygna Energy Services. The work presented herein is the statistical analysis of the existing test data, and the incorporation of the results of this analysis into strength calculations for the boundaries of the masonry walls at Pilgrim Station, Unit 1. The end result is a set of boundary allowables for use in the evaluation of the walls.

Section 2 of the report describes the boundary types under consideration. Sections 3 and 4 describe the test procedures and test findings respectively. The statistical analysis of the test results is briefly described in Section 5, and the calculation of boundary strengths is described in Section 6. Section 7 presents a summary of boundary strengths for use in the evaluation of the walls, and the report closes with conclusions in Section 8. In Appendix A a detailed description of the statistical analyses is presented.

2 BOUNDARY TYPES

In this section, the various types of boundaries found in the walls at Pilgrim are described. The top boundaries are described first, followed by the sides.

2.1 Top Boundary

There are essentially three types of top boundaries, although for strength purposes, one of these types is further sub-divided.

A. Metal Q-Decking. This is a standard ribbed metal deck over which is poured a concrete slab forming the floor. Design drawings call for self drilling anchors at 16" centers and for grout to be forced into the space between the top of the wall and the ribs of the Q-deck. The ribs of the metal deck generally run either perpendicular or parallel to the wall. It is because of this directional distinction that this top boundary category will later be further divided into two sub-categories.

B. Structural Steel WF Section. For this top boundary condition, the wall butts up against a structural steel section. Anchors are specified to be welded to the flange of the steel section at 16" centers.

C. Concrete. In this condition, the top of the wall butts up against structural concrete (typically a concrete slab). Again, anchors are specified at 16" centers.

2.2 Side Boundary

There are four distinct conditions found at the side boundaries of the walls.

A. Structural Steel WF Section. In this condition, the side of the wall is in contact with a structural steel section. Anchors are specified (welded to the flange) at bond beam locations only, with a minimum of one anchor for every two horizontal reinforcing bars.

B. Concrete. In this case the side of the wall is in contact with structural concrete (typically a column or another wall). Anchors are specified at bond beam locations only, with a minimum of one anchor for every two horizontal reinforcing bars.

C. Intersecting Masonry L. For this boundary condition, the side of the wall meets another masonry wall (at right angles). Joint or horizontal reinforcement in the wall is specified to be continuous around the L-joint. Interlocking of blocks is also required.

D. Intersecting Masonry T. In this case the side of the masonry wall forms a T junction with another masonry wall. Joint (Dur-o-wall) reinforcement is specified as continuous through the T-joint. For this boundary type, interlocking of blocks will depend on construction sequence, and cannot be generally guaranteed.

3 TEST PROCEDURES

The test procedures used to examine the walls for anchorages are now described. The procedures as required by Cygna are first presented, followed by an interpretation of the compliance with the procedures as deduced from the reporting of test results.

3.1 Specified Procedures

The following procedures are taken from Cygna document 80034, WI-6, Revision 3, January 1982, "Work Instruction for Testing of Masonry Walls".

Along the top edge of the wall, three consecutive blocks were required to be cut away. The cut into the block was to be to a minimum depth of one half the block thickness (if anchorage is found), and to a maximum of the entire depth of grout in the cells, care being taken not to break through the far face shell of the block. Furthermore, if only one dowel was observed in the section of the wall that was cut away, an additional amount of block was to be cut away along the boundary, to expose a minimum of 24" on either side of the observed dowel. The wall was considered to be in accordance with the design drawings if a dowel was observed in each block that was cut away. (This corresponds to the required 16" spacing.)

Along the side boundaries, four consecutive blocks were to be cut away. The requirements for the cuts were the same as for the top cuts, including the 24" exposed length. The wall was considered to be in compliance with the design drawings if at least half of the exposed blocks contained dowels positively anchored to the adjacent structure. This corresponds to a 16" spacing as required.

After inspection of the cut-away blocks for the existence of dowels, and the documentation of their size, number and location, along with the quality of grout and welds, all portions of the walls disturbed or damaged by the testing were to be restored with grout.

3.2 Compliance with Procedures

This section evaluates the compliance of the anchorage tests with the appropriate instruction in "Work Instruction for Testing of Masonry Walls". In general, the individual tests have not been carried out in a consistent manner and in accordance with the instruction.

1. In testing for top anchorages, the work instruction requires 3 consecutive blocks to be inspected for dowels, and the wall is deemed satisfactory only if all 3 blocks do have dowels. Also, number and location of dowels were to be reported in the test results. This has not been done in all cases. The wall would be deemed to "fail" the test if 0, 1 or 2 dowels were observed. As a wall would "fail" the anchorage test as soon as a block is found with no anchor, it appears that many tests have stopped short of

the 3-block requirement, although some have gone more than the 3-block requirement, perhaps looking for anchorages. If the data had been collected as specified, then the additional information would have been statistically useful in the context of a revised analysis of the data.

2. In testing for side anchorages, the work instruction requires 4 consecutive blocks to be "chipped," and the wall is deemed satisfactory if at least 2 anchorages are found. This requirement is reasonable for the wider multiwythe walls where 6 bars are specified every 8 feet. However, for the 8" and 12" walls only 1 or 2 bars are specified every 8 feet where called for. Thus, even if these walls were built in accordance with the drawings, they would be very likely to "fail" the side anchorage tests.

   Again, as for the top anchorages, some violation of the 4-block requirement is apparent, although the side tests conform better to the work instructions than do the tests for anchorage on the tops of the walls.

Anomalies such as those mentioned above tend to give a higher scatter in the test data than the inherent variability due to variations in wall anchorages alone.

3.3 General Comments on Procedures

The statistical procedure used as the basis of the field testing, although sophisticated, is not suited for the walls at Pilgrim. Also, application of the procedure in the field has not followed the basis of the method. Nonetheless, on examination of the test data and the criteria developed for evaluating the test data by Cygna, the conclusion that the anchorages do not conform to design drawings is correct. However, the same conclusion could have been reached with substantially less testing. Also, given the amount of data collected, the final conclusion made by Cygna that no anchorages can be relied upon is unfairly harsh on the evaluation of the wall anchorages, since not all the specified anchorages are required for the structural integrity of the walls.

It is the position of CES that the purpose of the test program was incorrectly defined. The objective should have been to determine what percentage of the anchorages could be relied upon (with a given confidence) rather than a yes or no on all of the anchorages being present. Although the test data does indicate that the anchorages are not in accordance with the design drawings, taking absolutely no account of boundary strengths (i.e., presuming that there are zero anchors present) is the most conservative stance possible.

It is, however, possible to use the existing data as the basis for a statistical reassessment of the state of the wall anchorages at Pilgrim Station Unit 1, to determine the percentage of anchorages present (with a given confidence) for each of the boundary types present at the plant. The original test data has therefore been re-examined with the aim of extracting a percentage of anchors which can be relied upon in any given wall. This is briefly described in Section 5 of this report, while the details of the statistical analyses are presented in Appendix A. The resulting boundary strengths are summarized in Section 7.

4 TEST FINDINGS

A summary of the results from the relevant field tests is given in Tables 4.1 (top boundary) and Table 4.2 (side boundary). A total of 51 useful tests were performed on the top boundaries and 37 on the side boundaries. There were other tests performed to examine the boundary anchorages, but insufficient information was recorded during those tests to include them in the data space.

It should be noted that the top boundary table (Table 4.1) does not distinguish between Q-deck (parallel) and Q-deck (perpendicular). This distinction is made later and described more fully in Section 6.2.

It should also be noted that the side boundary table (Table 4.2) has no data on either masonry L or masonry T boundaries. Cygna's conclusions regarding the masonry L and T side boundary types were sufficient for the purposes of this work, and were accepted without further study. The relevant conclusions from Cygna work are that at masonry L side boundaries interlocking between the blocks from the two incoming walls can be relied upon, whereas at masonry T boundaries, no interlocking of blocks between the two sections of the T is apparent. These conclusions are used directly to develop allowable strengths for these boundary types in Section 6.3 of this report.

WALL NUMBER	BOUNDARY TYPE	EXPOSED LENGTH (Inches)	NUMBER OF ANCHORAGES
45.2	Concrete	66	2
62.2	Concrete	48	2
63.5	Q-Deck	53	0
64.4	Steel	38	2
64.4	Steel	36	2
64.6	Q-Deck	51	1
64.12	Concrete	38	2
65.16	Steel	43	1
65.21	Steel	60	1
66.0	Steel	42	1
66.2	Q-Deck	72.5	0
66.5	Q-Deck	28	0
66.5	Q-Deck	16	0
66.6	Steel	13	1
66.6	Steel	51	2
66.6	Concrete	30	2
66.7	Q-Deck	48	2
66.11	Concrete	48	1
66.12	Q-Deck	47	0
66.18	Steel	50	0
67.1	Steel	58	1
68.10	Steel	48	0
111.7	Q-Deck	40	2
184.2	Steel	38	0
184.4	Steel	48	1
184.7	Concrete	48	3
84.8	Concrete	49	3
188.2	Steel	48	0
188.3	Q-Deck	56	3
188.9	Steel	47	0
188.10	Concrete	49	3
191.26	Steel	73	1
191.35	Steel	81	1
191.49	Q-Deck	64	0
191.55	Steel	48	1
194.21	Q-Deck	59	0
194.21	Q-Deck	47	0
194.22	Q-Deck	49	0
194.23	Steel	49	1
194.25	Q-Deck	32	0
195.14	Steel	48	2
195.18	Q-Deck	41	0
195.22	Steel	68	1
195.23	Q-Deck	6	0
195.23	Q-Deck	35	0
195.23	Q-Deck	12	0
196.6	Steel	57.5	2
196.7	Steel	10	0
196.7	Steel	60.5	2
209.0	Concrete	44	1
212.1	Concrete	48	1

TABLE 4.1 : RESULTS FROM TOP BOUNDARY TESTS

WALL NUMBER	BOUNDARY TYPE	EXPOSED LENGTH (Inches)	NUMBER OF ANCHORAGES
45.2	Concrete	33	1
62.2	Concrete	38	2
63.5	Concrete	30	0
64.4	Concrete	36	0
64.13	Steel	37	2
65.21	Concrete	33	1
66.0	Concrete	33	1
66.2	Concrete	48	2
66.5	Concrete	40	2
66.6	Steel	33	2
66.6	Steel	33	2
67.1	Steel	45	2
67.1	Concrete	43	2
68.10	Concrete	52	2
68.10	Concrete	36	2
184.4	Steel	32	1
184.9	Concrete	32	2
188.2	Steel	42	2
188.2	Steel	37	2
188.3	Steel	43	2
188.9	Concrete	36	1
188.10	Steel	38	2
191.26	Steel	33	1
191.35	Steel	54	3
191.35	Steel	65	3
191.55	Steel	69	3
194.2	Steel	32	1
194.2	Concrete	32	0
194.22	Concrete	54	3
194.25	Concrete	32	1
195.9	Steel	55.5	3
195.14	Concrete	48	2
196.6	Steel	59	3
198.0	Concrete	31	1
198.3	Concrete	36	2
209.0	Concrete	38	1
212.0	Steel	33	0

TABLE 4.2 : RESULTS FROM SIDE BOUNDARY TESTS

5 STATISTICAL ANALYSIS OF TEST FINDINGS

In this Section, the statistical methodology used in developing the boundary strengths for the walls at Pilgrim Station Unit 1 is briefly described. The results of the statistical analyses are then summarized for use in Section 7. A detailed presentation of the statistical analyses is made in Appendix A.

5.1 Statistical Methodology

The test data presented in Tables 4.1 (top data) and 4.2 (side data) consists of data taken from inherently different populations. The top data includes tests on three different types of boundaries: Q-deck, structural steel and concrete. The side data includes tests on concrete and steel boundaries. Because of differing construction conditions and requirements for each of these boundary types, there is no reason to believe that each population will have the same underlying distribution. Therefore it is imperative that the data be separated into sub-groups, with each sub-group of data containing results from anchorage tests on similar boundary types. This leads to five sample spaces from five underlying populations.

The statistical methodology adopted for each boundary type is very similar. Data from each boundary type is firstly converted to number of anchorages per unit length, and the analysis is performed on this set of numbers. In some cases it is convenient to perform an "inverse" analysis whereby the data is converted to length per unit anchorage instead. In either case, a percentage of anchorages which can be relied upon with a certain confidence is calculated. This is further described in Appendix A. The confidence level chosen for all analyses is 95%. This level is consistent with previously adopted confidence levels for interpretation of test data for use in the nuclear industry.

For consistency of analysis, it is assumed that the spacing for anchorages on all boundary types is 16". This is <u>not</u> intended to represent the design conditon for each boundary, but rather is a convenience for analysis. "Ideal strengths" are subsequently calculated on this basis, making it a simple matter to factor the results of the statistical analyses into the ideal strength to arrive at a reliable strength.

5.2 Summary of Results

The following results give the percentage of the "ideal anchors" for each boundary type which can be relied upon with 95% confidence. These results are drawn from Appendix A.

1. Top boundary to Q-deck - 0%

2. Top boundary to structural steel - 0%

3. Top boundary to concrete - 19.3% (80 inc...)

4. Side boundary to structural steel - 33% (48 inches)

5. Side boundary to concrete - 20% (80 inches)

"Ideal anchors" are spaced at 16" for all boundary types, as discussed in Section 5.1. Inherent in the percentage of "Ideal anchors" which can be relied upon is a minimum boundary length. Boundaries shorter than this minimum length shall have a zero allowable load. This minimum length is calculated from the ideal spacing (16") and the reliable percentage of "ideal anchors". For the boundaries with zero allowables, this minimum length has no meaning. However, for the remaining boundaries, the minimum lengths are given in parentheses above.

6 IDEAL BOUNDARY STRENGTHS

This Section describes the calculation of the allowable boundary line loads for use in the wall evaluation. Section 6.1 discusses the strength criteria and shear transfer mechanisms appropriate at the various boundaries. Section 6.2 uses the results from Section 6.1 together with the anchorage properties specified on the design drawings, to arrive at ideal boundary strengths. Those boundaries which rely on mechanical interlock for shear transfer (Q-deck parallel and masonry L boundaries) are assigned strengths in Section 6.3.

6.1 Strength Criteria

Chapter 26 of the UBC states that shear friction provisions are appropriate at an interface between dissimilar materials or an interface between concrete cast at different times. It is our belief that this applies to the boundary conditions of the walls at the Pilgrim Station, Unit 1.

An ultimate strength formulation is adopted for the boundary allowables. This is consistent with the extreme nature of the loading conditions considered in the wall and boundary evaluation. Seismic, tornado and PBOC loads are applied to the walls. These loads (SSE, tornado and PBOC) are considered ultimate conditions, and allowable stresses under these loads are generally increased from the usual working stress allowables.

Allowable stresses based on shear friction are derived from the total normal force applied across the interface by the reinforcing and the coefficient of friction between the masonry and the boundary material. For reinforced concrete the total anchorage force is the area of steel times its yield strength. In the case of the masonry walls at Pilgrim, this force is taken as the least of the forces calculated from:

1. the area of steel anchorage times its yield strength.

2. the product of the allowable bond stress around the anchorage times the length of the anchorage times its perimeter.

3. the capacity of the weld or insert of the anchorage bar into the boundary.

The lowest of the three so calculated forces is taken as the limiting normal force across the boundary.

The coefficient of friction is assumed to be 1.0 for concrete boundaries and 0.7 for steel boundaries.

The values for material strengths (such as steel yield strength) have been based on the requirements for the original design, and experimental data has been used for bond and insert capacities as discussed below.

6.1.1 Steel Yield Strength

In all strength calculations requiring a yield strength for steel, the following values have been used:

(a) mild steel rebar: 40 ksi.

(b) steel flat plates and structural sections: 36 ksi.

6.1.2 Weld Strength

The usual permissable weld stress of 21 ksi has been increased by an ultimate strength factor of 1.67, giving an allowable stress of 35 ksi. 1/4" fillet welds all round have been used as the basis for the tensile strength calculation for the welds.

