Enclosure 2

Justification for Various Aspects of
the Re-evaluation Criteria in response to
NRC's December 22, 1981 request for
additional information concerning
IE Bulletin No. 80-11 "Masonry Wall Design"

Consolidated Edison Company of New York, Inc.
Indian Point Unit No. 2
Docket No. 50-247
August, 1982

INDIAN POINT GENERATING STATION, UNIT 2

REEVALUATION OF MASONRY WALLS

RESPONSE

TO

NUCLEAR REGULATORY COMMISSION REVIEW

Prepared for

CONSOLIDATED EDISON CO.
New York, N.Y.

by

COMPUTECH ENGINEERING SERVICES, INC.
2855 Telegraph Avenue
Berkeley, CA. 94705

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1   INTRODUCTION

The Nuclear Regulatory Commission (NRC) review of the criteria and justification for the evaluation of masonry walls at the Indian Point. Unit 2 resulted in a request for additional information on a number of items.

Some of the information was provided to the NRC in a submittal from Consolidated Edison dated 24 February, 1982. In the submittal it was noted that a more detailed response on a number of the items would be forwarded at a later date. In the following sections the more detailed response on these items is presented. These items refer to the single wythe analysis of multi-wythe walls and also values of allowable shear strains and allowable tensions given in the criteria.

2 SINGLE vs MULTI–WYTHE ANALYSIS

Item 3 of the NRC request for additional information stated:

> "The licensee should justify the use of single wythe
> analysis for multiple wythe walls".

The masonry walls in the Primary Auxiliary Building are multi–wythe concrete block walls. Because of the difficulty in verifying the adequacy of the collar joints between the wythes all wythes were assumed to respond as single wythe walls. This was assumed to be conservative and in the following sections the results of a number of analyses to validate this assumption are presented.

2.1 Analysis Methodology

Four multi–wythe walls were selected to compare the results of analyses assuming both single wythe and multi wythe action. These walls were selected from the 21 Primary Auxiliary walls on the basis of the following criteria:

a.   Walls incorporating the full range of two, three and four wythes were represented.

b.   Walls both with and without added equipment loads were included.

c.   The selected walls represented the maximum dimensions of all the walls and thus had the highest ratios of maximum moment to allowable moment when analyzed as single wythe walls.

The four walls selected were as follows:

Wall 5-059-03	2-6" wythes	= 12"
Wall 5-080-07	3-8" wythes +	
	1-6" wythe	= 30"
Wall 5-098-23	3-8" wythes	= 24"
Wall 5-098-31	3-8" wythes	= 24"

All walls were analyzed in accordance with the procedures given in "Criteria for the Reevaluation of Masonry Walls for Indian Point, Unit 2". Finite element plate analyses were used to assess the out-of-plane response of the selected walls using the computer program SAP5A to perform dynamic analyses by the response spectrum technique.

2.2 Results on Analysis

Tables 2.1 and 2.2 summarize the results from the analyses of the walls. In Table 2.1 the effect of the single wythe assumption on the frequencies is tabulated. Table 2.2 lists the maximum stress ratios obtained. For wall 5-080-07 which had wythes of both 6" and 8" the single wythe analysis was carried out for each of these thicknesses.

2.3 Discussion of Results

From the results presented in Tables 2.1 and 2.2 the following points may be noted:

a. The frequency of all walls increases for multi-wythe response compared with single wythe response. All multi-wythe walls had frequencies in the rigid range so that the spectral acceleration would be the ZPA without amplification.

b. In all cases the maximum stress ratios reduced for multi-wythe analysis, by factors ranging from 3 to 16.

c. The maximum reduction in stress ratios occurred for the walls with equipment loads. This was because the equipment loads remained the same but was distributed over the greater wall thickness. For walls without added mass the wall mass and thus inertia forces increased proportionately to the thickness.

2.4 Summary

Four walls were used to demonstrate that the single wythe assumption for multiple wythe walls results in a conservative analysis. The frequencies for the multi-wythe walls were increased to the rigid range and the stress ratios reduced several times if composite action was assumed.

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WALL NUMBER	THICKNESS (inches)	NUMBER OF WYTHES	FREQUENCY (hz)				
			MODE 1	MODE 2	MODE 3	MODE 4	MODE 5
5-093-03	6	1	15.78	23.86	37.46	55.37	56.66
	12	2	33.65	50.91	79.91	118.1	
5-098-31	8	1	30.14	67.40	83.84	121.5	
	24	3	94.86	212.1			
5-080-07	6	1	27.24	59.31	77.35	109.8	
	8	1	36.92	80.39	104.9		
	30	4	145.2				
5-098-23	8	1	24.41	54.59	67.91	98.44	
	24	3	76.84	171.8			

NOTE:

1. Wall numbers 5-098-31 and 5-080-07 included added loads from equipment. The remaining two walls had self-weight only.

2. Wall 5-080-07 has both 6" and 8" wythes and so was analyzed for each of these thicknesses.

TABLE 2.1 : COMPARISON OF SINGLE AND MULTI WYTHE FREQUENCIES

WALL NUMBER	THICKNESS (inches)	NUMBER OF WYTHES	$\dfrac{M_x}{M_{xa}}$	$\dfrac{M_y}{M_{ya}}$
5-059-03	6	1	0.552	0.283
	12	2	0.127	0.067
5-098-31	8	1	0.254	0.379
	24	3	0.034	0.043
5-080-07	6	1	0.597	0.745
	8	1	0.342	0.425
	30	4	0.036	0.042
5-098-23	8	1	0.131	0.151
	24	3	0.039	0.045

NOTE:

1. Wall numbers 5-098-31 and 5-080-07 included added loads from equipment. The remaining two walls had self-weight only.

2. Wall 5-080-07 has both 6" and 8" wythes and so was analyzed for each of these thicknesses.

TABLE 2.2 : COMPARISON OF SINGLE AND MULTI WYTHE MOMENT RATIOS

3 ALLOWABLE SHEAR STRAINS

Request number 7 from the NRC review stated:

> "The licensee should justify the proposed 67% increase
> in gross shear strains for factored loads".

In the following sub-sections data from an extensive series of tests is statistically analyzed to provide probabilities of exceedance and demonstrate that the 67% increase factor is reasonable.

3.1 Overview of Test Program

The results from an ongoing masonry test program being performed at the Earthquake Engineering Research Center, University of California, Berkeley, were used to evaluate the in-plane shear strength of masonry piers. The tests subjected masonry piers to an in-plane cyclic shear load with the test setup shown in Figure 3-1. The results of the research have been reported in References 3.1, 3.2, 3.3, 3.4 and 3.5. The piers are tested by applying three cycles of load at a specified amplitude. The amplitude is gradually increased as the test progresses until the pier is unable to resist any further load. Each test was photographed after each set of three cycles of load, thereby providing detailed records of the crack pattern.

To date over one hundred piers have been tested using three different types of materials. Thirty-five of the piers tested were constructed from hollow concrete block masonry units and of these six had a height-to-width ratio of 0.5, fifteen had a height-to-width ratio of 1, and fourteen had a height-to-width ratio of 2. The piers were constructed from either 6-inch or 8-inch wide hollow concrete block units using Type M mortar. The strength of prisms constructed from the same materials that were used in the piers varied from 1350 to 3500 psi.

The information obtained from each test consisted of a plot of the force-deflection relationship for each cycle of loading. From this set of curves several parameters could be determined, including:

    (a)   Ultimate Strength

    (b)   Stiffness Degradation

    (c)   Hysteresis Envelope

    (d)   Deflection of Pier at each Loading Stage

3.1.1 Applicability of Test Results

The information obtained from the Berkeley test program is valuable in evaluating the in-plane shear performance of masonry piers subjected to seismic loads. A discussion on the applicablity of the test results is discussed separately with respect to the following variables -- loading, size of test specimen, boundary conditions, material strengths and reinforcement.

A. Loading

Although an earthquake type time history was not used as the input motion to the test specimen, the gradually increasing, amplitude dependent, cyclic loading was typical of that used in many other test programs on reinforced concrete and steel structural elements. The most important aspect of loading required to evaluate the seismic performance of structural elements is that the loading be cyclic or reversed. Other variables such as the rate of loading, sequence of loads, etc., may be important but are secondary in comparison to the requirement that the loading be cyclic.

B. Size of Test Specimen

The size of the test specimen used in the Berkeley test program was limited by the capacity of the actuators. The piers with a height-to-width ratio of 0.5 were 3 ft. 4 inches high and 6 ft. 8 inches long, the 1 to 1 piers were 4 ft. 8 inches high and 4 ft. long whereas the 2 to 1 piers were 5 ft. 4 inches high by 2 ft. 8 inches long. Although these sizes are generaly smaller than the walls found in Indian Point, Unit 2, it is assumed that they are of adequate size to represent the behavior of larger sized walls with the same aspect or height-to-width ratio. It should be noted that no experimental evidence is available to validate or refute this assumption.

The aspect or height-to-width ratios included in the test program cover all the walls at Indian Point, Unit 2.

C. Boundary Conditions

The boundary conditions of the piers tested in the Berkeley program were such that moment fixity was forced at both the top and bottom of the piers with no constraints on the vertical edges. Although this set of boundary conditions is different from that of most of the walls at Indian Point, Unit 2, it is believed that if the walls at the plant are confined either on three or four sides or at the top and bottom, then the performance of the walls will be similar to those tested in the Berkeley program. Confinement should be

provided by either walls or columns capable of resisting the loads imposed by the concrete block walls.