6.1.3 Bond Stress

As a first step in arriving at an allowable bond stress, the values for bond stress inherent in Section 2612 of the UBC were examined. For deformed bars, this section of the code gives the following "development lengths":

$$L_d \not< 0.04\ A_b f_y \ / \sqrt{f'_c}$$

$$L_d \not< 0.0004\ d_b f_y$$

Assuming that the development length is based upon

$$L_d = A_s f_y \ / \ f_b O_b$$

where O_b is the bar perimeter, then these two equations reduce to:

$$f_b = 7.96 \sqrt{f'_c} \ / \ d_b \not< 625 \text{ psi}$$

These equations imply the following bond stresses (as a function of bar diameter) for 3000 psi concrete: for a #5 bar, a bond stress of 625 psi, and for a #6 bar, a bond stress of 581 psi.

From Figure 6.1 (reproduced from Park and Paulay [1] Figure 9.11), for a cover of approximately 5 bar diameters, (typical of cover to a central #5 bar in an 8" block) the allowable bond stress is approximately 1000 psi. To be on the conservative side, a bond stress of 700 psi will be used in all strength calculations for deformed bars. This is entirely consistent with the implied bond stresses in the UBC code for the bar sizes in question (as calculated above). It should be noted that the implied code values should be expected to be conservative. This is again consistent with the values calculated above. From Park and Paulay Figures 9.6 and 9.7 (reproduced herein as Figures 6.2 and 6.3), it can be seen that a value of 300 psi is a reasonable value for bond stress in plain bars.

These two values (700 psi and 300 psi) are used in subsequent strength calculations involving deformed bars and plain bars or flat plates respectively.

6.1.4 Insert Capacity

The allowable forces for embedded concrete inserts are taken from the Clemson tests on such inserts [2]. From the results of this extensive test program, it is reasonable (and on the conservative side) to assign a value of 10 times the UBC value for the ultimate load on these inserts. This provides for the following allowable concrete insert forces: 7500 lbs for #5 bars and 11000 lbs for #6 bars.

6.1.5 Shear Strength of Grout

In the case where the ribs of the Q-deck run parallel to the wall, mechanical interlock exists at the top boundary since the design drawings indicate that grout is forced between the top of the wall and the ribs of the Q-deck. A boundary shear can be developed as a result of this interlock.

The grout at Pilgrim Station Unit 1 is specified to have a minimum strength of 2000 psi, and it is treated as plain concrete. Section 2622(d) of the UBC Code applies, which gives an allowable shear stress in plain concrete of $0.02f'_C$. With a unit strength of 2000 psi, this implies an allowable shear stress of 40 psi in the grout. This allowable stress is used in Section 6.3.

6.1.6 Shear Strength of Mortar

At side boundaries to other masonry walls where interlocking of blocks is demonstrated, boundary line loads can be developed due to shear across the plane of interlock. This is a shear in the mortar between the blocks in question. An allowable shear stress is required for this situation.

The mortar at Pilgrim Station Unit 1 is specified to be type PL of standard STMC-476 (1964), with a minimum 28 day strength of 2000 psi. This standard is now outdated, but type PL corresponds very closely to type S mortar in the latest standard. Type S mortar has a specified minimum strength of 1800 psi.

Colville [3] indicates an average shear bond in type M mortar of 47 psi. It is reasonable to assume in the absence of other data, that the average shear bond for type S mortar can be scaled from the value for type M by the ratio of the minimum compressive strengths for the two mortar types. This gives a value for the average shear bond in type

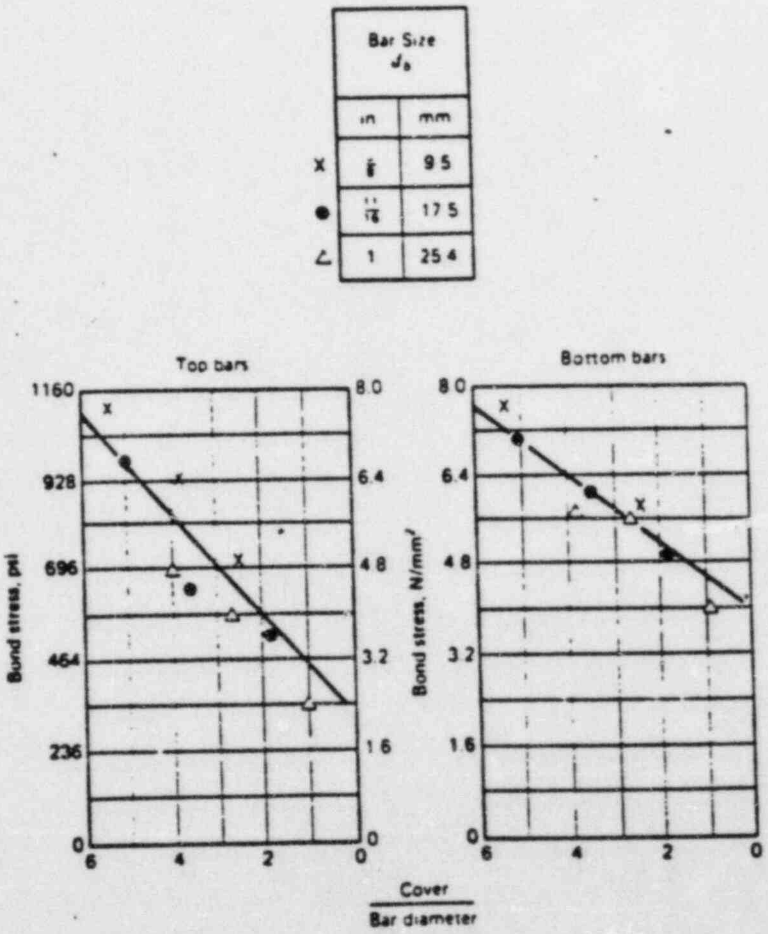

Figure 6.1 : The Effect of Cover on Bond Strength

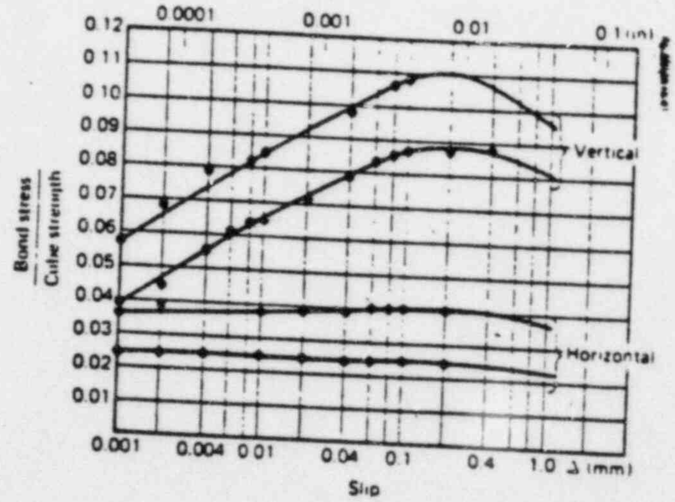

Figure 6.2 : The Load-Slip Relationships for #5 Plain Round Bar

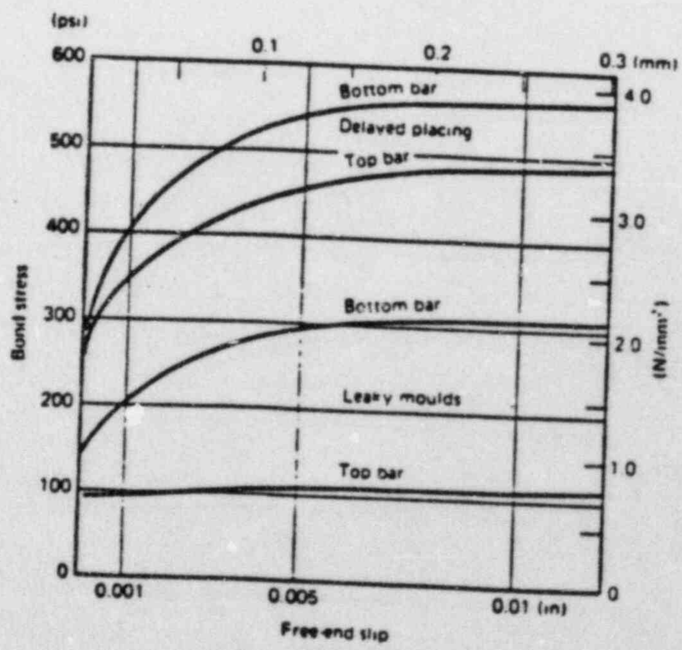

Figure 6.3 : Bond Stress-Slip Relationship for Plain Round Bars (as Affected by the Settlement of Fresh Concrete)

S (and hence PL) mortar of

$$47 \text{ psi} \times 1800/2500 = 34 \text{ psi}.$$

It is reasonable to use an average strength since along any side boundary there are a large number of bed joints where the mortar is in shear. There is also a margin of safety in the calculation since the mortar at the plant was specified with a minimum strength of 2000 psi, and the scaling was done using 1800 psi. Also, due to long term aging, the mortar is likely to have a current strength considerably in excess of either of these two numbers. This value of 34 psi is consistent with the allowable shear for the grout (40 psi) developed in Section 6.1.5 and is used in Section 6.3.

6.2 Ideal Strength (Specified Anchors)

The anchorages specified on the design drawings vary with wall thickness and boundary type. Anchorages along the top edge of the walls are specified at 16" centers for all wall thicknesses. Anchorages along the side edges are specified at bond beam locations only, with a minimum of one anchor for every two horizontal reinforcing bars. Bond beams are specified at 8'-0" centers.

For the case of the top of a wall against a metal deck or a concrete slab, specified anchors are #5 bars at 16" centers for 8" and 12" walls, and two #5 bars at 16" centers for multi-wythe walls. For the case of the top of a wall against structural steel, specified anchors are #5 bars at 16" centers for 8" and 12" walls, two #5 bars at 16" centers for multi-wythe walls less than 2'-6" thick, and two #6 bars at 16" centers for thicker multi-wyhte walls.

For the case of side boundaries one bar every 8'-0" is specified for 8" walls, two bars for 12" walls, three bars for 18" walls, four bars for walls between 2'-0" and 3'-0" thick, and six bars every 8'-0" for 3'-6" walls. From the field test program, these anchors were generally #5 bars. While not specified on any of the design drawings, in many instances side anchorages to structural steel sections were flat plates approximately 1" x 1/4" in section.

The actual anchorage properties used for the ideal strength calculations are thus generally #5 bars (with 1" x 1/4" plates for side boundaries to structural steel), with #6 bars used where specified on the top boundaries. For the side boundary to concrete, #5 bars are used as the basis for boundary strength, as this diameter of bar was most often found at these boundaries (and its capacity is smaller than the #6 bars specified). Lengths of anchorages are taken from the standard details applicable to these walls, as no test data is available on anchorage lengths.

In order to more readily incorporate the findings from the statistical analyses, (which related percentages to an assumed ideal spacing of 16") side boundary ideal strengths are based on 1 anchorage per wythe at 16" spacing. This allows direct scaling of the strength by the percentage of a[illegible]ors which can be relied upon from the statistical analyses, and normalizes the varied spacing requirements for the side anchorages.

On this basis, Table 6.1 is prepared, giving "ideal" boundary strengths under the above assumptions. These strengths are then factored by the results from the statistical analysis of the test data. This is discussed in Section 7.1.

WALL THICKNESS	ALLOWABLE LINE LOAD (lb/in)				
	TOP BOUNDARY			SIDE BOUNDARY	
	Q-DECK	STEEL	CONCRETE	STEEL	CONCRETE
8"	328	480	468	390	468
12"	328	480	468	390	468
1'-6"	656	960	937	780	937
2'-0"	656	960	937	780	937
2'-2"	656	960	937	780	937
2'-6"	656	1156	937	780	937
3'-0"	656	1156	937	780	937
3'-6"	656	1156	937	1170	1404

TABLE 6.1 : IDEAL BOUNDARY STRENGTHS (SHEAR FRICTION FORMULATION)

6.3 Strengths Based on Interlock

Two boundary strengths are considered in this section. The first is the case of a masonry L side boundary where field tests and inspection confirmed that interlocking of blocks at such boundaries is present. The second is the case of the Q-deck top boundary in the situation where the ribs of the metal deck run parallel to the wall.

In each case there is also another boundary of seemingly similar type, namely a masonry T side boundary, and a Q-deck (perpendicular) top boundary. However, for the masonry T side boundary, field inspection and testing indicated that interlocking of blocks could not be assumed at this boundary. Also, the presence of horizontal Dur-o-wall reinforcement was doubtful (conclusion from field tests on this type of reinforcement). Thus a masonry T side boundary is assigned zero strength. In the case of Q-decking running perpendicular to the wall, no strength can be assumed at this boundary for out-of-plane wall loading.

6.3.1 Masonry L Side Boundary

Shear is assumed to be transferred across half the area of the block (excluding the central web), at each bed joint. The allowable shear stress is 34 psi as developed in Section 6.1.6. This formulation gives the following allowable boundary line loads:

6" block - 60 lb/in
8" block - 100 lb/in
12" block - 160 lb/in

6.3.2 Q-Deck (Parallel) Top Boundary

For this boundary type, shear is transferred through the grout extending up into the troughs of the metal decking. The basic shear stress is 40 psi (Section 6.1.5). For each wall thickness, the minimum number of complete troughs across the thickness of the wall is determined from the dimensions of the Q-decking. This gives a shear area per inch of wall, which is then converted to a line load via the allowable stress. The following allowable line loads result:

WALL WIDTH	ALLOWABLE LINE LOAD (lb/in.)
8"	120
12"	120
1'-6"	240
2'-0"	360
2'-2"	360
2'-6"	480
3'-0"	600
3'-6"	720

TABLE 6.2 : ALLOWABLE LOADS - Q-DECK (PARALLEL)

7 ALLOWABLE BOUNDARY LOADS

In Section 7.1, the ideal strengths from Section 6 are factored, to account for the actual number of anchors which can be relied upon (with 95% confidence) at any particular boundary. Exceptions to the "generic" strengths based on actual anchors found in a particular wall are presented in Section 7.2.

A summary of allowable boundary line loads is presented in Table 7.2. The particular boundary allowable is categorized by the wall thickness at the boundary, and the type and position of the boundary.

7.1 Reliable Strength (Actual Anchors)

The allowable strength values for boundaries relying on mechanical anchorages for shear transfer are calculated from the product of the appropriate ideal strength value from Section 6.2 and the reliable percentage of anchorages (95% confidence) for that particular boundary type from the statistical analyses (Section 5.2).

The appropriate percentage of the ideal strength for each boundary type is summarized in Table 7.1. The final boundary allowables incorporating these percentages, as well as allowables for boundaries relying on mechanical interlock for shear transfer (see Section 6.3) are presented in Table 7.2.

BOUNDARY TYPE	BOUNDARY LOCATION	RELIABLE PERCENTAGE OF IDEAL STRENGTH
Q-Deck	Top	0%
Steel	Top	0%
Concrete	Top	19.3%
Steel	Side	33.0%
Concrete	Side	20.0%

TABLE 7.1 : SUMMARY OF RELIABLE PERCENTAGE OF IDEAL STRENGTH

7.2 Exceptions to Generic Results

Nine walls (45.2, 184.7, 191.35, 191.55, 194.22, 194.23, 195.22, 196.6, and 196.7) have one boundary which is stronger than the generic criteria value. This is based on the actual anchors found along that boundary when the particular wall was tested.

For example, suppose a particular wall is 12 feet in length and was included in the test program. Suppose also that its top boundary was examined and 3 anchors were found when 60 inches of the boundary were chipped away. This data is then included as a sample point for the generic category of walls with this particular type of top boundary. Suppose also that upon completion of the statistical analysis of this data, the conclusion is reached that with 95% confidence, one anchor is present every 6 feet on this kind of boundary. Therefore on a 12 foot boundary, one would expect to find 2 anchors. The generic strength is based on this number. But for this particular wall, it is known for a fact that <u>at least three</u> anchorages are present on the top boundary. Therefore this particular wall has a strength greater than the generic criteria value.

It is on this basis that the nine walls mentioned above have one boundary strength which is an exception to the generic criteria. A summary of the particular boundary strengths for these walls is given in Table 7.3.

WALL THICKNESS	POSITION	BOUNDARY TYPES	ALLOWABLE SHEAR v_a (lb/in)	COMMENTS
8"	TOP	Q	120	See Note 3
		W	0	
		C	90	See Note 4
	SIDE	W	127	See Note 5
		C	93	See Note 6
		M	100	See Note 7
12"	TOP	Q	120	See Note 3
		W	0	
		C	90	See Note 4
	SIDE	W	127	See Note 5
		C	93	See Note 6
		M	160	See Note 7
1'-6"	TOP	Q	240	See Note 3
		W	0	
		C	180	See Note 4
	SIDE	W	255	See Note 5
		C	185	See Note 6
		M	*	See Note 8
2'-0" to 2'-2"	TOP	Q	360	See Note 3
		W	0	
		C	180	See Note 4
	SIDE	W	255	See Note 5
		C	185	See Note 6
		M	*	See Note 8
2'-6" to 3'-0"	TOP	Q	480	See Note 3
		Q	600	3'-0" Wall
		W	0	
		C	180	See Note 4
	SIDE	W	255	See Note 5
		C	185	See Note 6
		M	*	See Note 8
3'-6"	TOP	Q	720	See Note 3
		W	0	
		C	180	See Note 4
	SIDE	W	382	See Note 5
		C	278	See Note 6
		M	*	See Note 8

For Notes see following page.

TABLE 7.2 : ALLOWABLE BOUNDARY LINE LOADS

NOTES:

1. Tabulated allowable shear force v_a is in pounds per inch length of boundary.

2. Boundary types are as follows:
   - Q: Q Deck slab
   - W: Structural Steel
   - C: Concrete
   - M: Intersecting masonry wall.

3. Strength applies to walls parallel to the ribs of the Q-deck. For walls perpendicular to the Q-deck ribs, allowable shall be taken as zero.

4. Allowable shear applies only to walls with length greater than 6'-8". For shorter walls, the allowable shall be taken as zero.

5. Allowable shear applies only to walls with height greater than 4'-0". For shorter walls, the allowable shall be taken as zero.

6. Allowable shear applies only to walls with height greater than 6'-8". For shorter walls, the allowable shall be taken as zero.

7. Values for boundaries formed of masonry walls apply to walls at which interlocking of blocks from the two walls is apparent. For walls where no interlocking joint exists between the two walls, the allo[illegible]le boundary force shall be taken as zero.

8. For multi-wythe walls with interlocking masonry side boundaries, allowable shears shall be summed over all wythes for each of the intersecting walls. The lowest such calculated shear shall apply to all side boundaries at the intersection. Note 7 applies to each wythe. The allowable shear for a 6" thick wythe is 60 lb/in.

9. For boundaries at which the field inspection has revealed the presence of visible cracks, the allowable boundary stress shall be taken as zero.

10. <u>Exceptions</u> based on actual anchors found during field survey are listed in Table 7.3.

<u>TABLE 7.2 : NOTES</u>

WALL I.D.	BOUNDARY		ALLOWABLE STRESS (lb/in)		
	POSITION	TYPE	ACTUAL ANCHORS	CRITERIA VALUE	GOVERNING VALUE
45.2	Top	Concrete	187	90	187
184.7	Top	Concrete	134	90	134
191.35	Top	Steel	68	0	68
191.55	Top	Steel	37	0	37
194.23	Top	Steel	52	0	52
195.22	Top	Steel	37	0	37
196.6	Top	Steel	174	0	174
196.7	Top	Steel	142	0	142
194.22	South	Concrete	171	93	171

NOTES:

1. This table includes only values for which the actual anchors found in the wall tests provided a higher allowable boundary stress than the criteria values.

TABLE 7.3 - ALLOWABLE BOUNDARY LINE LOADS
(WALLS INCLUDED IN TEST PROGRAM)

8 CONCLUSIONS

This report has presented a set of allowable boundary line loads for the masonry walls at Pilgrim Nuclear Generating Station Unit 1. The allowables are soundly based on engineering principles and well documented material strengths.

Factored into these allowables are the results from the field test program performed at the plant. While the results of these tests indicated that the boundary anchorages were not in accordance with the design drawings, detailed statistical analysis of the available test data has indicated that certain percentages of the specified anchors can be relied upon (with 95% confidence). These percentages have been used to factor the ideal strength (assuming anchors as specified on the drawings) to arrive at the set of allowable boundary line loads for use in the masonry wall re-evaluation at Pilgrim Unit 1.

For certain walls, exceptions to the allowable boundary strengths have been presented. These exceptions are for those walls on which boundary strength tests were performed, and for which the actual number of anchorages found in the inspected region provided a greater strength than the statistically dependable (95% confidence) number of anchorages along the entire boundary.

9 REFERENCES

1 Park, R. and Paulay, T., "Reinforced Concrete Structures." John Wiley and Sons, 1975.

2 Brown, R. H. and Whitlock, A. R., "Strength of Anchor Bolts in Grouted Concrete Masonry." ASCE Journal of Structural Engineering, Vol. 109, No. 6, June 1983.

3 Colville, J., "Response to Comments on 'Appendix B of Dr. Colville's Report of 2/13/80 on Trojan Masonry Walls'." NRC Submittal, April 8, 1980.

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5 Benjamin, J.R., and Cornell, C.A., "Probability, Statistics, and Decision for Civil Engineers." McGraw-Hill, 1970.

APPENDIX A

This Appendix presents the details of the statistical analyses of the test data for each of five boundary types -- side boundaries to steel and concrete, and top boundaries to concrete, steel and metal Q-decking.

A.1 SIDE BOUNDARY TO STRUCTURAL STEEL

The sample space for side anchorages to structural steel is summarized in Table A.1. A total of 17 tests were performed on this boundary type. The histogram of this data is presented in Figure A.1. Each data point is plotted as a ratio of the specified anchors (based on a spacing of 16", or 0.0625 anchors per inch of wall).

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
SS-1	64.13	37.0	2	0.05405	18.5
SS-2	66.6	33.0	2	0.06061	16.5
SS-3	66.6	33.0	2	0.06061	16.5
SS-4	67.1	45.0	2	0.04444	22.5
SS-5	184.4	32.0	1	0.03125	32.0
SS-6	188.2	42.0	2	0.04762	21.0
SS-7	188.2	37.0	2	0.05405	18.5
SS-8	188.3	43.0	2	0.04651	21.5
SS-9	188.10	38.0	2	0.05263	19.0
SS-10	199.26	33.0	1	0.03030	33.0
SS-11	191.35	54.0	3	0.05556	18.0
SS-12	191.35	65.0	3	0.04615	21.7
SS-13	191.55	69.0	3	0.04348	23.0
SS-14	194.2	32.0	1	0.03125	32.0
SS-15	195.9	55.5	3	0.05405	18.5
SS-16	196.6	59.0	3	0.05085	19.7
SS-17	212.0	33.0	0	0.00000	∞

TABLE A.1 : SIDE ANCHORAGES TO STRUCTURAL STEEL

First of all the "anchorage per unit length" data is examined. The sample statistics for this data are as follows:

1. the mean number of anchorages per unit length is 0.0449, or 72% of the specified anchors.

2. the standard deviation of number of anchors per unit length is 0.0145, or 23% of the specified anchors. This implies a coefficient of variation of 32%.

3. the correlation coefficient between the exposed length and the number of anchors found is 0.82. Thus the data supports the

statement that as longer lengths are exposed, more anchorages are likely to be found.

The fact that there is a very strong correlation between length exposed and number of anchorages found indicates that little weight should be placed on any sample point with a relatively short exposed length and no anchorages.

Using the full set of data, the following analysis is performed. It is assumed that the underlying distribution is normal. Firstly, the distribution for the population mean is examined. With the sample statistics:

$$\overline{X} = 0.72, \; S_X = 0.23, \; N = 17$$

the standard deviation of the mean is:

$$S_{\overline{X}} = \frac{S_X}{\sqrt{N}} = 0.056$$

and hence (assuming that the distribution of the mean is normal) it can be stated with 95% confidence that the **population** mean is at least

$$0.72 - 1.64 \times 0.056 = 0.63$$

It is assumed that the population standard deviation is equal to the sample standard deviation. This is a reasonable (and probably conservative) assumption in light of the comments made in Section 3.2 regarding the contribution to scatter in the data due to anomalies in the test procedures. Thus the population has a mean of 0.63 and a standard deviation of 0.23. Assuming a normal distribution, these figures imply that over 99.5% of the population lies above the zero point, which is considered satisfactory. Using these population parameters, it can be concluded with 95% confidence that there are **at least** the following ratio of anchors present

$$0.63 - 1.64 \times 0.23 = 0.25$$

That is, 25% of the specified anchorages are present. This corresponds to one anchorage every 64 inches.

A similar analysis performed on those sample points with an exposed length greater than or equal to 40" indicates that 62% of the anchors could be relied upon with 95% confidence. 62% of the specified anchors amounts to 1 anchor approximately every 26 inches. However, there is one data point with a 33 inch length and no anchors. This data point suggests that examining only the "40 inch" data would lead to erroneous conclusions, and the 95% strength is significantly less than 62% of the specified anchors.

A detailed study of the effect of various assumptions regarding the "zero" data point (test SS-17) is made. The one data point in question does not effect the sample mean a great deal, but it does have a significant affect on the sample (and hence by assumption, the population) standard deviation.

The following assump[illegible]ns are made regarding the zero data point. Firstly, it is ignored completely; [illegible]econdly, it is replaced with a test finding one anchorage

In an exposed length of 64" (the conclusion from the first analysis of the data including the zero point), and finally it is replaced with a data point finding one anchorage in an exposed length of 80" (a more conservative assumption). It is reasonable to replace the "zero" point with another data point having a greater exposed length and one anchorage due to the high correlation between number of anchorages found and exposed length. The effect on the sample mean and standard deviation is as follows:

Database	Sample Mean	Standard Deviation
All data	0.72	0.23
Ignore test SS-17	0.76	0.15
SS-17 (1 anch. in 64")	0.73	0.19
SS-17 (1 anch. in 80")	0.73	0.20

An analysis similar to those previosly described was performed on the third data base ("zero" test replaced with 1 anchorage in 64") and the result is that with 95% confidence, 33% of the specified anchors can be relied upon. This can alternatively be interpreted as finding one anchorage every 48". Indeed out of the eight tests with exposed lengths greater than 40", there was at least one anchorage found in each case, which is consistent with the conclusion from this analysis.

As a final substantiation for replacing the "zero" point, an analysis is performed assuming there is a uniform anchorage spacing of 48" (the last conclusion). There are six tests with exposed lengths in the range of 30" to 36", one of which found zero anchors. The probability of one test out of six (all assumed to have 33" lengths) where no anchors are found is calculated. The binomial distribution is assumed, with the occurrence probability for a single event calculated from the geometry of 48" spacing and 33" exposed length. This gives a probability of 0.29 of not finding any anchors in 33". The binomial distribution gives a probability of finding one test out of six (33" length) with no anchors as 0.31. There is a probability of 0.12 of finding zero tests out of six with no anchors. Thus the observed occurrence of one test out of six with no anchors in a length of approximately 33" would be a relatively common occurrence for an anchorage spacing of 48" (probability of 31%).

On this basis, the hypothesis that there is one anchor every 48" (33% of the specified anchors) is accepted with 95% confidence.

These analyses have been based on the assumption that the underlying distribution is a normal one. A Kolmogorov-Smirnov goodness of fit test has been performed on the complete set of data points (17 points). At both the 5% and 10% significance levels it is concluded that the hypothesis of a normal distribution should not be rejected.

As a final check on the above analyses, the "inverse" set of data is analysed in a similar manner to that described above. The zero point is again replaced with a test finding one anchorage in an exposed length of 64". and it is concluded with 95% confidence that there is one anchor every 47". This conclusion is extremely consistent with that from the anchorage per unit length data, and therefore it can be concluded with 95% confidence, that 33% of the specified anchors are present on the side boundaries to structural steel.

A.2 SIDE BOUNDARY TO CONCRETE

The sample space for side anchorages to concrete is summarized in Table A.2. A total of 20 tests were performed on this boundary type. The histogram of this data is presented in Figure A.2. Each data point is plotted as a ratio of the ideal anchors which are based on a spacing of 16". or 0.0625 anchors per inch of wall.

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
SC-1	45.2	33.0	1	0.03030	33.0
SC-2	62.2	38.0	2	0.05263	19.0
SC-3	63.5	30.0	0	0.00000	∞
SC-4	64.4	36.0	0	0.00000	∞
SC-5	65.21	33.0	1	0.03030	33.0
SC-6	66.0	33.0	1	0.03030	33.0
SC-7	66.2	48.0	2	0.04167	24.0
SC-8	66.5	40.0	2	0.05000	20.0
SC-9	67.1	43.0	2	0.04651	21.5
SC-10	68.10	52.0	2	0.03846	26.0
SC-11	68.10	36.0	2	0.05556	18.0
SC-12	184.9	32.0	2	0.06250	16.0
SC-13	188.9	36.0	1	0.02778	36.0
SC-14	194.2	32.0	0	0.00000	∞
SC-15	194.22	54.0	3	0.05556	18.0
SC-16	194.25	32.0	1	0.03125	32.0
SC-17	195.14	48.0	2	0.04167	24.0
SC-18	198.0	31.0	1	0.03226	31.0
SC-19	198.3	36.0	2	0.05556	18.0
SC-20	209.0	38.0	1	0.02632	38.0

TABLE A.2 : SIDE ANCHORAGES TO CONCRETE

First of all the "anchorage per unit length" data is examined. The sample statistics for this data are as follows:

1. the mean number of anchorages per unit length is 0.0354, or 57% of the specified anchors.

2. the standard deviation of number of anchors per unit length is 0.0183, or 29% of the specified anchors. This implies a coefficient of variation of 52%.

3. the correlation coefficient between the exposed length and the number of anchors found is 0.70. Thus the data supports the statement that as longer lengths are exposed, more anchorages are likely to be found.

The fact that there is a strong correlation between length exposed and number of anchorages found indicates that relatively little weight should be placed on any sample point with a relatively short exposed length and no anchorages.

Using the full set of data, an analysis similar to that described in Section A.1 was performed. However, when a normal distribution was fitted to the data, approximately 5% of the distribution was below zero. This is a physically unrealizable situation, and this analysis was carried no further.

A similar analysis performed on those sample points with an exposed length greater than or equal to 40" indicates that 52% of the anchors could be relied upon with 95% confidence. 52% of the specified anchors amounts to 1 anchor approximately every 31 inches. However, there are three data points with lengths of 30", 32" and 36", all with no anchors. These data points suggest that examining only the "40 inch" data leads to erroneous conclusions, and the 95% strength is significantly less than 52% of the specified anchors.

A detailed study of the effect of various assumptions regarding the "zero" data points (tests SC-3,4 and 14) is now made. The three data points in question do not effect the sample mean a great deal, but do have a significant effect on the sample (and hence by assumption, the population) standard deviation. The following assumptions are made regarding the zero data points. Firstly, they are ignored completely. Secondly, they are each replaced with a test finding one anchorage in a length 32" greater than the actual test length (one anchor in 32" was the conclusion from the 40" data), and finally they are replaced with data points finding one anchorage in an exposed length of 80" (a more conservative assumption). It is reasonable to replace the "zero" points with an equal number of data points having a greater exposed length and one anchorage, due to the high correlation between number of anchorages found and exposed length. The effect on the sample mean and standard deviation is as follows:

Database	Sample Mean	Standard Deviation
All data	0.57	0.29
Ignore tests SC-3,4 and 14	0.67	0.18
SC-3,4 and 14 (add 1 anch. in 32")	0.60	0.23
SC-3,4 and 14 (1 anch. in 80")	0.60	0.24

An analysis similar to those previosly described was performed on the third data base ("zero" tests replaced with an equal number of tests finding 1 anchorage in a length 32" greater than the actual test length) and the result is that with 95% confidence, 14% of the specified anchors can be relied upon. This can alternatively be interpreted as finding one anchorage every 112".