D. Material Strengths

The assumed compressive strength $f'm$ of the walls at Indian Point, Unit 2 was 750 psi. This is lower than the range of 1350-3500 psi of the prism strength of the piers included in the Berkeley test program. However, it is our opinion that the actual insitu $f'm$ of the walls is within the range tested in the Berkeley program.

E. Reinforcement

All the walls at Indian Point, Unit 2 are unreinforced whereas the piers of the Berkeley test program were reinforced. It is our belief that provided the walls at the Indian Point, Unit 2 are confined on three or four sides or at the top and bottom, then cracks in the unreinforced wall will occur at similar strain levels to the piers tested.

3.2 Evaluation of Test Data

The data from thirty-five tests performed on hollow concrete block piers was evaluated on the basis of shear strain and details of this evaluation are presented in the following sub-sections.

3.2.1 Shear Strains for OBE and SSE Events

The test results from the Berkeley program were evaluated to determine in-plane shear strains appropriate for OBE and SSE events. The evaluation was performed so that the function of a wall would not be impaired while it was resisting out-of-plane loads. During each pier test, photographs were taken after each set of three cycles of load at a specified amplitude. These photographs in conjunction with the hysteresis envelopes developed for each test were used to determine the appropriate state of cracking due to in-plane loads that could be tolerated for an OBE and SSE event. For an OBE event, the loading stage at which initial visible cracks occurred was used. For an SSE event, additional cracking was permitted, however, the loading stage was prior to any significant diagonal cracking. Obviously the evaluation for an SSE event required judgment and photographs shown in Figures 3-2, 3-3, and 3-4 show the typical state of cracking used for both an OBE and SSE event for piers with height-to-width ratios of 0.5, 1, and 2, respectively. At each level of cracking the corresponding displacement was determined. The displacements were then divided by the wall height to determine the corresponding shear strains which were statistically evaluated as presented in the following subsection.

3.2.2 Statistical Analysis of the Data

A total of 34 and 35 tests were used to evaluate the shear strain for the OBE and SSE events respectively. The shear strains were determined by the procedure described in the foregoing subsection. From this data the following parameters were calculated for the shear strain for both the OBE and SSE events.

(i)   Sample mean, $\bar{X}$

(ii)   Sample standard deviation, s

These statistics were then used as the parameters for the distribution of the population. For each case (OBE and SSE), two underlying distributions were assumed, and the effect of the choice of distribution on the results was examined. The more reasonable distribution was then accepted. The two underlying distributions were the normal distribution and the gamma distribution, and the gamma distribution was chosen as best representing the test data for reasons given later in this section.

The 95% confidence interval for the mean of the population M_x was calculated, assuming that the normalized variable

$$\frac{\bar{X} - M_x}{S/\sqrt{n}}$$

is t-distributed, and that the actual population standard deviation, σ_x, is unknown. Here n is the sample size.

For the gamma distribution, confidence intervals on parameters such as $M_x - k\sigma_x$ have no meaning and must be reinterpreted. On the normal curve $M - 1\sigma$ corresponds to a point on the cumulative distribution curve with an ordinate of 0.1587. This means that approximately 16% of the area under the probability density curve lies to the left of $M - 1\sigma$. Similarly $M - 2\sigma$ and $M - 3\sigma$ correspond to points with ordinates 0.02275 and 0.00135 respectively. Based on the confidence interval for the mean, confidence intervals were calculated for values of the gamma distribution for which the cumulative distribution function had values of 0.1587, 0.0025 and 0.00135 respectively.

These actual distributions were then compared with the criteria specified allowable shear strains, i.e., 0.0008, for the OBE condition, and 1.67 times this value, for the SSE condition. Probabilities that the criteria specified allowable strain would exceed the actual strain based on the test results were calculated under two assumptions: firstly, that the population mean was equal to the sample mean, and secondly, that it was at the lower end of the 95% confidence interval.

Finally, safety factors based on the 95% confidence interval for the mean were calculated for the shear strain. The results are presented below.

	OBE	SSE
Sample Size	34	35
Sample Mean	0.00202	0.00318
Sample Standard Deviation	0.00085	0.00094
Coefficient of Variation	42%	30%

The 95% confidence intervals on the population mean are:

$$\text{OBE } 0.00172 \leqslant M \leqslant 0.00232$$
$$\text{SSE } 0.00286 \leqslant M \leqslant 0.00350$$

The effect of the assumption of normal distribution versus the assumption of gamma distribution was studied. Plots of the histograms of test data for both the OBE and SSE conditions are shown in Figure 3-5. Two observations are as follows.

(i) The data never takes on negative values.

(ii) The distribution of data is skewed, especially for the OBE condition.

Both of these observations indicate that the gamma distribution is preferable to the normal distribution. The gamma distribution is defined by

$$f_x (x) = \frac{\lambda(\lambda x)^{k-1} e^{-\lambda x}}{(k-1)!} \qquad x \geqslant 0$$

and has a mean value of k/λ and a coefficient of variation of $1/\sqrt{k}$. The following values of k and λ give best fits to the OBE and SSE data:

Case	k	λ
OBE	6	2970.3
SSE	11	3459.1

These curves are also plotted in Figure 3-5.

It should be noted that there is no physical reason why shear strains should have a gamma distribution. However, by suitable adjustment of the parameters k and λ the gamma distribution can be made to describe the best data far more accurately than can the normal distribution.

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The 95% confidence intervals corresonding to the 1σ, 2σ, and 3σ levels are as follows:

(i)   Corresponding to cumulative distribution function = 0.1587 (1σ level)

         OBE $0.00090 \leqslant \bar{X} \leqslant 0.00146$
         SSE $0.00192 \leqslant \bar{X} \leqslant 0.00257$

(ii)   Corresponding to cumulative distribution function = 0.02275 (2σ level)

         OBE $0.00045 \leqslant \bar{X} \leqslant 0.00091$
         SSE $0.00129 \leqslant \bar{X} \leqslant 0.00189$

(iii)   Corresponding to cumulative distribution function = 0.00135 (3σ level)

         OBE $0.00020 \leqslant \bar{X} \leqslant 0.00053$
         SSE $0.00081 \leqslant \bar{X} \leqslant 0.00134$

Using the allowable strain values from the criteria for confined walls and the above 95% confidence interval on the mean the following limits on the factor of safety are established:

OBE: $2.15 \leqslant SF \leqslant 2.90$

SSE: $2.13 \leqslant SF \leqslant 2.61$

The probabilities that the criteria specified strain values will exceed the available strain capacity based on test results and the gamma distribution are then as follows:

Key	OBE	SSE
A	0.034	0.008
B	0.119	0.029

Notes: The key A gives the probabilities of exceedance assuming the population mean equals the sample mean. B gives the probabilities of exceedance assuming the population mean is at the lower end of the 95% confidence interval.

3.2.3   Discussion of Results

The test data generated in the Berkeley test program enables a reasonable estimate of the deflections or strains at which various levels of cracking

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could be expected in a masonry wall.

By taking the 95% confidence intervals on the population mean, the factor of safety associated with the allowable strain at 0.0008 for an OBE event is 2.15 < SF < 2.90. For an SSE event the corresponding range is 2.13 < SF < 2.61 based on an allowable strain of 0.0017. In terms of probability, it can be stated that code allowable strain will exceed the actual strain obtained from the tests 34 times in 1000 for OBE events and 8 times in 1000 for SSE events if the population mean strength is taken at the center of the 95% confidence intervals. If one considers the extreme case where the population mean is taken to be at the lower end of its 95% confidence interval, then these figures become 119 times in 1000 for OBE events and 29 times in 1000 for SSE events. Given the extreme nature of the assumption on which these second estimates are based and the self-limiting nature of the load, these probabilities of exceedance are deemed satisfactory.

3.3   Summary

From the above discussion of results it is concluded that the criteria specified increase factor of 1.67 for shear strain for factored loads is reasonable for the reevaluation of the masonry walls at Indian Point, Unit 2.

3.4   References

3.1   Mayes, R.L., Omote, Y., and Clough, R.W., "Cyclic Shear Tests of Masonry Piers, Volume 1 - Test Results." EERC Report No. 76-8, May, 1976.

3.2   Hidalgo, P.A., et al., "Cyclic Loading Tests of Masonry Single Piers, Volume 1 - Height to Width Ratio of 2.0," EERC Report No. 78-27, November 1978.

3.3   Chen, S.W., et al., "Cyclic Loading Tests of Masonry Single Piers, Volume 2 - Height to Width Ratio of 1," EERC Report No. 78-28, Dec., 1978.

3.4   Hidalgo, P.A., et al., "Cyclic Loading Tests of Masonry Single Piers, Volume 3 - Height to Width Ratio of 0.5," EERC Report No. 79-12, May, 1979.