Again, as a substantiation for replacing the "zero" points, an analysis is performed assuming there is a uniform anchorage spacing of 112" (the last conclusion). There are eleven tests with exposed lengths in the range of 30" to 36", three of which found zero anchors. The probability of three tests out of eleven (all assumed to have 36" lengths) where no anchors are found is calculated. The binomial distribution is assumed, with the occurrence probability for a single event calculated from the geometry of 112" spacing and 36" exposed length. This gives a probability of not finding any anchors in 36" of 0.67. The binomial distribution gives a probability of finding three tests out of eleven (36" length) with no anchors as less than 1%. This is a very rare event, it being most likely to find seven or eight tests out of eleven with no anchorages in 36" given a uniform spacing of 112" for the anchorages, and random positioning of the 36" length. This analysis indicates that the 112" spacing may be quite conservative.

In an attempt to clarify the situation, the "inverse" set of data is analysed in a similar manner to that described above. The zero points are again replaced as described above and it is concluded with 95% confidence that there is one anchor every 75". In light of this conclusion, the probability of occurrence of the zero data points is re-examined. It is concluded that there is a 7% chance of finding three tests out of eleven (of 36" length) with no anchorages. Thus the zero points are a much more likely occurrence under the 75" spacing than under the 112" spacing. This 75" spacing is increased to 80" (an even multiple of the 8" block height).

These analyses have been based on the assumption that the underlying distribution is a normal one. A Kolmogorov-Smirnov goodness of fit test has been performed on the complete set of data points (20 points). At both the 5% and 10% significance levels it is concluded that the hypothesis of a normal distribution should not be rejected.

On this basis, the hypothesis that there is one anchor every 80" (20% of the specified anchors), is accepted as the 95% strength.

A.3 TOP BOUNDARY TO CONCRETE

The sample space for top anchorages to concrete is summarized in Table A.3. A total of 10 tests were performed on this boundary type. The histogram of this data is presented in Figure A.3. Each data p nt is plotted as a ratio of the specified anchors (based on a spacing of 16", or 0.0625 anchors per inch of wall).

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
TC-1	45.2	66.0	2	0.03030	33.0
TC-2	62.2	48.0	2	0.04167	24.0
TC-3	64.12	38.0	2	0.05263	19.0
TC-4	66.6	30.0	2	0.06667	15.0
TC-5	66.11	48.0	1	0.02083	48.0
TC-6	184.7	48.0	3	0.06250	16.0
TC-7	184.8	49.0	3	0.06122	16.3
TC-8	188.10	49.0	3	0.06122	16.3
TC-9	209.0	44.0	1	0.02273	44.0
TC-10	212.1	48.0	1	0.02083	48.0

TABLE A.3 : TOP ANCHORAGES TO CONCRETE

First of all the "anchorage per unit length" data is examined. The sample statistics for this data are as follows:

1. the mean number of anchorages per unit length is 0.0441, or 71% of the specified anchors.

2. the standard deviation of number of anchors per unit length is 0.0180, or 29% of the specified anchors. This implies a coefficient of variation of 41%.

From a visual examination of the histogram of this data (Figure A.3), there is no reason to fit any distribution other than a uniform distribution. The question arises as to what parameters should be given to the uniform distribution, i.e., what are its lower and upper limits?

Using the sample statistics above to fit a uniform distribution, the lower bound is calculated as 0.21, and the upper bound as 1.21. This gives some guidelines for establishing the lower bound at around 0.2. It also gives weight to the logical choice of 1.0 for the upper bound on the distribution. However, the 95% strength is very sensitive to these assumed bounds for the distribution, and these were recalculated based on the 7 tests with lengths ranging from 44" to 49". (This range is very close to the test requirement of 48".) With this data base, the sample mean is 0.0416 anchorages per unit length (67% of those specified), with a sample standard deviation of 0.0186 (30%). When the above analysis is repeated using these numbers, the lower bound for the uniform distribution is 0.15 and the upper bound is 1.19.

In view of these two analyses, it is reasonable to assign the lower end of the uniform distribution at 0.15 and the upper bound at 1.0. This assumed distribution is plotted in Figure A.3. 95% of this distribution lies above

$$(1.0 - 0.15) \times 0.05 + 0.15 = 0.193$$

or 19.3% of the specified anchors.

Thus 19.3% of the specified anchors at the top boundary to concrete can be relied upon with 95% confidence. This is equivalent to 1 anchorage every 80 inches.

A.4 TOP BOUNDARY TO STRUCTURAL STEEL

The sample space for the top anchorages to structural steel is summarized in Table A.4. A total of 23 tests were performed on this boundary type. The histogram of this data is presented in Figure A.4. Each data point is plotted as a ratio of the specified anchors (based on a spacing of 16", or 0.0625 anchors per inch of boundary).

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH
TS-1	64.4	38.0	2	0.05263
TS-2	64.4	36.0	2	0.05556
TS-3	65.16	43.0	1	0.02326
TS-4	65.21	60.0	1	C.01667
TS-5	66.0	42.0	1	0.02381
TS-6	66.6	13.0	1	0.07692
TS-7	66.6	51.0	2	0.03922
TS-8	66.18	50.0	0	0.00000
TS-9	67.1	58.0	1	0.01724
TS-10	68.10	48.0	0	0.00000
TS-11	184.2	38.0	0	0.00000
TS-12	184.4	48.0	1	0.02083
TS-13	188.2	48.0	0	0.00000
TS-14	188.9	47.0	0	0.00000
TS-15	191.26	73.0	1	0.01370
TS-16	191.35	81.0	1	0.01235
TS-17	191.55	48.0	1	0.02083
TS-18	194.23	49.0	1	0.02041
TS-19	195.14	48.0	2	0.04167
TS-20	195.22	68.0	1	0.01471
TS-21	196.6	57.5	2	0.03478
TS-22	196.7	10.0	0	0.00000
TS-23	196.7	60.5	2	0.03306

TABLE A.4 : TOP ANCHORAGES TO STRUCTURAL STEEL

It should be noted that more than 25% of the data (6 of 23 tests) is lumped at the zero point. Four of these six data points come from tests with considerable length (47 inches or more). It is thus apparent that substitution of data points for this zero data is a rather suspect course of action in this case. Also, with many data points having large exposed lengths and zero anchors, the correlation coefficient between exposed length and number of anchors found will be low. Again, this warns against substution of data.

With these considerations in mind, the only possible value for the 95% strength for this boundary type is zero.

A.5 TOP BOUNDARY TO Q-DECK

The sample space for the top anchorages to metal Q-deck is summarized in Table A.5. A total of 18 tests were performed on this boundary type. The histogram of this data is presented in Figure A.5. Each data point is plotted as a ratio of the specified anchors (based on a spacing of 16", or 0.0625 anchors per inch of boundary).

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH
TQ-1	63.5	53.0	0	0.00000
TQ-2	64.6	51.0	1	0.01961
TQ-3	66.2	72.5	0	0.00000
TQ-4	66.5	28.0	0	0.00000
TQ-5	66.5	16.0	0	0.00000
TQ-6	66.7	48.0	2	0.04167
TQ-7	66.12	47.0	0	0.00000
TQ-8	111.7	40.0	2	0.05000
TQ-9	188.3	56.0	3	0.05357
TQ-10	191.49	64.0	0	0.00000
TQ-11	194.21	59.0	0	0.00000
TQ-12	194.21	47.0	0	0.00000
TQ-13	194.22	49.0	0	0.00000
TQ-14	194.25	32.0	0	0.00000
TQ-15	195.18	41.0	0	0.00000
TQ-16	195.23	6.0	0	0.00000
TQ-17	195.23	35.0	0	0.00000
TQ-18	195.23	12.0	0	0.00000

TABLE A.5 : TOP ANCHORAGES TO Q-DECK

It is readily apparent from a visual inspection of the histogram that no anchorages can be relied upon with any confidence for this boundary type. This is due to the extremely large (78%) proportion of the sample points for which no anchorages were found.

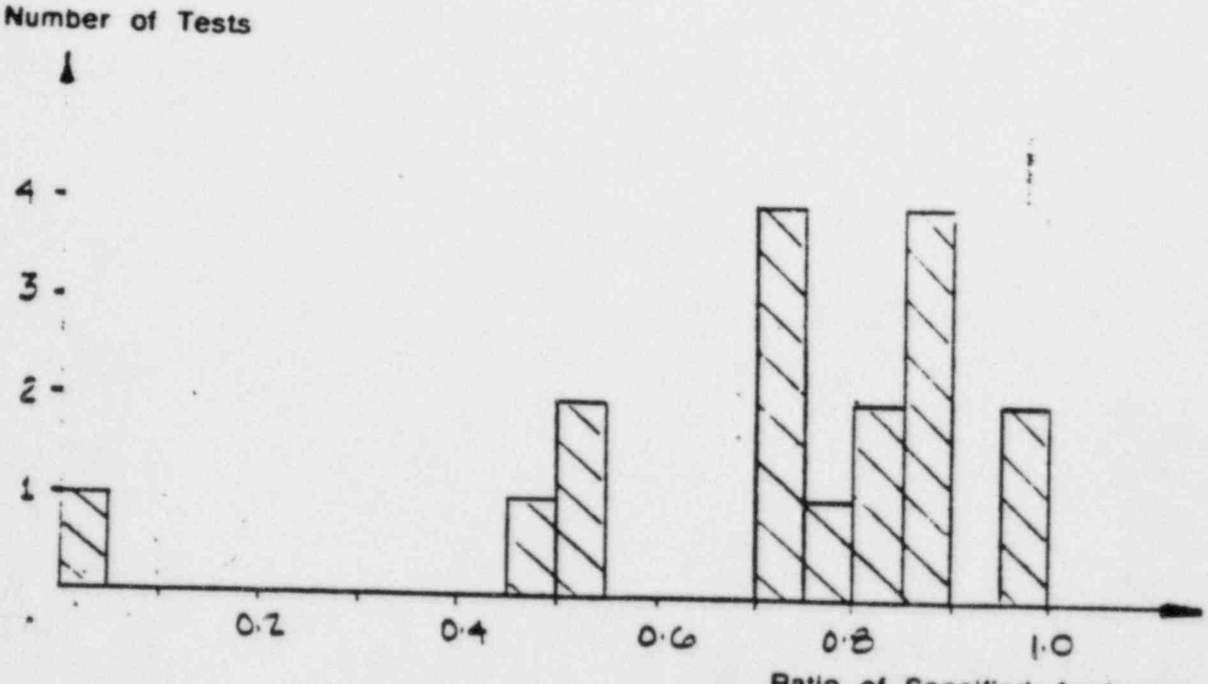


Figure A.1 : Side Anchorages to Steel Boundary

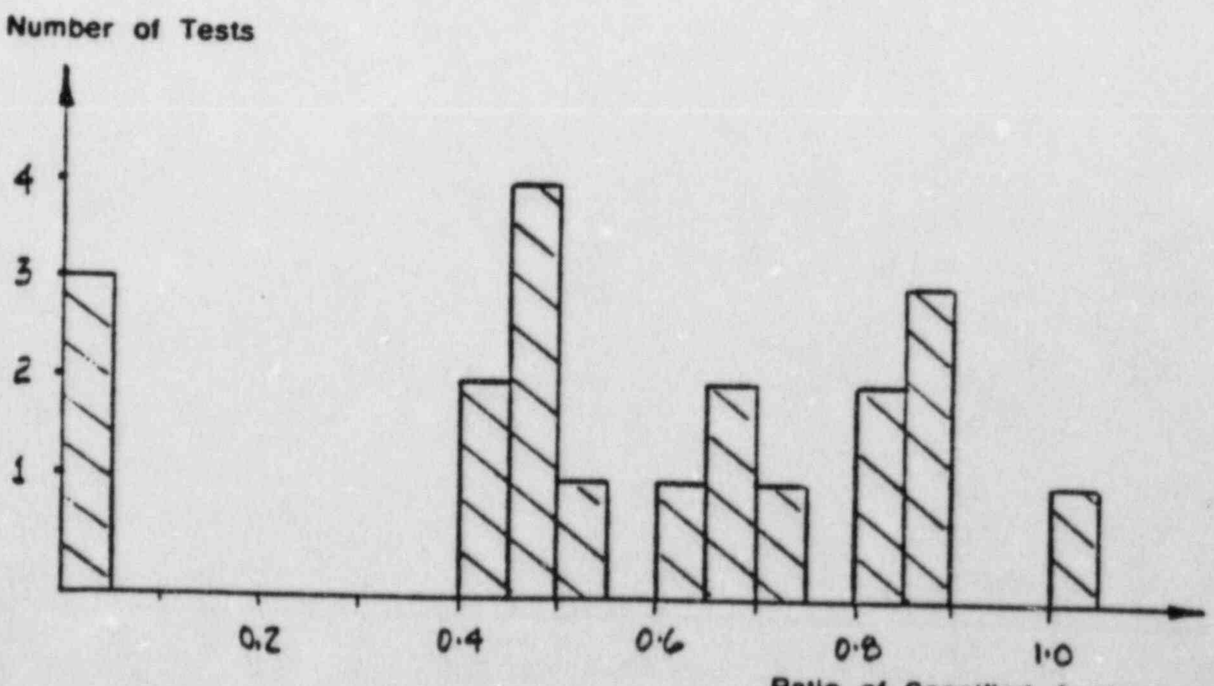


Figure A.2 : Side Anchorages to Concrete Boundary

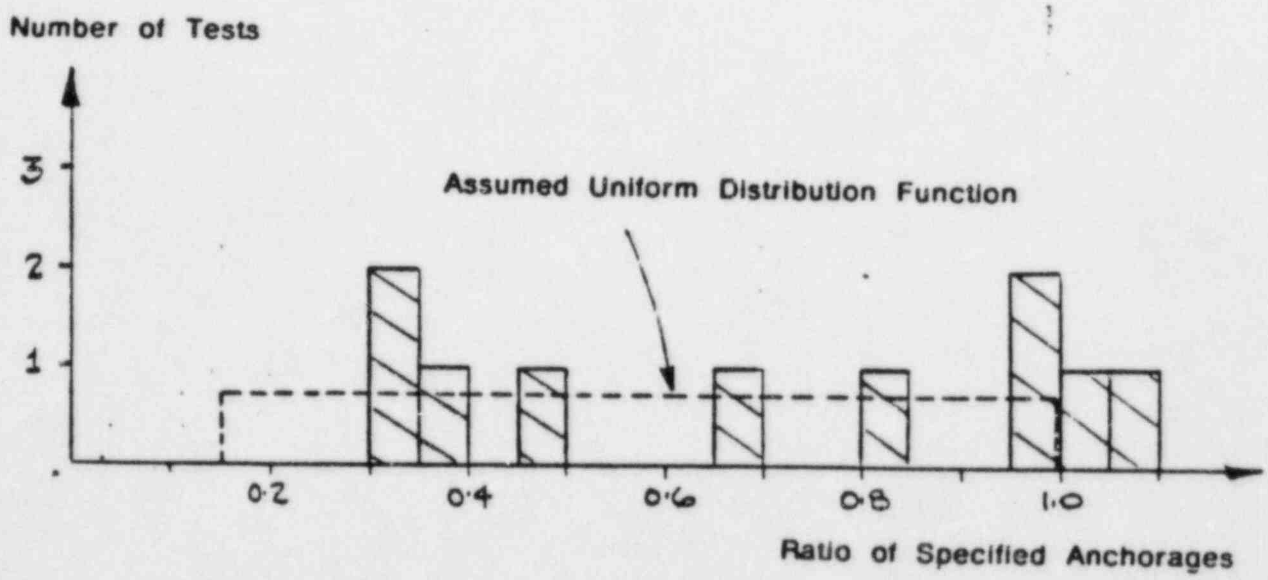


Figure A.3 : Top Anchorages to Concrete Boundary

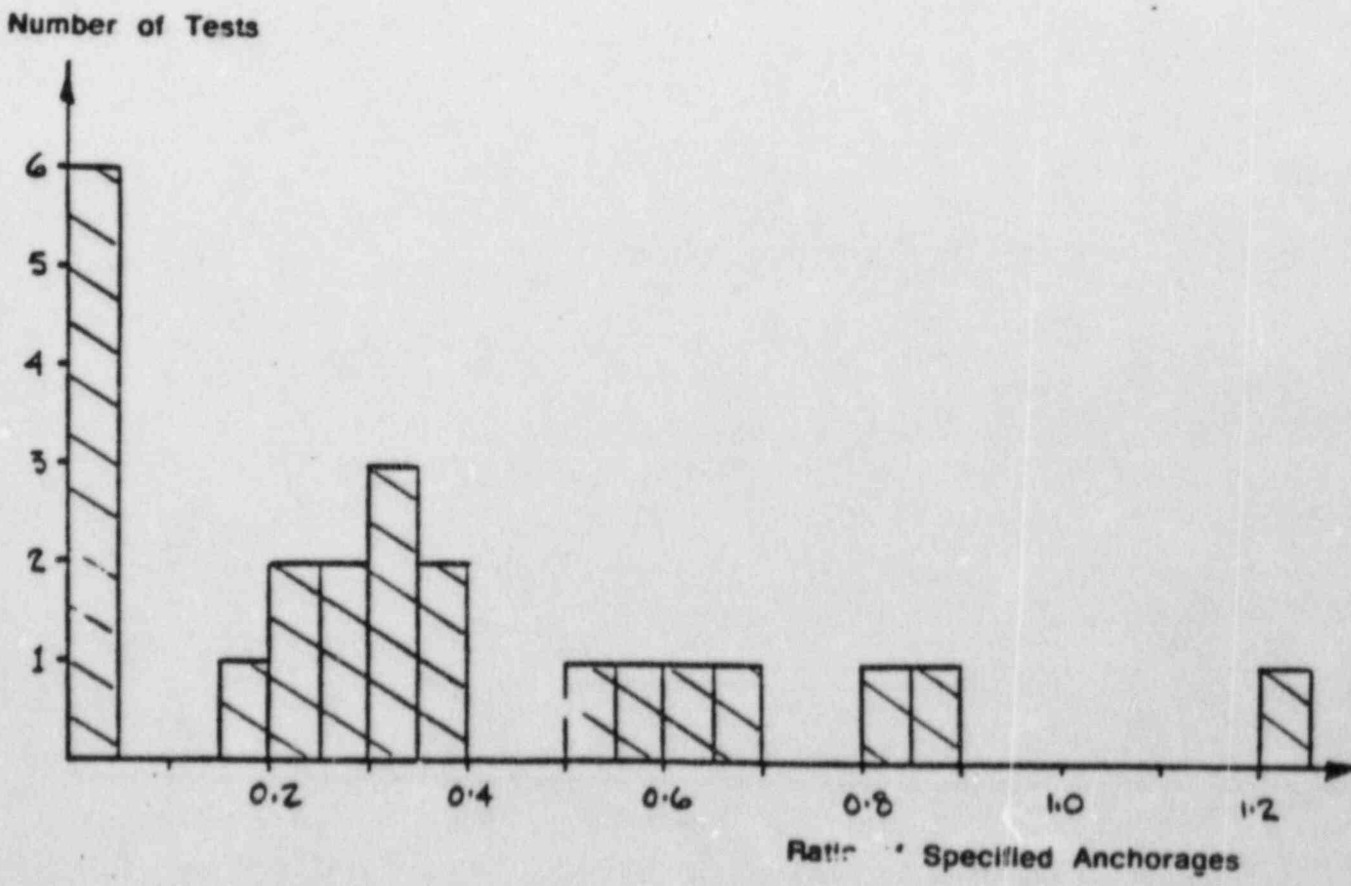


Figure A.4 : Top Anchorages to Steel Boundary

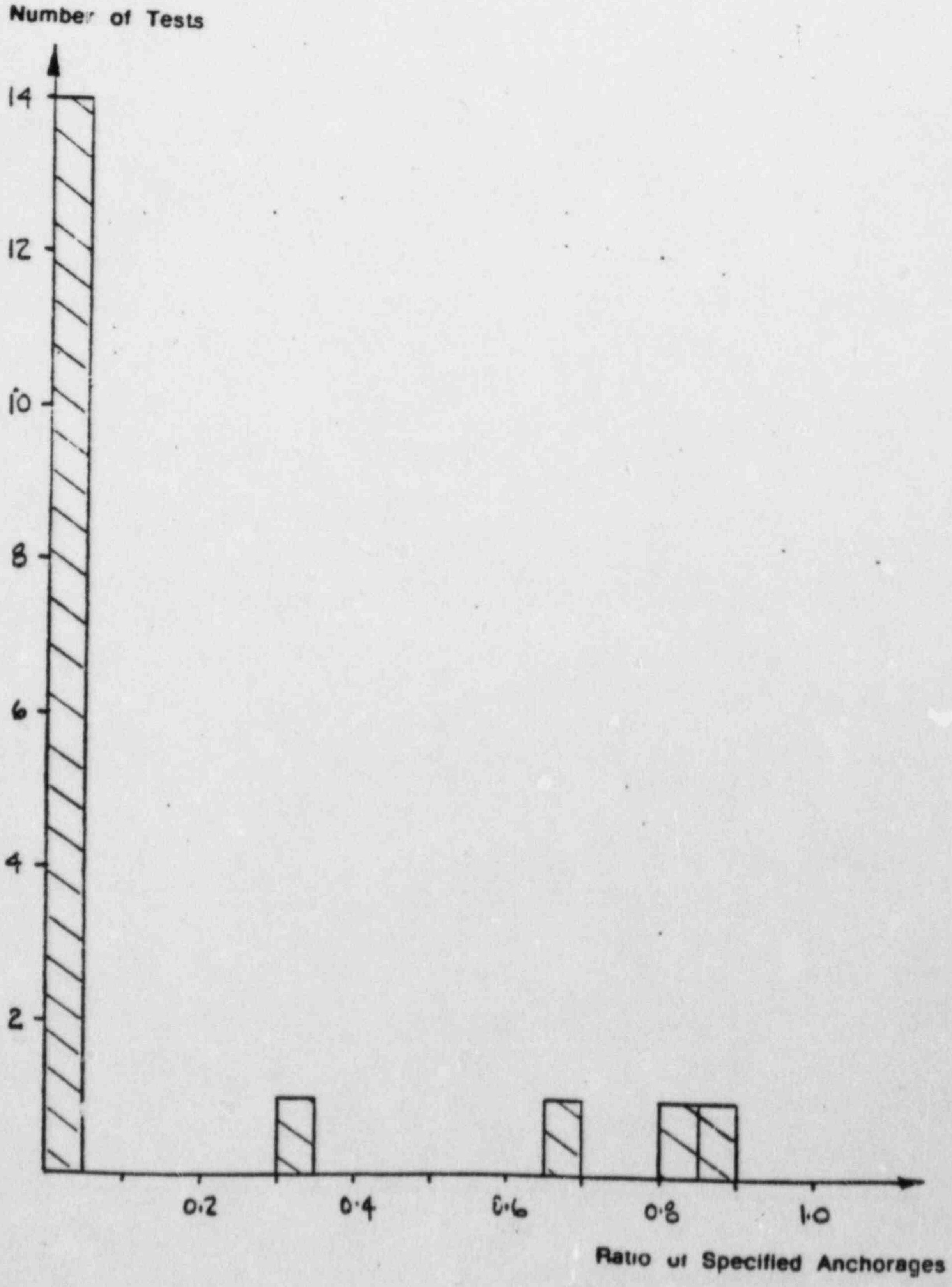


Figure A.5 : Top Anchorages to Metal Q-Deck

ATTACHMENT 7 (Response to items 3 and 4 of Attachment 1)

PILGRIM NUCLEAR GENERATING STATION

UNIT 1

STATISTICAL ANALYSIS OF BOUNDARY STRENGTHS FOR MASONRY WALLS FROM FIELD TEST DATA

- INTRODUCTION
- RESULTS
- STATISTICS REVIEW
- DETAILS OF ANALYSES

INTRODUCTION

- BOUNDARIES OF MASONRY WALLS AT PILGRIM WERE "CHIPPED AWAY" OVER LIMITED LENGTHS TO CHECK ANCHORAGE CONFORMANCE WITH CONSTRUCTION DRAWINGS.

- ANCHORAGES DID NOT CONFORM TO SPECIFICATIONS; HOWEVER, THERE WERE A SUBSTANTIAL NUMBER OF ANCHORS PRESENT.

- STATISTICAL ANALYSES OF THE FIELD TEST DATA WERE PERFORMED TO DETERMINE WHAT PERCENTAGE OF THE ANCHORS SPECIFIED COULD BE RELIED UPON WITH 95% CONFIDENCE.

RESULTS

- DRAWINGS SPECIFIED ONE ANCHOR EVERY 16" FOR ALL BOUNDARIES.

- THE FOLLOWING ARE CONCLUSIONS FROM THE STATISTICAL ANALYSES OF THE TEST DATA:

BOUNDARY TYPE	SPACING
SIDE - STEEL	48"
SIDE - CONCRETE	80"
TOP - CONCRETE	80"
TOP - STEEL	NO ANCHORS
TOP - Q-DECK	NO ANCHORS

- THESE RESULTS ARE AT THE 95% CONFIDENCE LEVEL.

BASIC STATISTICS

- SAMPLE MEAN $$\bar{X} = \frac{1}{N} \sum_{i=1}^{N} X_i$$

- SAMPLE STANDARD DEVIATION
$$S = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (X_i - \bar{X})^2}$$

- POPULATION MEAN IS A RANDOM VARIABLE WITH:

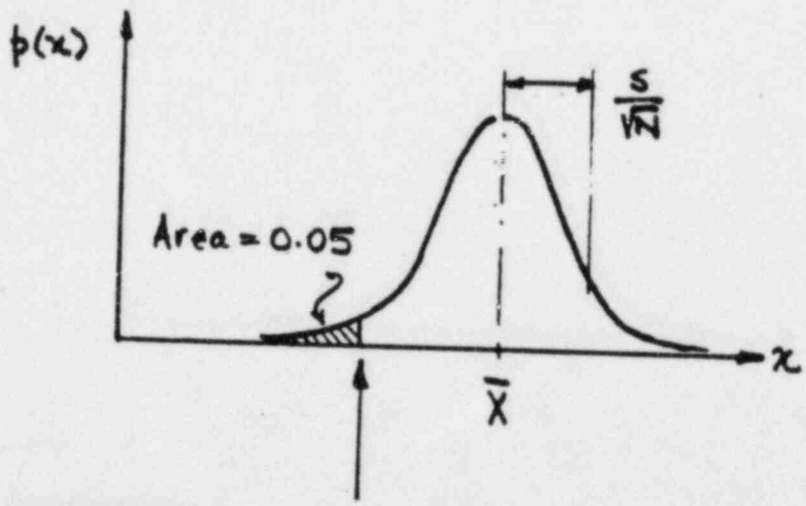
  MEAN $= \bar{X}$

  STANDARD DEVIATION $= \frac{S}{\sqrt{N}}$

- POPULATION STANDARD DEVIATION

  TAKE $\sigma = S$

POPULATION MEAN



WITH 95% CONFIDENCE,

POPULATION MEAN = $\mu \geq \bar{X} - 1.64\frac{s}{\sqrt{N}}$

WE USE THIS VALUE FOR THE POPULATION MEAN.

POPULATION STANDARD DEVIATION

WE TAKE THE POPULATION STANDARD DEVIATION (σ) TO BE EQUAL TO THE SAMPLE STANDARD DEVIATION (S).

$$\sigma = s$$

THIS IS CONSERVATIVE BECAUSE SOME SCATTER IN THE SAMPLE IS A RESULT OF THE FIELD TEST PROCEDURES.

- NOT ALL TESTS WERE PERFORMED OVER THE SAME LENGTH

- RANGE IS FROM 13" TO 81"

POPULATION STATISTICS AND DISTRIBUTION

THE POPULATION IS CHARACTERIZED BY:

MEAN $= \mu$

STANDARD DEVIATION $= \sigma$

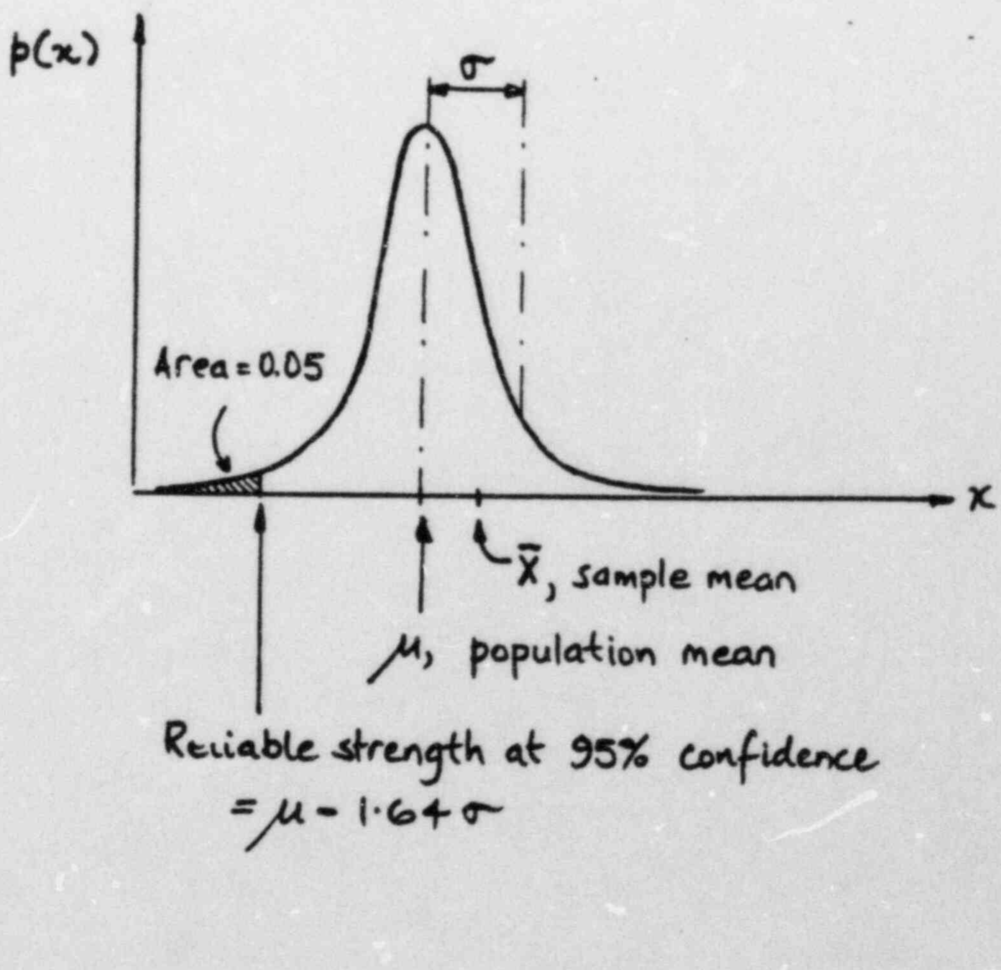
THE DISTRIBUTION IS ASSUMED NORMAL AND A "GOODNESS OF FIT" TEST IS PERFORMED TO CONFIRM THIS ASSUMPTION.

IN ONE CASE, THE POPULATION IS ASSUMED TO HAVE A UNIFORM DISTRIBUTION.

RELIABLE ANCHORAGES

FOR EACH BOUNDARY TYPE, USING THE POPULATION STATISTICS AND THE DISTRIBUTION, THAT STRENGTH ABOVE WHICH 95% OF THE POPULATION LIES IS CALCULATED.