3.5   Sveinsson, B.I., et al., "Evaluation of Seismic Design Provisions for Masonry in the United States." EERC Report 81-10, August 1981.
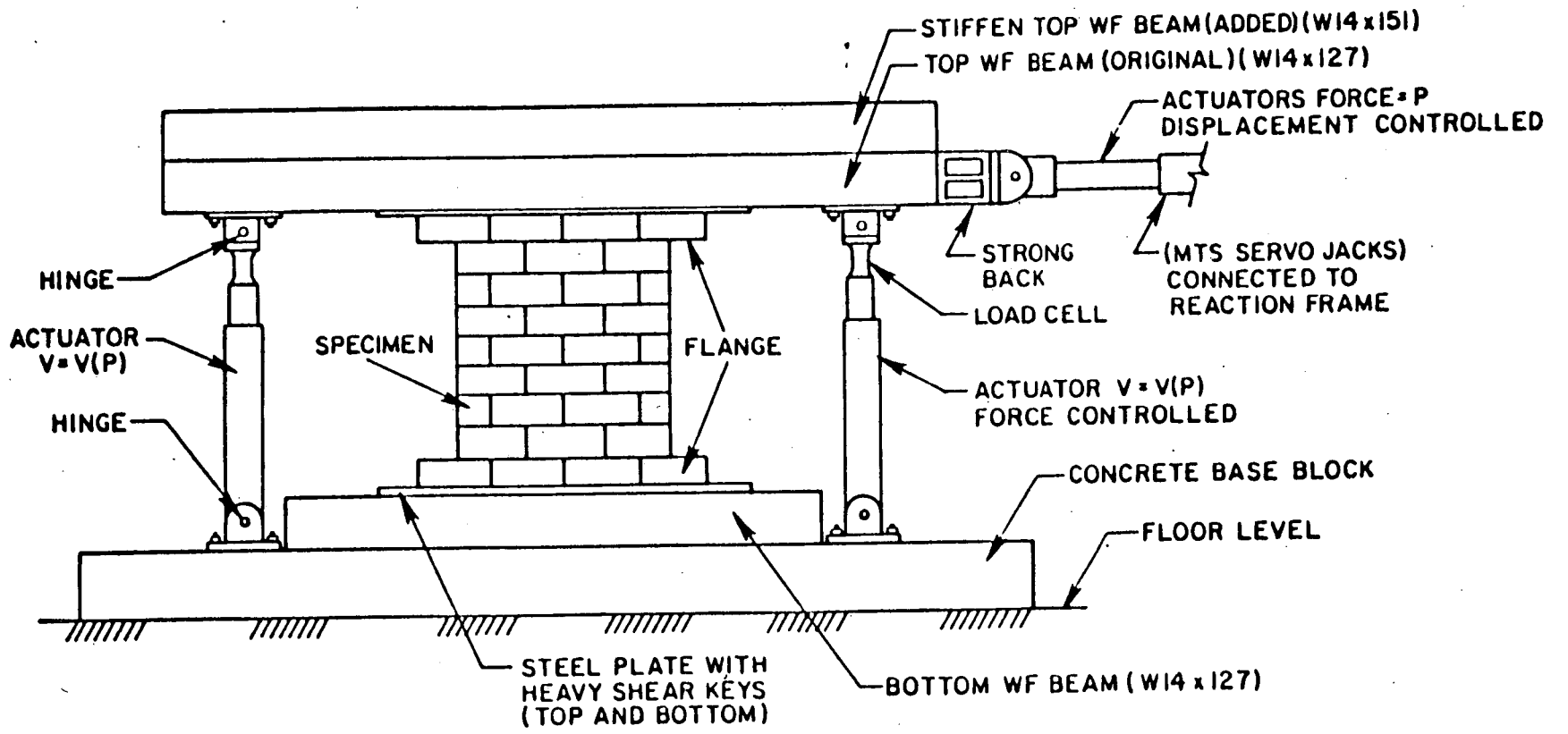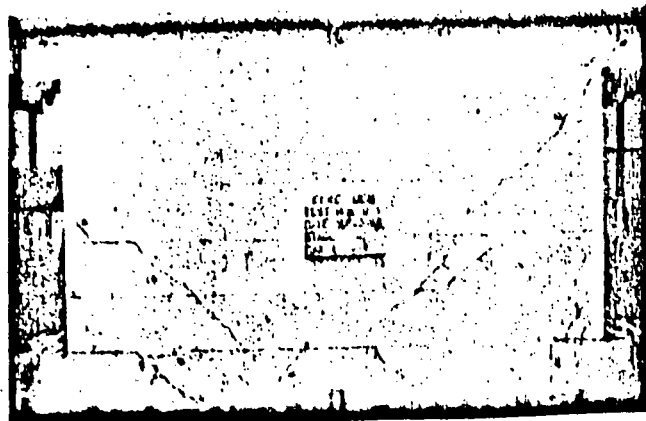
FIGURE 3-1   SCHEMATIC ILLUSTRATION OF SINGLE PIER TEST

Stage 10

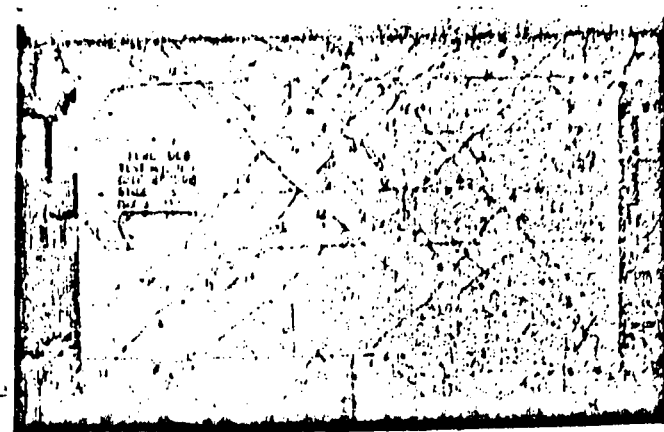Stage 12

Stage 16

Stage 18

FIGURE 3-2   SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS, TEST HCBL-12-3
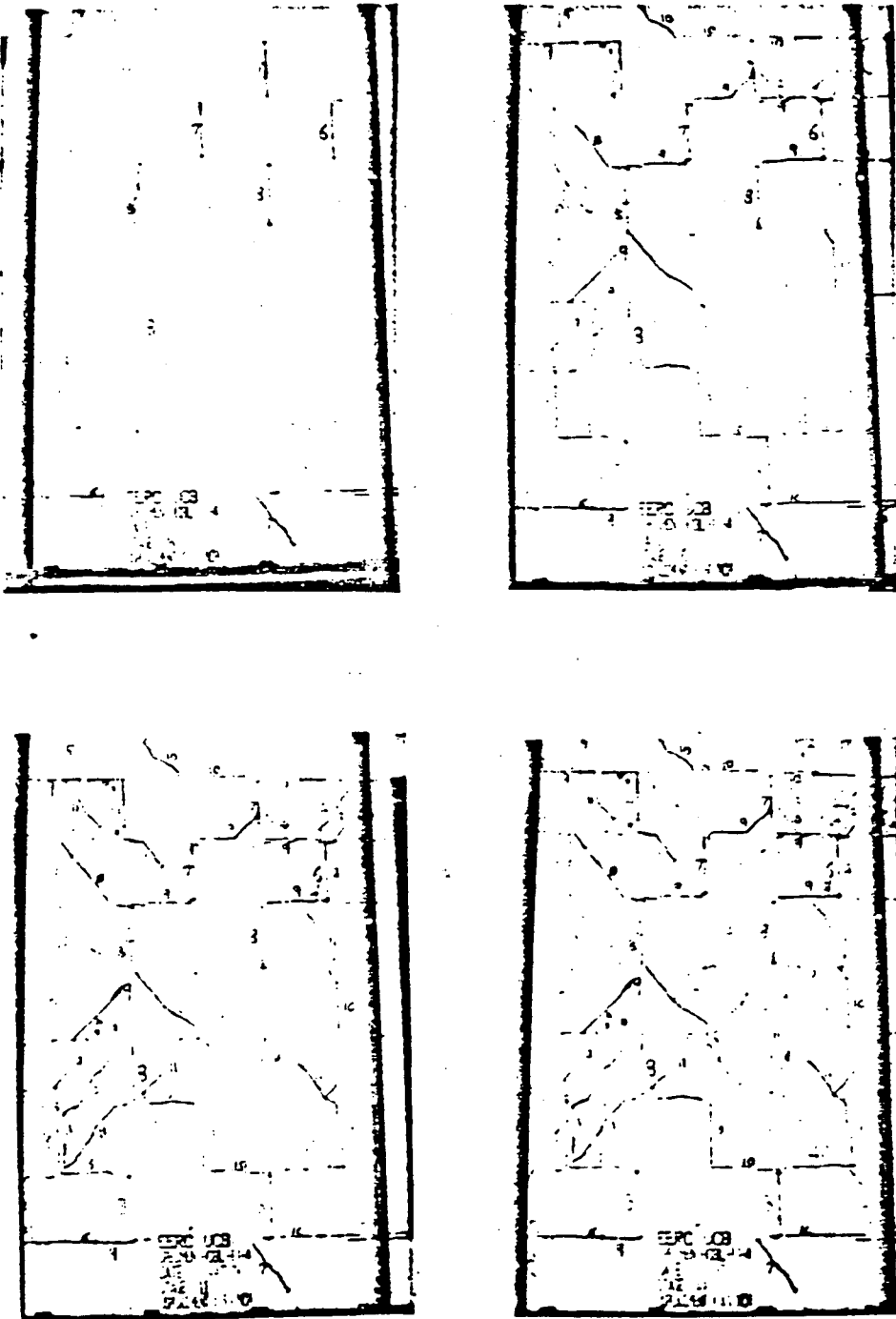
FIGURE 3-3   SUCCESSIVE CRACK FORMATION AND
EXPERIMENTAL RESULTS, TEST HCBL-11-4

- VE DIRECTION
- VE LATERAL DISPL. CORRES
SHEAR FORCE AND SHEAR STRESS
- VE LATERAL DISPL. CORRES.
SHEAR FORCE AND SHEAR STRESS
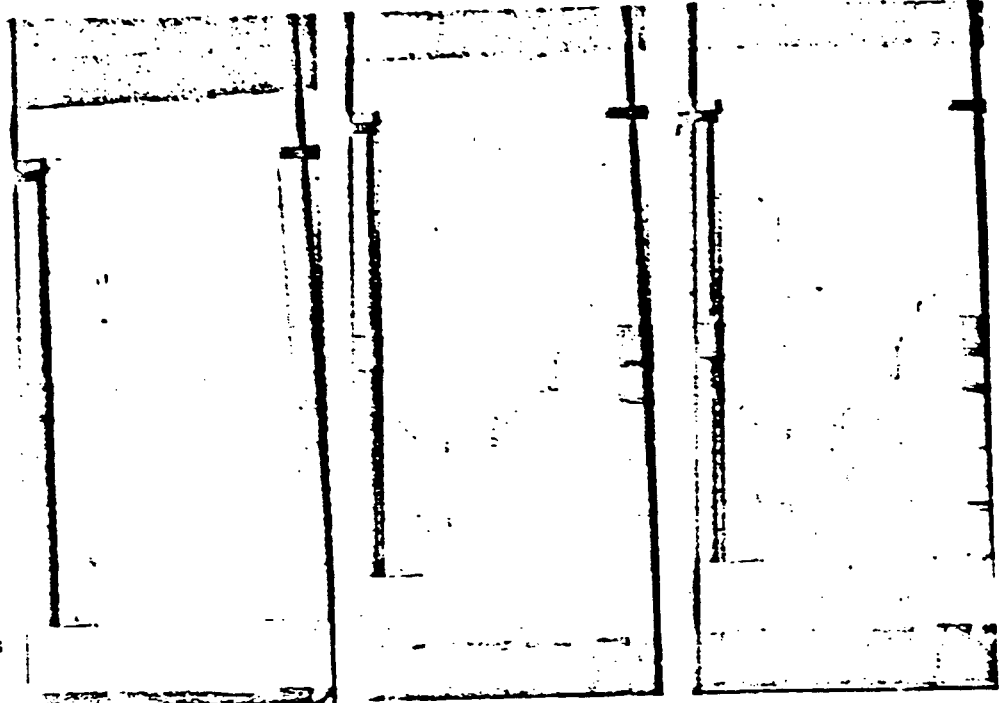
0.16"; 23.0 KIPS
120 PSI
0.13"; 20.1 KIPS
150 PSI

0.29"; 18.8 KIPS
98 PSI
0.18"; 21.7 KIPS
113 PSI

0.32"; 19.9 KIPS
104 PSI
0.24"; 19.1 KIPS
99 PSI
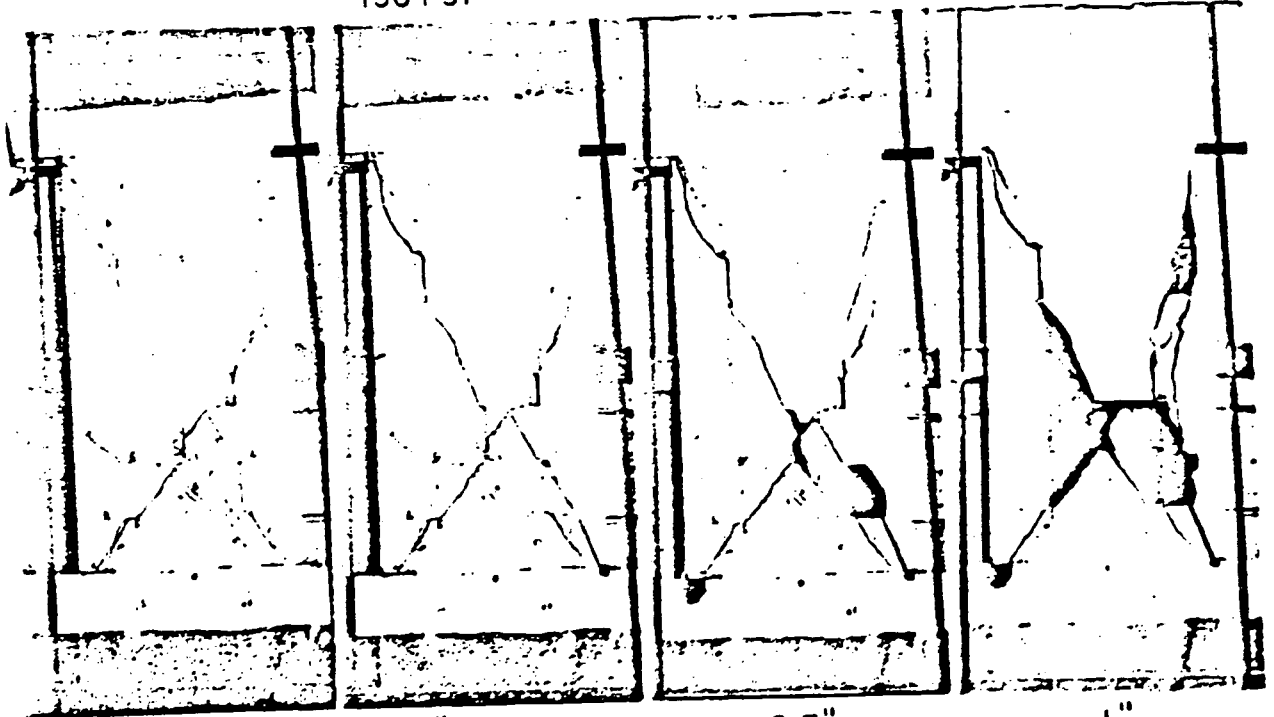
0.37"; 19.9 KIPS
104 PSI
0.28"; 19.7 KIPS
102 PSI

0.51"; 10.5 KIPS
55 PSI
0.41"; 13.6 KIPS
71 PSI

0.7"

0.7"

1"

1"

FIGURE 3-4   SUCCESSIVE CRACK FORMATION AND
EXPERIMENTAL RESULTS, TEST HCBL-21-6

FIGURE 3-5 DISTRIBUTIONS OF OBE AND SSE STRAIN DATA

4 ALLOWABLE SHEAR STRESSES

Two of the requests from the NRC review referred to the allowable in-plane shear, as follows:

Request No. 5:

> "The licensee should exercise caution in using the test results of Figure 2, Attachment 2 since some of the tests have insufficient data. The licensee should justify the applicability of Berkeley tests to the Indian Point, Unit 2 masonry structures".

In-plane shear stresses are further addressed in the list of questions:
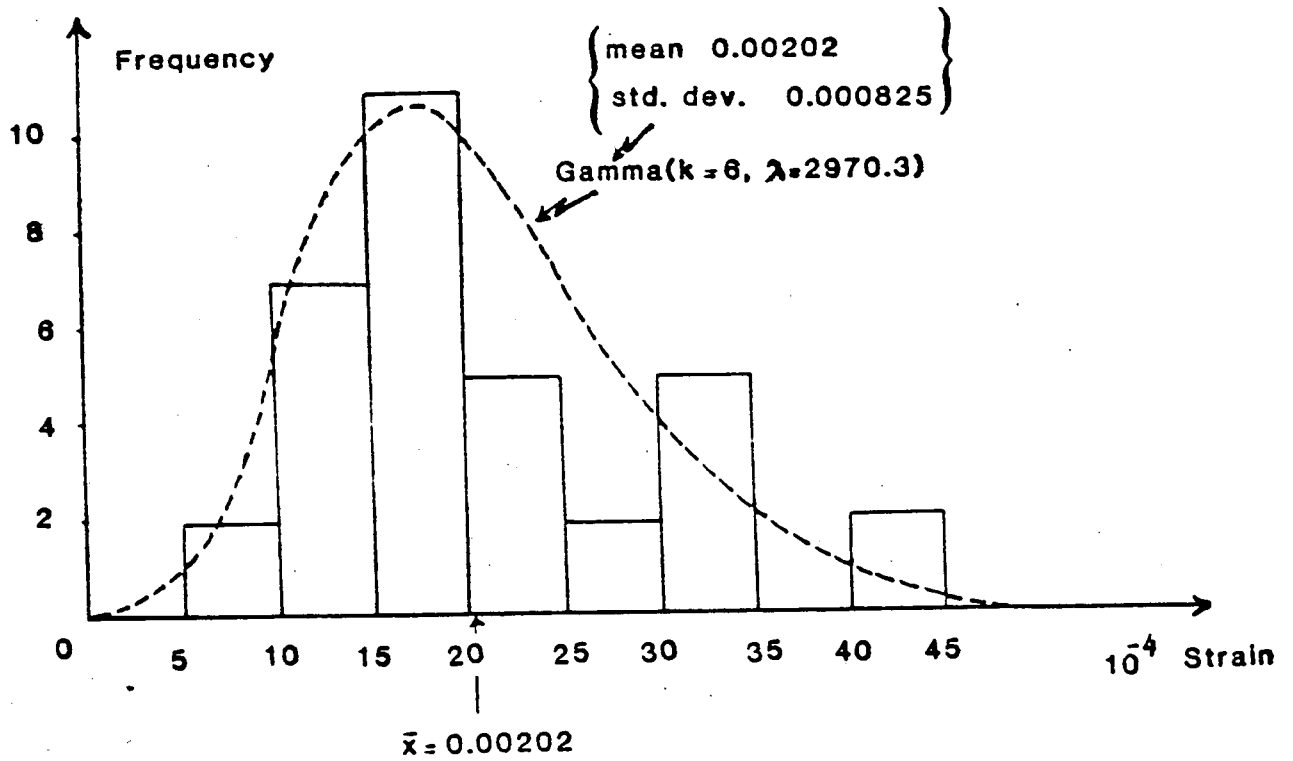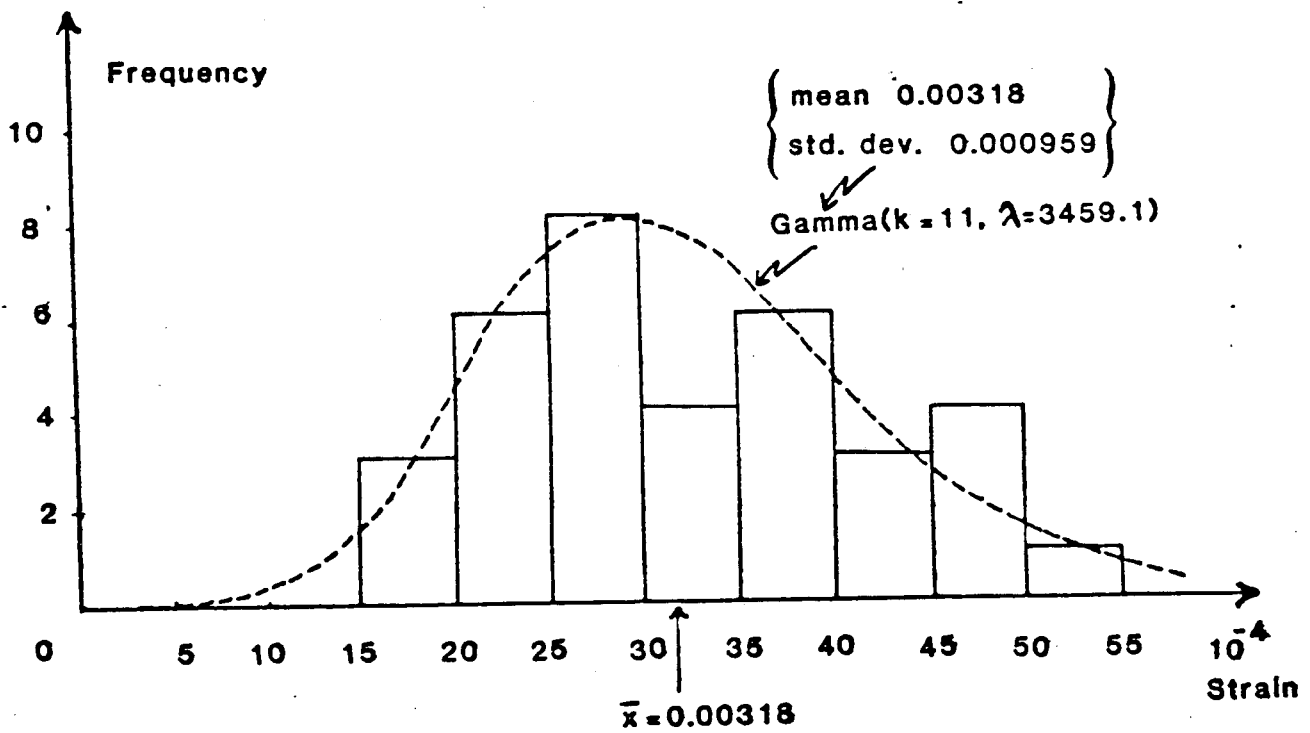
Request No. 11 (b)