FOR A NORMAL DISTRIBUTION, THIS IS:



ZERO STRENGTHS

IN SOME CASES, A LARGE PROPORTION OF THE TEST DATA GIVES ZERO ANCHORS IN SIGNIFICANT EXPOSED LENGTHS.

IN THESE CASES, NO STRENGTH FROM ANCHORS IS RELIED UPON.

SIDE BOUNDARY TO STEEL

CONCLUSION: 1 ANCHOR EVERY 48" (33% OF SPECIFIED)

SAMPLE: 17 TESTS

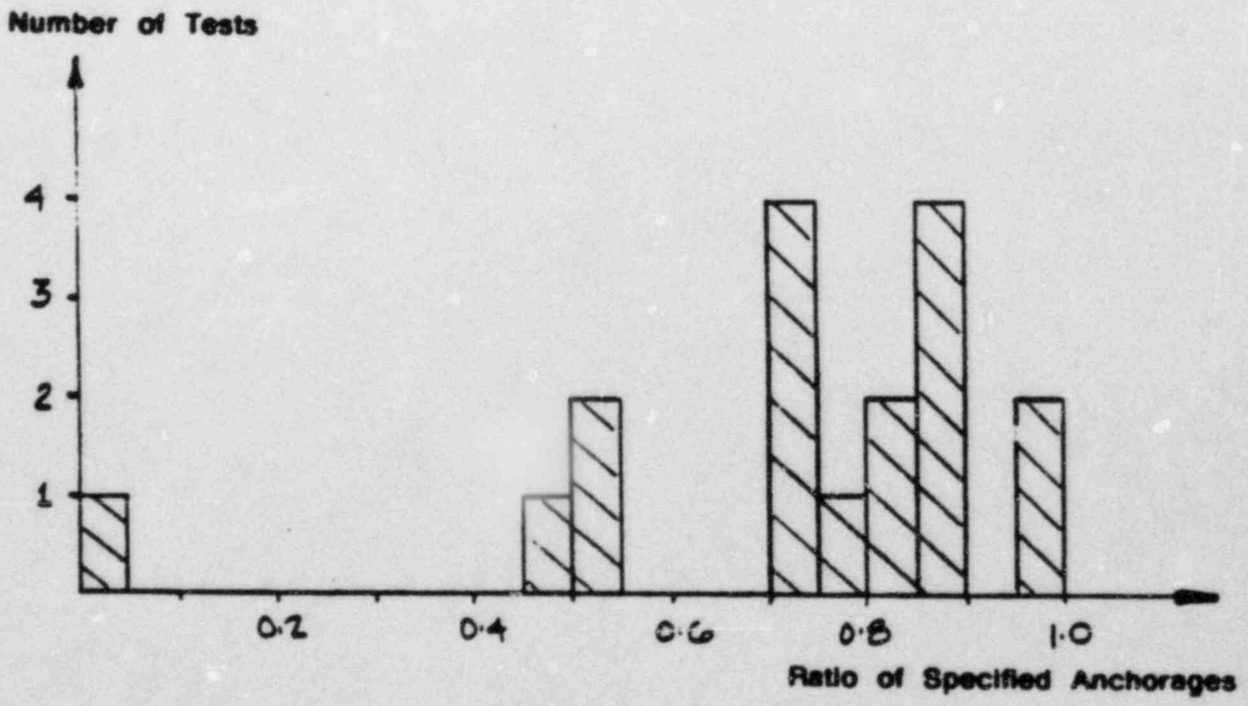
$X = 0.72$, $s = 0.23$, $\rho_{AL} = 0.82$

1 ZERO DATA POINT (33")

LITTLE WEIGHT SHOULD BE PLACED ON "SHORTER" DATA.

SIDE ANCHORAGES TO STRUCTURAL STEEL

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
SS-1	64.13	37.0	2	0.05405	18.5
SS-2	66.6	33.0	2	0.06061	16.5
SS-3	66.6	33.0	2	0.06061	16.5
SS-4	67.1	45.0	2	0.04444	22.5
SS-5	184.4	32.0	1	0.03125	32.0
SS-6	188.2	42.0	2	0.04762	21.0
SS-7	188.2	37.0	2	0.05405	18.5
SS-8	188.3	43.0	2	0.04651	21.5
SS-9	188.10	38.0	2	0.05263	19.0
SS-10	191.26	33.0	1	0.03030	33.0
SS-11	191.35	54.0	3	0.05556	18.0
SS-12	191.35	65.0	3	0.04615	21.7
SS-13	191.55	69.0	3	0.04348	23.0
SS-14	194.20	32.0	1	0.03125	32.0
SS-15	195.9	55.5	3	0.05405	18.5
SS-16	196.6	59.0	3	0.05085	19.7
SS-17	212.0	33.0	0	0.00000	∞



SIDE BOUNDARY TO STEEL

USING DATA GREATER THAN 40" (8 TESTS)

⇒ 1 ANCHOR EVERY 26" (62% OF SPECIFIED)

. BUT THIS CONCLUSION IS POSSIBLY VIOLATED BY 1 DATA POINT WITH NO ANCHORS IN 33".

⇒ STRENGTH IS LESS THAN 62%

SIDE BOUNDARY TO STEEL

IF WE REPLACE THE ZERO POINT WITH 1 ANCHOR IN 64" (THE CONCLUSION FROM ALL THE DATA):

⇒ 1 ANCHOR EVERY 48"

- THIS IS SUBSTANTIATED BY 8 TESTS WITH LENGTHS > 40" WHERE AT LEAST ONE ANCHOR WAS FOUND IN EACH CASE.

- ALSO THERE ARE 6 TESTS IN THE RANGE 30" TO 36"". ONE OF WHICH GIVES NO ANCHORS.
  - BASED ON THE 48" SPACING. THIS EVENT HAS A PROBABILITY OF 31%
  - AND TWO OUT OF SIX WITH NO ANCHORS WOULD BE EXPECTED 32% OF THE TIME.

THUS. THE ZERO POINT IS QUITE IN KEEPING WITH THE 48" SPACING.

ON THIS BASIS, WE ACCEPT 1 ANCHOR EVERY 48"

SIDE BOUNDARY TO CONCRETE

CONCLUSION: 1 ANCHOR EVERY 80" (20% OF SPECIFIED)

SAMPLE: 20 TESTS

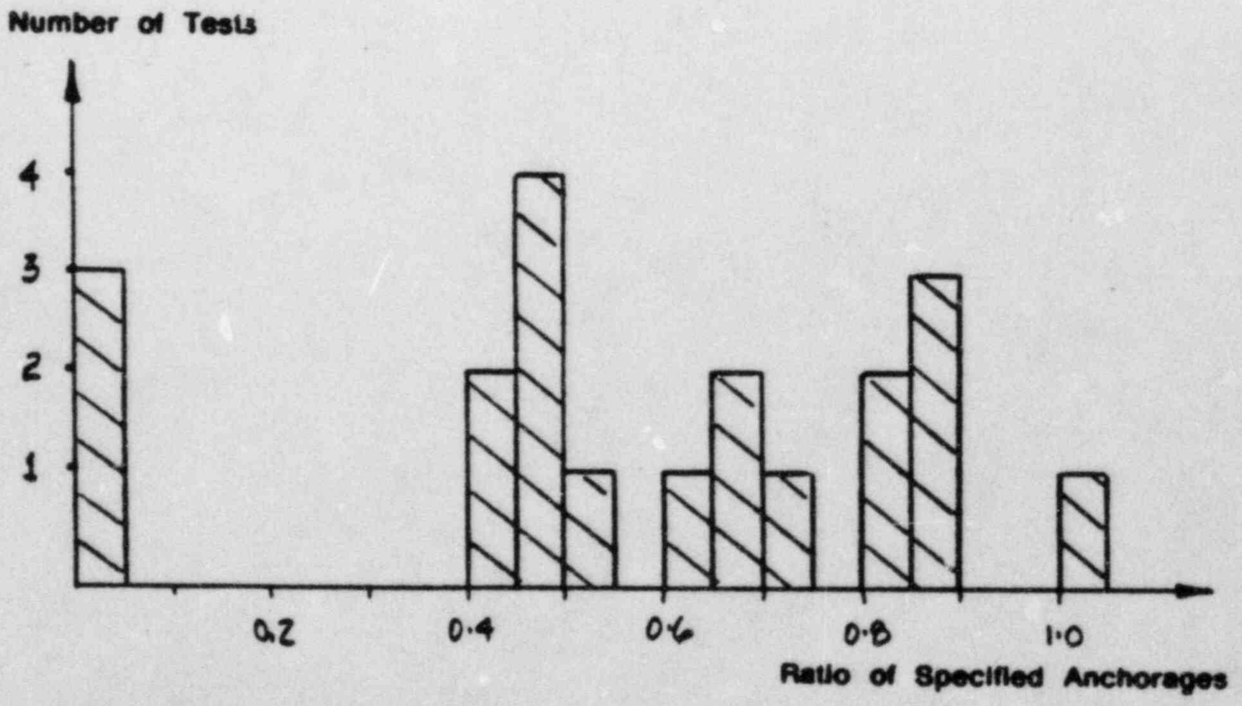
$X = 0.57$ $s = 0.29$ $\rho_{AL} = 0.7$

3 ZERO DATA POINTS (30", 32", 36")

LITTLE WEIGHT SHOULD BE PLACED ON "SHORTER" DATA

SIDE ANCHORAGES TO CONCRETE

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
SC-1	45.2	33.0	1	0.03030	33.0
SC-2	62.2	38.0	2	0.05263	19.0
SC-3	63.5	30.0	0	0.00000	∞
SC-4	64.4	36.0	0	0.00000	∞
SC-5	65.21	33.0	1	0.03030	33.0
SC-6	66.0	33.0	1	0.03030	33.0
SC-7	66.2	48.0	2	0.04167	24.0
SC-8	66.5	40.0	2	0.05000	20.0
SC-9	67.1	43.0	2	0.04651	21.5
SC-10	68.10	52.0	2	0.03846	26.0
SC-11	68.10	36.0	2	0.05556	18.0
SC-12	184.9	32.0	2	0.06250	16.0
SC-13	188.9	36.0	1	0.02778	36.0
SC-14	194.20	32.0	0	0.00000	∞
SC-15	194.22	54.0	3	0.05556	18.0
SC-16	194.25	32.0	1	0.03125	32.0
SC-17	195.14	48.0	2	0.04167	24.0
SC-18	198.0	31.0	1	0.03226	31.0
SC-19	198.3	36.0	2	0.05556	18.0
SC-20	209.0	38.0	1	0.02632	38.0



SIDE BOUNDARY TO CONCRETE

USING DATA GREATER THAN 40" (6 TESTS)

⇒ 1 ANCHOR EVERY 32" (50% OF SPECIFIED)

HOWEVER, WE HAVE 30", 32" AND 36" LENGTHS WITH NO ANCHORS GIVING POSSIBLE VIOLATION OF THE ABOVE ASSUMPTION.

⇒ STRENGTH IS LESS THAN 50%

SIDE BOUNDARY TO CONCRETE

IF WE REPLACE THE ZERO POINTS WITH AN EQUAL NUMBER OF DATA POINTS ADDING 32" AND 1 ANCHOR (THE CONCLUSION FROM THE 40" DATA) TO THE ACTUAL RECORDED DATA, AND ANALYZE LENGTH/ANCHORAGE DATA:

$\Longrightarrow$ 1 ANCHOR EVERY 75"

- THERE ARE 11 TESTS IN THE RANGE OF 30" TO 36", THREE OF WHICH HAVE NO ANCHORS.
  - BASED ON A 75" SPACING, THIS EVENT HAS A PROBABILITY OF 7%
  - OTHER PROBABILITIES ARE:

4 ZERO TESTS:	15%
5 ZERO TESTS:	22%
6 ZERO TESTS:	23%
7 ZERO TESTS:	17%

THUS, THE THREE ZERO POINTS ARE AT THE LOW END OF WHAT WOULD BE EXPECTED FROM A 75" SPACING.

ON THIS BASIS, WE ACCEPT 1 ANCHOR EVERY 80"

(AN EVEN MULTIPLE OF THE BLOCK HEIGHT)

TOP BOUNDARY TO CONCRETE

CONCLUSION: 1 ANCHOR EVERY 80" (20% OF SPECIFIED)

SAMPLE: 10 TESTS

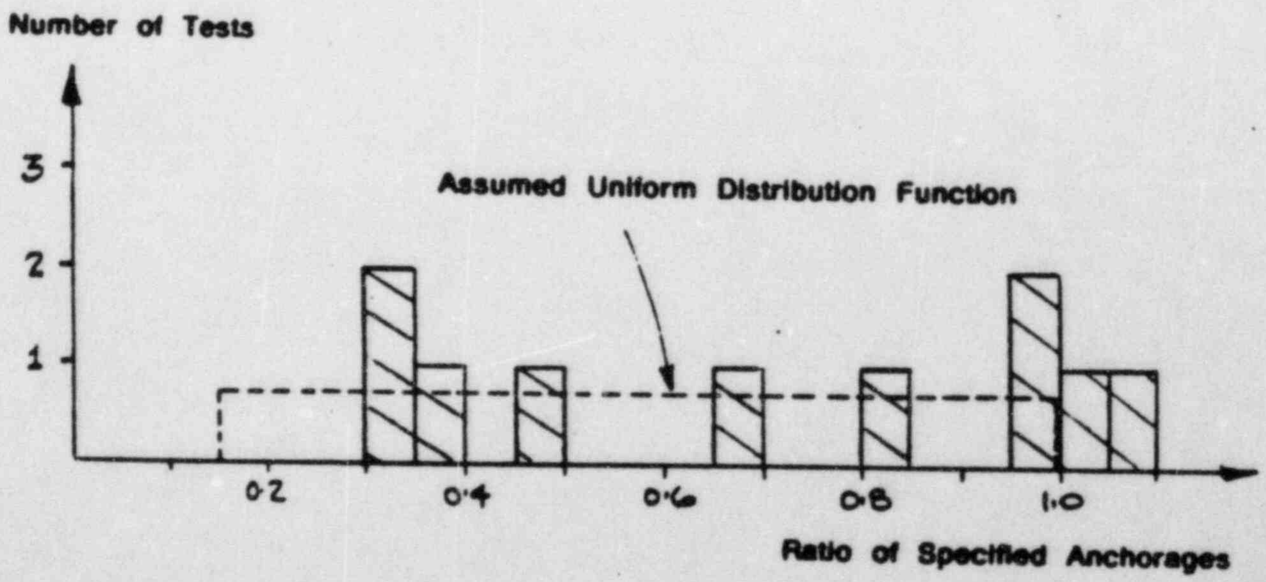
$X = 0.71$ $s = 0.29$ $\rho_{AL} = .09$

NO ZERO DATA POINTS

WEIGHT SHOULD BE PLACED EQUALLY ON ALL DATA POINTS DUE TO LOW CORRELATION BETWEEN NUMBER OF ANCHORS AND EXPOSED LENGTH.

TOP ANCHORAGES TO CONCRETE

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH	LENGTH / UNIT ANCHORAGE
TC-1	45.2	66.0	2	0.03030	33.0
TC-2	62.2	48.0	2	0.04167	24.0
TC-3	64.12	38.0	2	0.05263	19.0
TC-4	66.6	30.0	2	0.06667	15.0
TC-5	66.11	48.0	1	0.02083	48.0
TC-6	184.7	48.0	3	0.06250	16.0
TC-7	184.8	49.0	3	0.06122	16.3
TC-8	188.10	49.0	3	0.06122	16.3
TC-9	209.0	44.0	1	0.02273	44.0
TC-10	212.1	48.0	1	0.02083	48.0



TOP BOUNDARY TO CONCRETE

A UNIFORM DISTRIBUTION IS ASSUMED

LOWER AND UPPER LIMITS ARE ESTABLISHED BY FITTING MEAN AND STANDARD DEVIATION OF THE SAMPLE TO THOSE OF THE ASSUMED DISTRIBUTION.