> "The licensee suggests a 67 to 70% increase in the allowable shear for both masonry and reinforcement for factored loads. In the corresponding SEB criteria, increases for factored loads are 30% for the masonry and 50% for the reinforcement".

The masonry walls at Indian Point, Unit 2 are all unreinforced. Therefore reference to the particular Berkeley test results in Figure 2, Attachment 2 have been removed from the justification in the criteria.

For masonry taking the shear the criteria limit has been revised in accordance with the SEB criteria, i.e. a 30% increase for factored loads. The computed ratios of maximum shear stress to allowable shear stress in the wall evaluation have been modified to reflect this criteria change.

5 APPLICABILITY OF TEST RESULTS

The first part of NRC request number 11 states that:

> "The licensee should explain the applicability of several
> test results presented in Reference 2 to the masonry
> structures at Indian Point, Unit 2 with specific
> reference to the type of mortar, the actual boundary
> conditions, and the dynamic nature of the loading".

These test results are used in the following sections to justify the allowable stress factors for tension parallel to and normal to the bed joint for both concrete and brick masonry. In general those tests with similar mortar types to that used at Indian Point, Unit 2 were selected for statistical analysis. As the walls tested had simply supported boundaries they were similar in this respect to those at Indian Point, Unit 2 which also had pinned boundaries. In our opinion the results of static, monotonic tests are applicable in determing allowable tension stresses for the following reasons:

1. An unreinforced masonry wall responds elastically to seismic
    loads provided it is not cracked.

2. There are no test results available indicating that dynamic
    loading reduces the tensile strength normal to the bed
    joint. In fact the only test data available for any type of
    cyclic loading on masonry structural elements indicates
    that the in-plane shear strength of masonry shear walls
    tested pseudostatically is 8-23% less than that of a 3
    cps equivalent dynamic test (Reference 5.1).

3. Cyclic or shake table tests are essential to determine the
    post-cracked or inelastic performance of structural elements.
    However, they are not essential to determine the ultimate
    or cracking strength of structural elements.

4. Points 1, 2 and 3 above indicate that the uniform or point
    load tests are reasonable methods to determine the cracking
    or tensile strength of an unreinforced masonry wall
    subjected to out-of-plane loads.

5.1 Reference

5.1   Mayes, R.L., Omote, Y., and Clough, R.W., "Cyclic Shear Tests
        of Masonry Piers, Volume 1 : Test Results", EERC Report No.
        76-8, May, 1976.

6 TENSION PARALLEL TO BED JOINT

Request number 11 (a) from the NRC review was as follows:

> "For factored loads, the licensee suggests a 67%
> increase in allowable stresses for tension parallel
> to and perpendicular to the bed joint. However, the
> SEB criteria allow only 50% and 30% increases,
> respectively".

This section examines available test data on tension parallel to the bed joint
and in the following section experimental evidence for tension normal to the
bed joint is presented. Both sections consider the two masonry types at Indian
Point, Unit 2, i.e. concrete block and brick masonry.

6.1 Concrete Masonry

In the following sub-sections the results of available tests on concrete masonry
walls are examined and the factors of safety inherent on the allowable
stresses for factored loads extracted.

6.1.1 Test programs to date

References 6.1, 6.2 and 6.3 contain results of 28 tests on concrete
masonry walls tested in the horizontal span. These test results form
the basis of the evaluation of the flexural tension parallel to the bed
joint that follows below.

6.1.2 Evaluation of the Test Results

In Table 6.1, the major results of the tests are listed. The two ratios
listed in the table, Ratio 1 and Ratio 2, are formed by dividing an
ultimate factor from the tests by an allowable factor from the criteria.
The ultimate factor from the tests is calculated by dividing the square
root of the actual mortar strength of the test specimens into the
calculated modulus of rupture of the same. The two ratios are thus
the following:

$$\text{Ratio } 1 = (f_r / \sqrt{M_o}) / 1.0$$

$$\text{Ratio } 2 = (f_r / \sqrt{M_o}) / 1.67$$

where f_r is modulus of rupture calculated from tests

M_o is corresponding mortar cube strength

1.0 is from criteria equation $1.0\sqrt{M_o}$ for
     an OBE event

1.67 is from criteria equation 1.67 $\sqrt{M}_{o}$ for
an SSE event

The averages and ranges of the available test data are 6.40 with a range of 2.18 to 18.56 for the unfactored loads and 3.83 with a range of 1.31 to 11.11 for the factored loads.

6.1.3 Summary

The average factor of safety for tension parallel to the bed joint for factored loads is 3.83 using a stress increase factor of 1.67. This margin of safety for a total of 28 tests is satisfactory and justifies the use of the 1.67 stress increase factor.

6.2 Brick Masonry

6.2.1 Overview of Test Programs

The results of several test programs providing the tensile strength of mortar parallel to the bed joint in brick are evaluated in this report. The programs referenced in 6.4, which is a summary report, involved static, monotonic load tests.

Results for all 19 unreinforced test specimens were based on mortar type S, as specified by proportion in ASTM C270. The fact that only type S mortar was used in these tests is not considered critical because tension parallel to the bed joint in brick is not sensitive to the type of mortar used. All the tests were carried out using a uniform pressure (air bag) loading. This produces a parabolic moment distribution over the height of the wall with the maximum moment at center of the span. The results of the 19 tests performed in the test programs are considered applicable in determining the tensile strength parallel to the bed joints for seismic loads for the reasons outlined in Section 5 of this response.

6.2.2 Evaluation of Test Results

The results from several different monotonic test programs on the tensile strength of mortar parallel to the bed joint in brick form the basis of the statistical analysis presented in this section. In total, data from 19 tests were available, involving type S mortar.

6.2.2.1 Description of Statistical Analysis

Statistical analysis of the test results was carried out using a Gamma distribution. A full description of the details of this analysis, and the reasons for selecting this particular distribution, is given in Section 7 of this response. A summary of the results of this statistical analysis is given in the table below:

Results of Statistical Analysis of Brick Data	
Sample Size	· 19
Sample Mean (m)	391.1 psi.
Standard Deviation	95.6 psi
Coefficient of Variation	24.4%
95% Confidence Levels:	
On (m)	$345.0 < m < 437.2$ psi
On (m-s)	$249.4 < m-s < 341.6$ psi
On (m-2s)	$153.8 < m-2s < 246.0$ psi
On (m-3s)	$58.2 < m-3s < 150.4$ psi

These results were then compared with the allowable stresses specified in the criteria. i.e. 56 psi for the OBE loading and 94 psi for the SSE loading.

Probabilities that the criteria specified allowable stress would exceed the stress based on the test results were calculated under two assumptions: firstly, that the population mean was equal to the sample mean, and secondly, that it was at the lower end of the 95% confidence interval.

Finally, safety factors based on the 95% confidence interval for the mean were calculated.

These results are presented below:

Exceedance Probabilities and Safety Factors		
	OBE	SSE
Probabilities of Exceedance		
KEY A	0.000228	0.00094
KEY B	0.001252	0.00432
Range of Safety Factors on Mean	$6.16 < SF < 7.81$	$3.67 < SF < 4.65$

NOTE:

(1)   KEY A in the table above gives the probabilities of

exceedance assuming the population mean equals the sample mean.

(2)   KEY B gives the probabilities of exceedance assuming the population mean is at the lower end of the 95% confidence interval.

6.2.2   Discussion of Results

The key results for the confidence intervals are plotted in Figure 6-1, together with the OBE and SSE allowable stresses from the reevaluation criteria.   The confidence intervals for the data are relatively narrow (+/-11.8% for the mean). It is seen that the OBE allowable stress lies below the "mean minus three standard deviations" confidence interval whereas the SSE allowable stress lies within the "mean minus three standard deviations" confidence interval.

This implies that criteria specified allowable stresses will exceed the actual tensile strength of the mortar parallel to the bed joint in brick about 2 times in 10,000 for OBE events and about 9 times in 10,000 for SSE events if the population mean strength is taken at the center of the 95% confidence interval. If one considers the extreme case where the population mean is taken to be at the lower end of its 95% confidence interval, then these probabilities change to 13 times and 43 times in 10,000 for OBE and SSE events respectively. Based on the extreme nature of this second assumption these probabilities are deemed satisfactory.

Alternatively, instead of calculating probabilities of exceedence, one may take the same data and calculate factors of safety based on the mean. If this is done for the OBE events, using the full range of the 95% confidence interval for the population mean, the safety factor lies in the range of $6.16 < SF < 7.81$.   Similarly, for SSE events, the range is $3.67 < SF < 4.65$.

6.2.3   Summary

In view of the above discussion of results, it is concluded that the criteria specified increase factor of 1.67 for tensile stresses parallel to the bed joint for brick walls for factored loads is reasonable for the reevaluation of the Indian Point Generating Plant, Unit 2.

6.3   References

6.1   Hedstrom, R. O. "Load Tests of Patterned Concrete Masonry Walls", Proceedings, American Concrete Institute, Vol. 57, p 1265, 1961.

6.2   Fishburn, Cyrus C., "Effect of Mortar Properties on Strength of Masonry", Monograph 36, National Bureau of Standards, 1961.

6.3   Livingston, A. R., Mangotich, E., and Dikkers, R., "Flexural Strength of Hollow Unit Concrete Masonry Walls in the Horizontal Span", Technical Report No. 62, NCMA, 1958.

6.4   Brick Institute of America, "Recommended Practice for Engineered Brick Masonry", November, 1969.

6.5   Mayes, R.L., et al "Cyclic Shear Tests of Masonry Piers, Volume 1 : Test Results", EERC Report No. 76-8, May, 1976.

REFERENCE	RATIO 1 ULTIMATE OBE ALLOWABLE	RATIO 2 ULTIMATE SSE ALLOWABLE
6.1	2.64	1.58
6.1	2.82	1.69
6.1	2.64	1.58
6.1	3.52	2.11
6.1	3.60	2.16
6.1	3.10	1.86
6.1	3.32	1.99
6.1	4.02	2.41
6.1	4.22	2.53
6.1	4.08	2.44
6.2	3.24	1.94
6.2	2.18	1.31
6.2	3.52	2.11
6.2	3.46	2.07
6.2	3.97	2.38
6.2	5.20	3.11
6.3	16.32	9.77
6.3	18.56	11.11
6.3	11.99	7.18
6.3	9.39	5.62
6.3	10.60	6.35
6.3	10.60	6.35
6.3	9.39	5.62
6.3	8.72	5.22
6.3	10.02	6.00
6.3	6.35	3.80
6.3	5.68	3.40
6.3	5.99	3.59
Mean	6.40	3.83
Minimum	2.18	1.31
Maximum	18.56	11.11

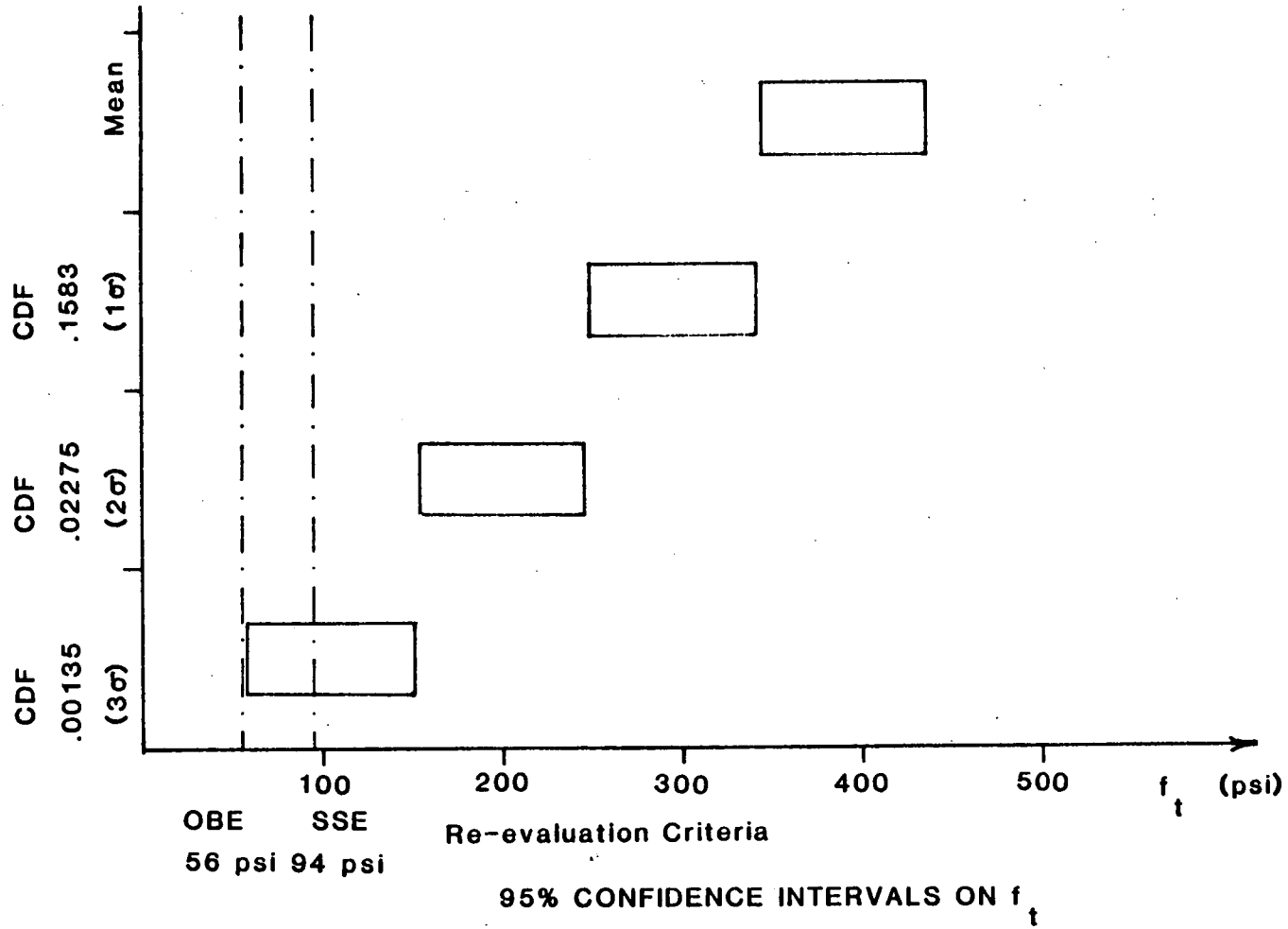TABLE 6.1 : FACTORS OF SAFETY FROM TESTS RESULTS

FIGURE 6-1 CONFIDENCE INTERVALS FOR POPULATION STATISTICS

7 TENSION NORMAL TO THE BED JOINT

This section is a continuation of the reply to NRC question 11 (a), reproduced at the beginning of Section 6 of this response. In the following sub-sections the maximum stress normal to the bed joint obtained from test results on both concrete and brick masonry is examined.

7.1 Concrete Masonry

This section provides the justification for using a stress increase of 1.67 for tension normal to the bed joints for factored loads for concrete masonry. The results of six different test programs providing the tensile strength of mortar normal to the bed joint are evaluated. All the test programs, given in References 7.1 through 7.6, involved static, monotonic load tests.

The test programs provided results for 81 unreinforced test specimens, involving four different mortar types, namely, M, S, N and O as specified by proportion in ASTM C270. Also varying between the six static test programs was the way in which the walls were loaded. Some tests were carried out using a uniform pressure (air bag) loading, some used concentrated center point loading, and others were performed with concentrated loads at the quarter points of the wall. The uniform load produces a parabolic moment distribution over the height of the wall, the central loading condition produces a symmetric triangular distribution with a maximum at midspan, and the quarter point loading produces a region of constant moment over half the height of the wall. In one series of experiments (7.3) the walls were tested after only 15 days of curing.

The results of the 81 tests performed in the six static test programs, in our opinion, are applicable in determining the tensile strength normal to the bed joints, for seismic loads, for the reasons outlined in Section 5 of this response.