THIS CONFIRMS AN UPPER LIMIT OF 1.0. LOWER LIMIT IS ESTABLISHED CONSERVATIVELY AT 0.15.

USING THIS DISTRIBUTION. WE ACCEPT A STRENGTH BASED ON 1 ANCHOR EVERY 80"

THIS IS SOUNDLY CONFIRMED BY ALL DATA POINTS.

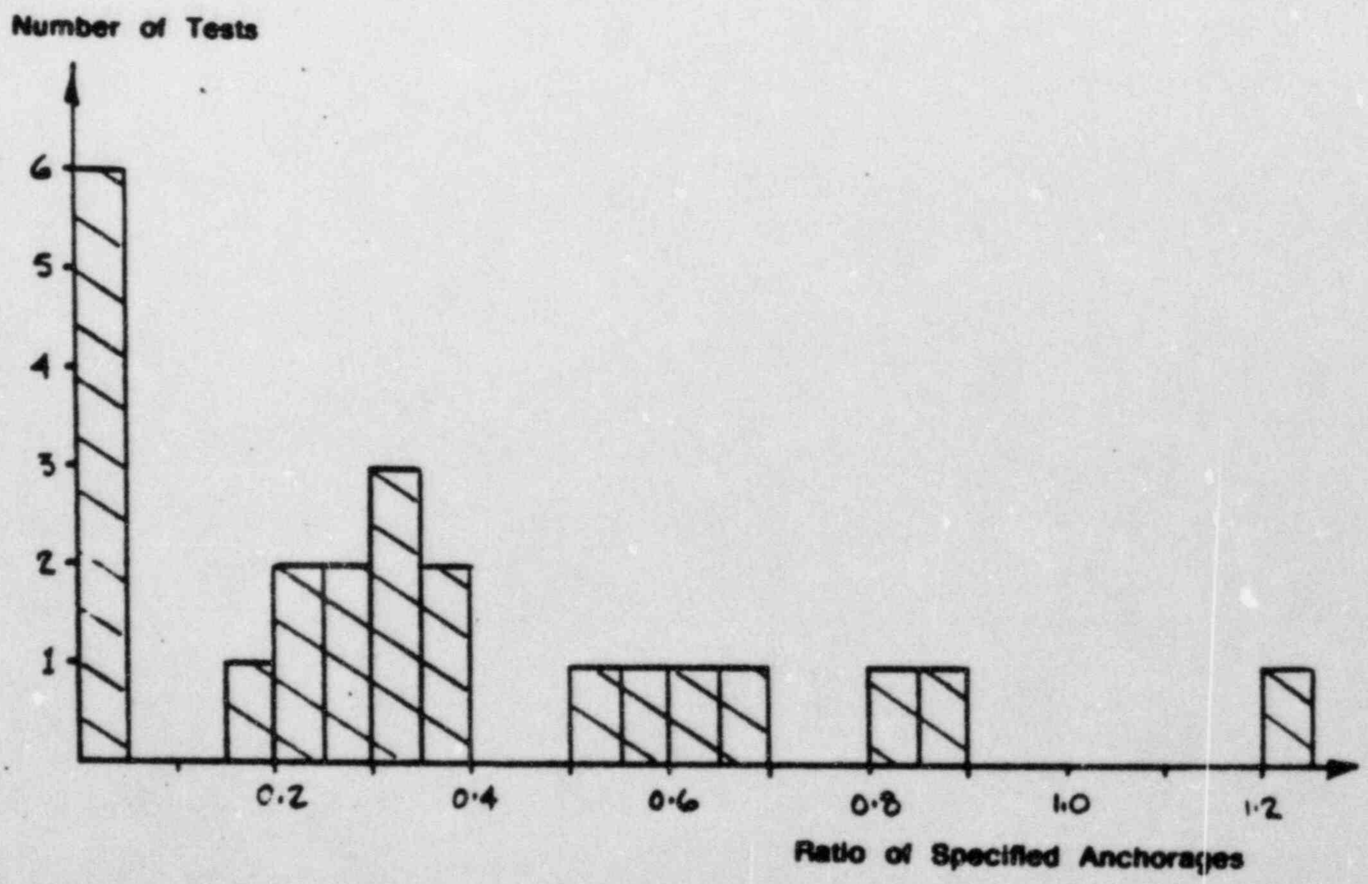
TOP BOUNDARY TO STEEL

6 OF 23 TESTS HAVE ZERO ANCHORS. AND 4 OF THESE 6 ARE FROM TESTS WITH CONSIDERABLE LENGTHS.

RELY ON NO ANCHORS FOR THIS BOUNDARY TYPE.

TOP ANCHORAGES TO STRUCTURAL STEEL

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH
TS-1	64.4	38.0	2	0.05263
TS-2	64.4	36.0	2	0.05556
TS-3	65.16	43.0	1	0.02326
TS-4	65.21	60.0	1	0.01667
TS-5	66.0	42.0	1	0.02381
TS-6	66.6	13.0	1	0.07692
TS-7	66.6	51.0	2	0.03922
TS-8	66.18	50.0	0	0.00000
TS-9	67.1	58.0	1	0.01724
TS-10	68.10	48.0	0	0.00000
TS-11	184.2	38.0	0	0.00000
TS-12	184.4	48.0	1	0.02083
TS-13	188.2	48.0	0	0.00000
TS-14	188.9	47.0	0	0.00000
TS-15	191.26	73.0	1	0.01370
TS-16	191.35	81.0	1	0.01235
TS-17	191.55	48.0	1	0.02083
TS-18	194.23	49.0	1	0.02041
TS-19	195.14	48.0	2	0.04167
TS-20	195.22	68.0	1	0.01471
TS-21	196.6	57.5	2	0.03478
TS-22	196.7	10.0	0	0.00000
TS-23	196.7	60.5	2	0.03306



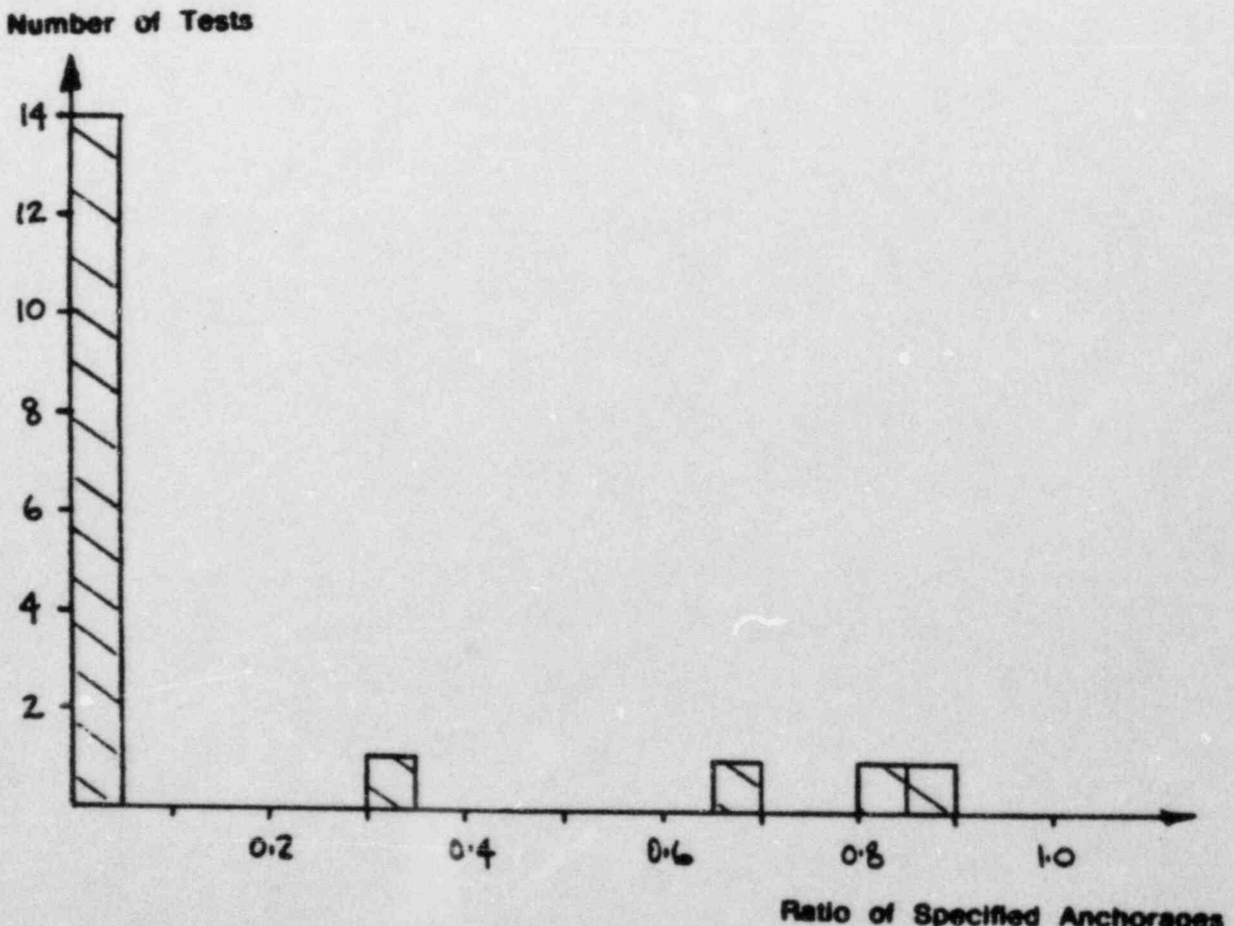
TOP BOUNDARY TO Q-DECK

14 OF 18 TESTS FOUND ZERO ANCHORS.

RELY ON NO ANCHORS FOR THIS BOUNDARY TYPE.

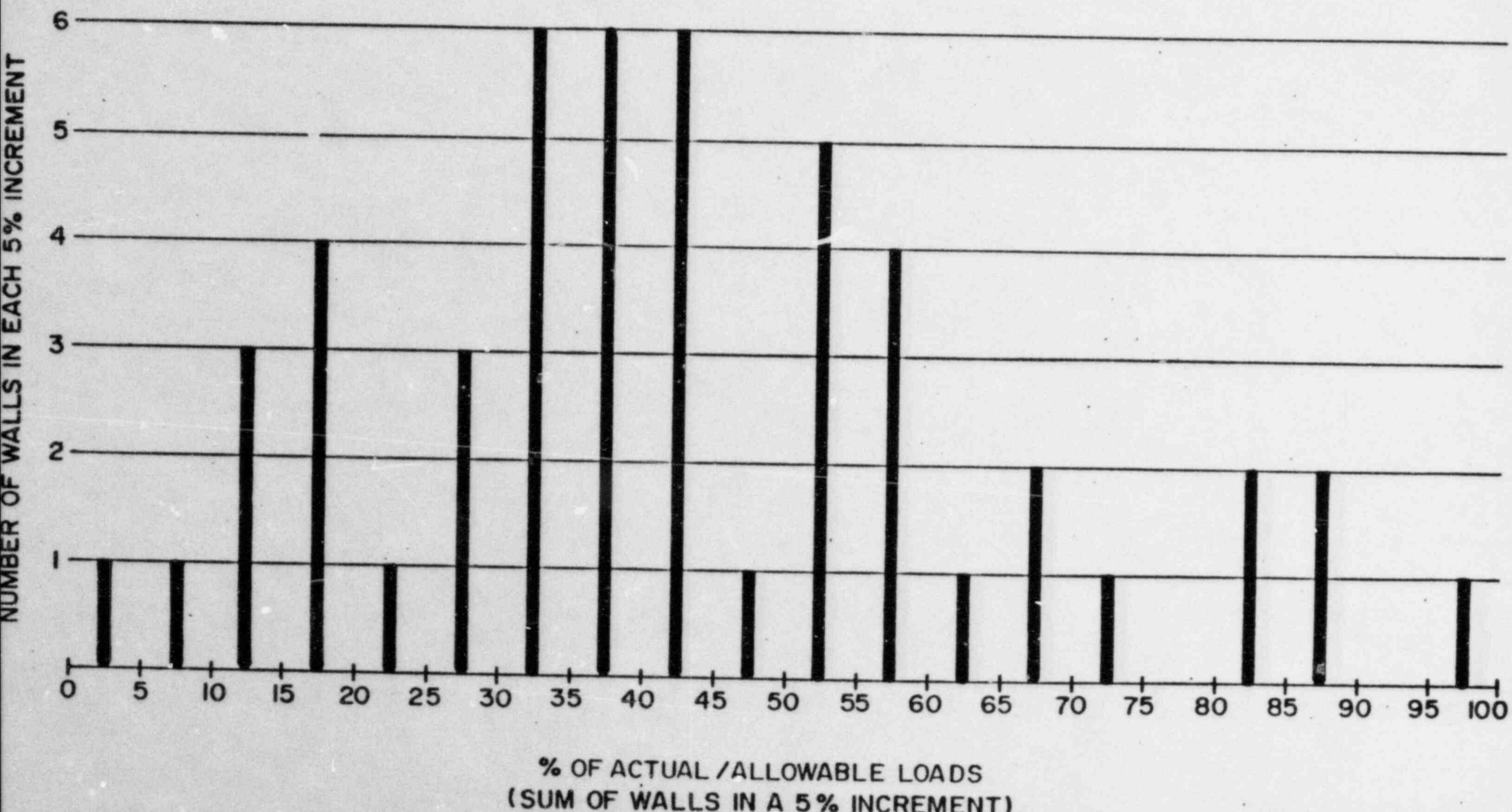
TOP ANCHORAGES TO Q-DECK

TEST NUMBER	WALL NUMBER	EXPOSED LENGTH	NUMBER OF ANCHORAGES	ANCHORAGES / UNIT LENGTH
TQ-1	63.5	53.0	0	0.00000
TQ-2	64.6	51.0	1	0.01961
TQ-3	66.2	72.5	0	0.00000
TQ-4	66.5	28.0	0	0.00000
TQ-5	66.5	16.0	0	0.00000
TQ-6	66.7	48.0	2	0.04167
TQ-7	66.12	47.0	0	0.00000
TQ-8	111.7	40.0	2	0.05000
TQ-9	188.3	56.0	3	0.05357
TQ-10	191.49	64.0	0	0.00000
TQ-11	194.21	59.0	0	0.00000
TQ-12	194.21	47.0	0	0.00000
TQ-13	194.22	49.0	0	0.00000
TQ-14	194.25	32.0	0	0.00000
TQ-15	195.18	41.0	0	0.00000
TQ-16	195.23	6.0	0	0.00000
TQ-17	195.23	35.0	0	0.00000
TQ-18	195.23	12.0	0	0.00000



ATTACHMENT 8 (Response to Item 4 of Attachment 1)

NUMBERS OF WALLS QUALIFIED USING
STATISTICAL APPLICATION OF TESTS
(TOTAL = 50 WALLS)