7.1.1 Evaluation of Monotonic Test Results

The results from six different monotonic test programs (7.1 to 7.6) on the tensile strength of mortar normal to the bed joint form the basis of the statistical analysis presented in this section. In total, data from 81 tests were available, involving four different mortar types, namely types M, S, N and O. Only the results of tests with type N mortar, as specified by proportion in ASTM C270, are used herein as this was the mortar type specified for Indian Point, Unit 2. Tests reported in (7.1) and (7.6) contain no data for type N mortar, and thus have no further part in this study.

The following table indicates the large variability between the remaining tests on type N mortar. It was necessary to make the modifications indicated to the data from (7.3) and (7.5) in order that all section moduli were based on the net mortar bedded area.

Reference	No. of tests	Loading	Comments
7.2	8	uniform	Section Modulus based on mortar bedded area
7.3	14	1/4 point	Tensile strength based on gross area. Values are multiplied by 1.4 to compare to net area. Tests were carried out after only 15 days of curing.
7.4	3	1/4 point	Section modulus based on mortar bedded area (83 sq.in/ft)
7.5	18	center point	Tensile strength based on gross area. Values are multiplied by 1.14 to compare to net area.

Tensile strength normal to the bed joint is influenced by several variables, perhaps the single most important of which is the mortar cube strength. The 43 samples with type N mortar (8 uniform load, 17 quarter point load and 18 center point load) cover a wide range of cube strengths from 610 psi on the low end to 2500 psi on the high end. The effect of this variable is taken into consideration when evaluating the tensile strengths applicable at Indian Point, Unit 2.

In two separately reported studies the effect of the loading condition (quarter point loading versus uniform loading) on the apparent tensile strength was evaluated. The first study by Monk (7.8) produced a theoretical analysis indicating that quarter point load tests would give tensile strengths apparently lower than uniform load tests, the actual difference being a function of the coefficient of variation of the mortar strength. For typical values of this coefficient of variation for type N mortar, the analysis indicated that point load tests would give tensile strengths approximately 10% lower than would uniform load tests. The paper then went on to compare tensile strength results from the two different kinds of loading, for the case of brick walls, and found the experimental ratio between the mean strength from uniform load tests and the mean strength from quarter point load tests to be 1.97. Although the paper did not specifically recommend the adoption of this factor of 1.97 to relate quarter point load data to uniform load data, the factor seems to have been used blindly in the past for this purpose. The second study (7.9) again looked at experimental data, and came up with a factor of 1.99 for concrete masonry walls.

It is instructive at this point to examine the reason why quarter point

loading produces lower apparent tensile strengths than does uniform loading. In the case of a uniform load, a parabolic moment distribution results, subjecting one joint to the maximum moment. However, for quarter point loading, fully one half of the wall or typically some 6 to 7 joints are subjected to the same maximum moment. Given the inherent variability in mortar strength, failure will occur in this case at the weakest joint of the 6 or 7, which may be well away from the center of the span. The same joint, under a parabolic moment distribution, may not fail since it would be subjected to lesser moment. Thus the apparent strength of the mortar normal to the bed joint will, on the average, be lower for the quarter point load cases. Strengths from uniform load tests and center point load tests would be expected to be similar, since in both cases only one joint is subjected to the maximum moment.

While references (7.8) and (7.9) have indicated use of "correlation factors" of 1.97 and 1.99 respectively to relate quarter point load data to uniform load data, we believe the actual factor should be closer to the 1.10 theoretically predicted in (7.8). We feel that other influences, such as mortar cube strength, air content, and friction between mortar and block, contributed to the large difference between data from the two load conditions reported in (7.8) and (7.9). For this reason in the analysis that follows, we have scaled the strengths from the quarter point tests by 1.10 to relate their values to results from the other tests, which we believe result in moment distributions more closely representing that in a real wall in a structure subjected to seismic loading.

The data from references 7.3, 7.4 and 7.5 have cube strengths reported corresponding to each tensile strength. The data from Reference 7.2, on the other hand, gives only the average cube strength from the 8 walls tested. For this reason, the data from Reference 7.2 has been analysed separately.

7.1.2 Description of Statistical Analyses

Statistical analyses were carried out for two cases: uniform load data and point load data. The reason for this separation is indicated in the preceding section. The tensile strength data from references 7.3 and 7.4 were first scaled by 1.10 to account for the lower average tensile strengths expected from quarter point tests. A plot of the tensile strength normal to the bed joint against the corresponding mortar cube strength was then made for all the data from references 7.3, 7.4 and 7.5. This plot is shown in Figure 7.1. Two least squares fits to this data were then made. The first was of the form

$$Y = k \ X^n$$

and the second was of the form

$$Y = k \ \sqrt{X}$$

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where

Y = tensile strength, normal to bed joint
X = mortar cube strength

The resulting curves are also plotted in Figure 7.1. The comparison indicates that the expressions found in codes with the tensile strength as a function of the square root of the cube strength are reasonable and in close agreement with the optimum fit when the exponent is not constrained to a value of 0.5. In view of the closeness of the two curves, the scatter of the data and the ACI-531 code use of functions involving the square root of the cube strength, the second curve will be used herein. Accepting this relationship between tensile bond strength and mortar cube strength, all data can then be normalized by dividing the test tensile strength by the square root of the corresponding mortar cube strength. This gives 35 normalized samples for the point load cases and 8 normalized samples for the uniform load cases. For each group, the following parameters were computed:

(i)   Sample mean, $\bar{X}$

(ii)   Sample standard deviation (minimum mean square estimator), s

These statistics were then used as the parameters for the distribution of the population. For each of the two cases (uniform load data and point load data) two underlying distributions were assumed, and the effect of the choice of distribution on the results was examined. The more reasonable distribution was then accepted. The two underlying distributions were the normal distribution and the gamma distribution. The 95% confidence interval for the mean of the population m was calculated, assuming that the normalized variable:

$$\frac{\bar{X} - m}{s / \sqrt{n}}$$

is t-distributed, and that the actual population standard deviation, σ, is unknown. Here n is the sample size.

For the case of the underlying distribution being normal, confidence intervals on the parameters $m-1\sigma$, $m-2\sigma$ and $m-3\sigma$ were estimated from the confidence interval on the mean and the sample standard deviation. For the case of the underlying distribution being gamma, a different approach was taken. $m-1\sigma$ corresponds to a value of the cumulative distribution function equal to 0.1587 for the normal distribution. This means that a little under 16% of the area under the probability density curve lies to the left of $m-1\sigma$. Similarly, $m-2\sigma$ and $m-3\sigma$ correspond to values of 0.02275 and 0.00135 on the cumulative distribution function respectively. Based on the confidence interval for the mean, confidence intervals were calculated for values of the gamma distribution for which its cumulative

distribution function had values of 0.1587, 0.02275 and 0.00135 respectively.

These actual distributions were then compared with the criteria specified allowable tensile stress normal to the bed joint, i.e., $0.5\sqrt{m_o}$ for the OBE condition, and 1.67 times that value for the SSE condition. Probabilities that the criteria specified allowable stress would exceed the joint strength based on the test results were calculated under two assumptions: firstly, that the population mean was equal to the sample mean, and secondly, that it was at the lower end of the 95% confidence interval. These conditions are termed A and B respectively in the table in Section 7.1.3.6.

Finally safety factors based on the 95% confidence interval for the mean were calculated.

7.1.3 Results of Statistical Analyses

7.1.3.1 Sample Statistics

In the table below, the test tensile strengths normal to the bed joint have been normalized by dividing each strength by the square root of the corresponding mortar cube strength.

	Normalized Uniform Load Data	Normalized Point Load Data
Sample Size	8	35
Sample Mean	1.1850	0.9621
Sample Standard Deviation	0.2778	0.3501
Coefficient of Variation	23%	36%

7.1.3.2 Confidence Intervals on the Population Mean

The normalized variables analyzed in Section 7.1.3.1 are compared to the factors multiplying the square root of the mortar strength, m_o, in the criteria,

$$\sigma = c\sqrt{m_o}$$

The following confidence intervals result:

Uniform Load Data $0.9532 < C < 1.4168$
Point Load Data $0.8420 < C < 1.0822$

7.1.3.3 Discussion of Normal vs Gamma Distribution

The normal distribution is well known, and requires no discussion other than the fact that it is a symmetric distribution with possible values in the range $(-\infty, \infty)$ We are concerned in this study with data that can only assume positive values (tensile strength), and this is a possible problem with using the normal distribution. For the case of the normal distribution fitted to the point load, normalized test data, approximately 1% of the total area under the probability density curve lies in the range of negative values. This will lead to erroneously high probabilities of exceedence. The Gamma distribution on the other hand, cannot assume negative values, and its shape may be adjusted by varying the parameters k and λ.

$$f_x (x) = \frac{\lambda (\lambda x)^{k-1} e^{-\lambda x}}{(k-1)!} \qquad x \geqslant 0$$

The distribution has a mean value of k/λ and a coefficient of variation of $1/\sqrt{k}$. Thus the value of k is adjusted to give the coefficient of variation observed from the sample, and then λ is calculated to give the correct mean value. The following values of k and λ arise:

Case	k	λ
Uniform Load Data	18	15.190
Point Load Data	7	7.276

For large k, the gamma distribution and the normal distribution are reasonably close. Thus the normal distribution is used for the uniform load data, and the gamma distribution is used for the point load data. The histogram of the normalized point load data is shown in Figure 7.2, and the gamma distribution which best fits the data is also shown.