					TOP				SIDE #1				SIDE #2			
WALL #	LOCATION	H	W	T	TYP	ACT	ALL	%	TYP	ACT	ALL	%	TYP	ACT	ALL	%
45.2	INTAKE 21'-6"	14'	14'	8"	C	53	187	28	C	52	93	56	C	52	93	56
62.3	REACT 23'-0"	8'	6'	30"	F	0	0	0	C	52	185	28	F	0	0	0
62.5	REACT 23'-0"	28'	9'	12"	F	0	0	0	C	47	93	51	ML	47	160	29
63.0	REACT 23'-0"	8'	18'	12"	F	0	0	0	C	48	93	52	F	0	0	0
63.1	REACT 23'-0"	26'	40'	30"	Q2	5	480	1	C	1	185	1	C	20	185	11
63.3	REACT 23'-0"	8'	5'	18"	F	0	0	0	C	84	185	45	F	0	0	0
63.5	REACT 23'-0"	26'	29'	30"&24"	Q2	117	360	33	ML	155	320	48	C	161	185	87
63.7	REACT 23'-0"	26'	7'	24"	F	0	0	0	C	154	185	83	ML	154	320	48
63.8	REACT 23'-0"	8'	22'	12"	C	38	90	42	C	0	93	0	F	0	0	0
63.9	REACT 23'-0"	8'	5'	12"	C	32	90	36	C	0	93	0	F	0	0	0
65.10	REACT 51'-0"	8'	8'	8"	C	76	90	84	ML	50	100	50	F	0	0	0
65.2	REACT 51'-0"	20'	23'	45"	Q2	477	720	66	ML	188	480	39	C	188	278	68
65.20	REACT 51'-0"	10'	13'	8"	C	40	90	44	ML	55	100	55	ML	55	100	55
65.4	REACT 51'-0"	20'	29'	48"	Q2	230	720	32	ML	200	480	42	C	50	278	17
65.6	REACT 51'-0"	10'	5'	12"	Q1	0	0	0	C	84	93	89	F	0	0	0
65.7	REACT 51'-0"	21'	11'	42"	Q1	0	0	0	ML	89	278	32	C	86.8	278	31
66.22	REACT 74 -3"	8'	5'	12"	F	0	0	0	ML	54	160	34	C	54	93	58
66.24	REACT 74'-3"	8'	5'	12"	F	0	0	0	ML	54	160	34	C	54	93	58
66.5	REACT 74'-3"	15'	22'	8"	Q2	48	120	40	C	31	93	33	ML	31	100	31
66.7	REACT 74'-3"	15'	33'	8"	Q2	27	120	23	MT	0	0	0	C	2.4	93	1
68.10	REACT 74'-3"	14'	9'	12"	W	0	0	0	C	16	93	17	C	30	93	32
68.2	REACT 74'-3"	11'	8'	30"	F	0	0	0	C	96	185	52	C	96	185	52
68.3	REACT 74'-3"	11'	10'	36"	F	0	0	0	C	97	185	52	C	97	185	52
68.4	REACT 74'-3"	11'	10'	36"	Q2	146	480	30	C	131	185	71	C	131	185	71
68.8	REACT 74'-3"	8'	18'	18"&24"	F	0	0	0	ML	86	260	33	C	51	185	28
77.0	REACT 13'-9"	7'	10'	18"	Q2	46	240	19	ML	46	60	77	C	72	185	39
111.2	REACT 61'-4"	11'	11'	30"	W	0	0	0	C	83	185	45	F	0	0	0
111.5	REACT 61'-0"	8'	8'	30"	Q2	148	480	31	C	178	185	96	ML	226	380	59
116.6	REACT 42'-3"	7'	8'	12"	Q1	0	0	0	C	63	127	50	C	40.8	127	32
185.10	AUX (-)17'-6"	18'	9'	8"	C	24.6	90	27	ML	24	100	24	ML	24.3	100	24
185.3	AUX 3'-0"	13'	13'	24"	Q1	0	0	0	C	31	185	17	F	0	0	0
185.4	AUX 3'-0"	13'	18'	24"	Q1	0	0	0	C	71	185	31	C	70	185	38
185.6	AUX 3'-0"	13'	6'	24"	F	0	0	0	ML	36	320	11	C	36	185	19
185.7	AUX (-)17'-6"	18'	6'	8"	F	0	0	0	ML	38	100	38	C	38	93	41
185.9	AUX (-)17'-6"	18'	6'	8"	F	0	0	0	ML	34	100	34	C	34	93	37
188.1	AUX 23'-0"	13'	6'	8"	Q2	64	120	53	MT	0	0	0	C	33	93	35
188.10	AUX 23'-0"	11'	41'	8"&12"	C	52	90	58	ML	25	100	25	ML	78	127	61
188.3	AUX 23'-0"	14'	18'	8"	C	39	90	43	ML	51	100	51	C	51	93	54
188.6	AUX 23'-0"	14'	4'	24"	F	0	0	0	C	38	185	21	C	38	185	21
188.7	AUX 23'-0"	13'	7'	24"	F	0	0	0	C	60	185	32	C	60	185	32
191.33	RADWST(-)1'-0"	8'	8'	8"	C	11	90	12	ML	13	100	13	ML	13	100	13

Attachment 8a (Response to Item d of Attachment 1)

TABLE 4-1 (Sheet 1 of 2)

WALL #	LOCATION	H	W	T	TOP TYP	TOP ACT	TOP ALL	TOP %	SIDE #1 TYP	SIDE #1 ACT	SIDE #1 ALL	SIDE #1 %	SIDE #2 TYP	SIDE #2 ACT	SIDE #2 ALL	SIDE #2 %
191.55	RADWST(-)1'-0"	21'	17'	8"	W	27	37	73	W	22	127	17	C	13	93	14
194.25	RADWST 23'-0"	11'	7'	8"	Q2	10	120	8	MT	0	0	0	C	11	93	12
195.19	RADWST 37'-0"	11'	19'	8"	Q2	19	120	16	MT	0	0	0	C	62	93	67
196.7	RADWST 51'-0"	16'	9'	8"	W	113	142	80	MT	0	0	0	C	32	93	34
198.1	GEN 23'-0"	11'	13'	8"	C	34	90	38	ML	29	100	29	C	29	93	31
198.2	GEN 23'-0"	11'	13'	8"	C	58	90	64	C	0	93	0	F	0	0	0
198.4	GEN 23'-0"	9'	5'	18"	W	0	0	0	C	81	185	44	F	0	0	0
209.0	TURBINE 23'-0"	13'	31'	8"	Q2	65	120	54	C	35	93	38	MT	0	0	0
210.3	TURBINE 37'-0"	2'	11'	12"	C	9	90	10	ML	0	160	0	F	0	0	0

BOUNDARY TYPES:
- C Reinforced Concrete
- MT Masonry T (non-interlocking)
- ML Masonry L (interlocking)
- F Free
- W Structural Steel
- Q1 Q deck with flutes perpendicular to wall
- Q2 Q deck with flutes parallel to wall

TABLE 4-1 (Sheet 2 of 2)

ATTACHMENT 9 (Response to Item 6 of Attachment 7)

SAFETY RELATED BLOCKOUTS

WALL.. NUMBER	WALL LOC (1)	WALL LOC (2)	SFTY RTNG
116.1	REACT	36-3	SR1
116.2	REACT	36-3	SR1
116.3	REACT	36-3	SR1
116.5	REACT	36-3	SR1
977.1	RADWASTE	51-0	SR1
979.11	AUX	23-0	SR1
979.12	AUX	23-0	SR1
979.3	TURBINE	30-0	SR1
980.9	TURBINE	37-0	SR1
982.1	REACT	-17-6	SR1
982.2	REACT	-17-6	SR1
982.3	REACT	-17-6	SR1
982.4	REACT	-17-6	SR1
984.1	REACT	-17-6	SR1
984.2	REACT	-17-6	SR1
984.3	REACT	-17-6	SR1
984.4	REACT	-17-6	SR1
984.5	REACT	-17-6	SR1
984.8	REACT	2-9	SR1
984.9	REACT	51-0	SR1

Attachment 9a (Response to item 6 of Attachment 1)

DESIGN CRITERIA FOR MASONRY BLOCKOUTS

I. Background

During the construction phase of the Pilgrim Station, numerous openings in reinforced concrete walls were made to allow for installation of piping and conduit runs later on in the construction phase. At the later phase of construction, when piping and conduit had been installed, the openings in concrete walls were closed in many cases using masonry units. In general, the location of such "blockouts" are shown on the mechanical drawings. It is the position of the Boston Edison Company that masonry blockouts are not masonry walls, and thus do not fall under the scope of NRC I&E Bulletin 80-11, "Masonry Wall Design." It is, however, prudent to examine masonry blockouts with consideration of potential impact to safety systems. This document details the requirements and acceptance criteria for structural evaluation of masonry blockouts.

II. Scope

This criteria applies to the use of masonry units wholly surrounded by reinforced concrete. There shall be no size limitation to the use of masonry in this classification as "blockout" except that floor to ceiling masonry installations shall be considered walls and shall be governed by the criteria developed for BECo's response to Bulletin 80-11.

III. Design Assumption

The following assumptions shall apply to analysis of masonry blockouts.

1. There shall be no credit taken for mechanical anchorage either horizontal or vertical between the masonry blockouts and surrounding concrete.

2. Blockouts do not contribute to the overall structural behavior of the surrounding concrete walls. Drawing C-121, Section A, shows additional wall reinforcing at blockout locations.

3. Anchorage is provided by mortar bond between the masonry and the surrounding concrete. The ultimate shear stress for the mortar joint is taken as 83 psi. For conservatism this ultimate value shall be reduced by a factor of 2 for SSE, PBOC, and tornado loads; and by a factor of 4 for OBE loads.

4. Blockouts shall be assumed to respond to out of plane loads as a monolithic whole. The predominant failure mechanism will be a shearing of the mortar bond, followed by the sliding of the masonry blockout out of the concrete in which it is confined.

5. This criteria is applicable to masonry blockouts that remain uncracked when:

   a) assumed simply supported at top and bottom; and,

   b) analyzed as a one-way span vertically.

6. Actual shear stress is assumed to be uniformly distributed across the mortared surface area (excluding hollow cells).

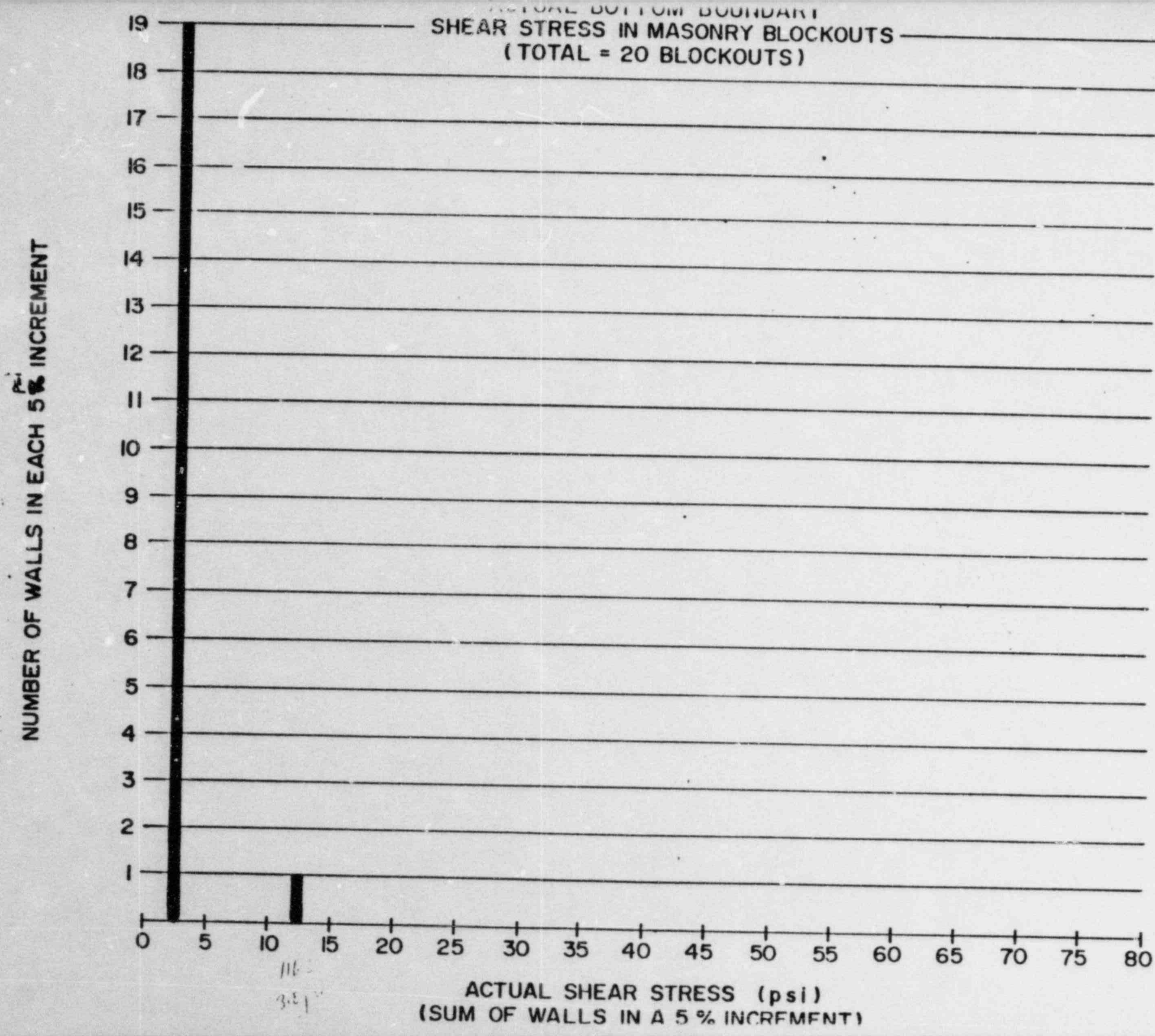
IV. Procedure

All masonry blockouts shall be examined in accordance with NEDWI 289, Rev. 0, "Walkdown of New Blockwall Scope," to determine relative potential to impact safety systems. Only those blockouts which are determined to have the potential to damage a safety related system or component shall be further analyzed for structural ability to withstand loads (safety related blockouts). All safety related blockouts shall be analyzed for the loading combinations set forth in the Design Criteria for Masonry Walls DC-1, Rev. 1, September 14, 1981. Acceptance criteria for masonry blockouts shall be based on the assumptions listed in Item III above.

Modifications, if any, shall be designed to the loading combinations and the acceptance criteria for masonry walls and design criteria DC-1.

V. Quality Assurance Requirements

Modifications to masonry blockouts shall be examined on a case by case basis. Where the blockout is used directly to support a Class I system or component, or where the blockout forms a part of secondary containment, the modifications shall be designated Class I and subject to the quality control requirements of quality category "Q." Where the masonry blockout forms an integral part of a designated three hour fire barrier, modifications shall be designated "FPQ." All other masonry blockouts which have an <u>indirect</u> impact on safety systems, i.e., the creation of a II/I situation, shall be modified to the procurement and installation requirements of Class II (quality category "C"). This is consistent with PNPS FSAR Chapter 12.2 which provides that "Class II designated structures and/or equipment shall not degrade the integrity of any structures and/or equipment designated Class I."


SHEAR STRESS IN MASONRY BLOCKOUTS
(TOTAL = 20 BLOCKOUTS)
NUMBER OF WALLS IN EACH 5 psi INCREMENT
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
ACTUAL SHEAR STRESS (psi)
(SUM OF WALLS IN A 5 % INCREMENT)

ATTACHMENT 10

ACTION ITEMS AND MEETING MINUTES

(JULY 18, 1984)

1. Based on discussions at the meeting and information provided in Attachment 5, the staff accepted the criterion used by the licensee in determining shear loads for the bottom boundary of walls.

2. The NRC staff accepted the block-out criteria (Attachment 9) proposed by the licensee, provided following conditions are met:

   a. The licensee to survey all block-outs (except 116.3) for cracks on boundaries - acceptance of criteria based on no evidence of through cracks;

   b. Provide the results of survey for the staff's review and acceptance, and

   c. For block-out 116.3, provide modifications (on one face) to boundaries to resist peak load resulting from the load combination involving PBOC load components. Notify the staff if tornado differential pressures are greater than 1.5 psi and not acting in the same direction as the PBOC load.

3. The licensee will provide representative calculations to show differences between prior Cygna analysis and subsequent refined analysis for walls qualified without reliance on the statistically determined line loads.

4. The licensee provided considerable information on the statistical analysis at the meeting. The NRC staff will inform the licensee regarding the acceptance of statistical concept and need for any further actions pending discussions with the NRC management.

5. The licensee is still reviewing alternate qualification scheme for walls 209.13 and 209.14 and will discuss with the staff at a later date.

6. The licensee no longer expects to pursue delaying any modification until the 1986 refueling outage (item 7 of action items of June 7, 1984 meetings).

7. The licensee provided calculations for walls 64.4, 63.4 and 188.10 at the meeting. The NRC staff will review these calculations and advise the licensee of any outstanding issues resulting from this review.