It should be noted that there is no physical reason why tensile strengths normal to the bed joint should have any particular distribution. However, the gamma distribution can assume a wide variety of shapes by varying the parameters k and λ. We have chosen the gamma distribution for the point load data because it describes the test data far more accurately than does the normal distribution. It should also be noted that any distribution fitted to a relatively small sample size (35 in this case) can be expected to differ somewhat from the histogram of test data.

7.1.3.4 95% Confidence Intervals

(i) at cumulative distribution function = 0.1587
(1σ level)

Uniform Load Data	0.6754	$\lessgtr X \lessgtr$ 1.1390
Point Load Data	0.508	$\lessgtr X \lessgtr$ 0.746

(ii) at cumulative distribution function = 0.02275
(2σ level)

Uniform Load Data	0.3976	$\lessgtr X \lessgtr$ 0.8612
Point Load Data	0.302	$\lessgtr X \lessgtr$ 0.511

(iii) at cumulative distribution function = 0.00135
(3σ level)

Uniform Load Data	0.1198	$\lessgtr X \lessgtr$ 0.5834
Point Load Data	0.164	$\lessgtr X \lessgtr$ 0.334

These intervals are displayed graphically in Figure 7.3.

7.1.3.5 Safety Factors Based on the Mean

The reevaluation criteria specifies that the cube strength for type N mortar shall be limited to m_o = 750 psi. This leads to "allowable" tensile stresses normal to the bed joint ($1/2 \sqrt{m_o}$) of 13.69 psi for the OBE condition and 22.73 psi for the SSE condition. These are lower than the maximum limiting values given in the criteria and thus the factors 0.5 and 0.83 may be evaluated directly by determining factors of safety for them.

However, acknowledging that the specified design mortar strength of 750 psi is only an absolute minimum value and that the strength increases with age, the actual in situ mortar strength is assumed greater. Past experience bears this out. Thus the factors of safety have also been calculated for an assumed mortar strength of 1800 psi.

Using the above "allowable" values for the OBE and SSE conditions, and the 95% confidence interval for the mean strength from the tests, scaled to cube strengths of 750 psi and 1800 psi, the limits shown in the following table arise for the safety factor based on the mean:

	OBE	SSE
m_O = 750 psi:		
Uniform Load Data	1.91 < SF < 2.83	1.15 < SF < 1.71
Point Load Data	1.68 < SF < 2.16	1.01 < SF < 1.30
m_O = 1800 psi:		
Uniform Load Data	2.95 < SF < 4.39	1.78 < SF < 2.64
Point Load Data	2.61 < SF < 3.35	1.57 < SF < 2.02

7.1.3.6 Probabilities of Exceedence

The probabilities that the code specified allowable stress will exceed the available strength based the test results and both the design specified minimum mortar strength of 750 psi and an assumed actual mortar strength of 1800 psi are as shown below:.

		m_O = 750 psi		m_O = 1800 psi	
CASE	KEY	OBE	SSE	OBE	SSE
Uniform Load	A	0.0068	0.1066	0.0010	0.0097
	B	0.0277	0.3554	0.0011	0.0427
Point Load	A	0.0683	0.3931	0.0082	0.0912
	B	0.1644	0.5433	0.0367	0.2009

NOTE:

(1)   Key A gives the probability of exceedance assuming the population mean equals the sample mean.

(2)   Key B gives the probability of exceedance assuming the population mean is at the lower end of the 95% confidence interval.

7.1.4 Discussion of Results

The key results for the confidence intervals are plotted in Figures 7.3 and 7.4 together with the OBE and SSE stresses from the re-evaluation

criteria. The width of the confidence intervals is greater for the uniform load data reflecting the smaller sample size for this data. It is seen that when accepting the mortar strength to be 750 psi, the absolute minimum specified for the design, the OBE stresses lie within the "mean minus two standard deviations" confidence intervals from both uniform load data and point load data. For the same case the SSE stresses within the "mean minus one standard deviation" confidence interval from the uniform load data but between the "mean" and "mean minus one standard deviation" confidence interval from the point load data.

When considering the assumed actual mortar strength of 1800 psi the OBE stresses lie within the "mean minus three standard deviations" confidence intervals from the uniform load data and within the "mean minus two standard deviations" confidence intervals from the point load data. The SSE stresses on the other hand lie within the "mean minus two standard deviations" confidence interval and the "mean minus one standard deviation" confidence interval for the uniform load data and the point load data respectively.

Were the actual mortar strength equal to 750 psi it can be stated that the criteria specified allowable stresses will exceed the actual tensile strength of the mortar normal to the bed joint between 7 and 68 times in 1000 for OBE events and between 107 and 393 times in 1000 for SSE events if the population mean strength is taken at the center of the 95% confidence interval. If one considers the extreme case where the population mean is taken to be at the lower end of its 95% confidence interval, then these figures become between 68 and 164 in 1000 for OBE events and 36 to 54 times in 100 for SSE events.

Were the actual mortar strength, on the other hand, equal to 1800 psi it can be stated that the criteria specified allowable stresses will exceed the actual tensile strength of the mortar normal to the bed joint between 1 and 8 times in 1000 for OBE events and between 10 and 91 times in 1000 for SSE events if the population mean strength is taken at the center of the 95% confidence interval. If one considers the extreme case where the population mean is taken to be at the lower end of its 95% confidence interval, then these figures become between 11 and 37 in 1000 for OBE events and 42 to 200 times in 1000 for SSE events.

Given the extreme nature of the assumptions that the lower end of the 95% interval is the population mean and that the actual in situ mortar strength is only 750 psi, these probabilities of exceedance are deemed satisfactory.

Alternatively, instead of calculating probabilities of exceedance, one may take the same data, and calculate factors of safety based on the mean. If this is done for the OBE events using the full range of the 95% confidence interval for the population mean and the mortar strength range of 750 psi to 1800 psi, and taking the extremes from uniform and point load data, the safety factor lies in the range $1.68 < SF <$

4.39. Similarly, for the SSE events, the range is $1.01 < SF < 2.64$.

The point load data must be viewed as a lower bound on the safety factor and an upper bound on the probability of exceedence, since 14 of its 35 sample points are from tests carried out on test specimens cured for only 15 days. We have not attempted to disguise this data, but its effect on the results (lower and upper bounds as discussed above) must be realized.

7.1.5 Summary

The purpose of this analysis was to provide a justification for the use of a stress increase factor of 1.67 for tension normal to the bed joint for factored loads. The values specified in the criteria are $0.5 \sqrt{m_o}$ for an OBE event and $0.83 \sqrt{m_o}$ for an SSE event. The criteria furthermore limits the strength of the mortar thus used to 750 psi, despite previous experience indicating an actual in situ mortar strength of up to and over 1800 psi for type N mortar. Therefore, in reality, the allowable tensile stresses normal to the bed joint are limitied to 13.7 psi for an OBE event and 22.7 psi for an SSE event. In view of the statistical analysis presented herein, this cut-off at 750 psi for the mortar strength is reasonable.

The range of the factors of safety, based on the test data and scaled to a cube strength of 1800 psi, is 2.61 to 4.39 for an OBE event and 1.57 to 2.64 for an SSE event. These factors of safety are based on the 95% confidence intervals of the mean strength of the test data. It is therefore concluded that using a stress increase factor of 1.67 for the tension normal to the bed joint, resulting in allowable stresses of 13.7 psi and 22.7 psi for OBE and SSE events respectively, is reasonable to use in the reevaluation criteria for the Indian Point Unit 2.

7.2 Brick Masonry

7.2.1 Overview of Test Programs

The results of several test programs regarding the tensile strength of mortar normal to the bed joint in brick are evaluated in this report. The programs referenced in (7.10, 7.11, 7.12, 7.13 and 7.14) involved static, monotonic load tests.

In total results for 86 unreinforced test specimens involving three different mortar types, namely M, S and N as specified by proportion in ASTM C270. All the tests were carried out using a uniform pressure (air bag) loading. This produces a parabolic moment distribution over the height of the wall with the maximum moment at center of the span.

7.2.2 Evaluation of Test Results

The results from several different monotonic test programs on the tensile strength of mortar normal to the bed joint in brick form the basis of the statistical analysis presented in this section. In total, data from 86 tests were available, involving three different mortar types, namely types M, S and N. Only the results of tests with type N mortar, as specified by proportion in ASTM C270, are used herein as this was the mortar type specified for the Indian Point Generating Plant, Unit 2. Tests reported in (7.12), (7.13) and (7.14) contain no data for type N mortar, and thus have no further part in this study.

The remaining tests, 11 in all, were evaluated statistically using the Gamma distribution as for the concrete masonry data in the preceding section. The results from this analysis are presented in the table below:.

Results of Statistical Analysis of Brick Data	
Sample Size	11
Sample Mean (m)	88.25 psi.
Standard Deviation	15.76 psi
Coefficient of Variation	17.9%
95% Confidence Levels:	
On (m)	$77.66 < m < 98.84$ psi
On (m-s)	$61.90 < m-s < 83.08$ psi
On (m-2s)	$46.14 < m-2s < 67.32$ psi
On (m-3s)	$30.38 < m-3s < 51.56$ psi

The results were then compared with the criteria specified allowable stresses normal to the bed joint of 28 psi and 47 psi for OBE and SSE loadings respectively. OBE and SSE events:

Probabilities that the criteria specified allowable stress would exceed the stress based on the test results were calculated under two assumptions: firstly, that the population mean was equal to the sample mean, and secondly, that it was at the lower end of the 95% confidence interval.

Finally, safety factors based on the 95% confidence interval for the mean were calculated for the shear stresses.

These results are as follows:

Exceedance Probabilities and Safety Factors		
	OBE	SSE
Probabilities of Exceedance		
KEY A	0.000066	0.0044
KEY B	0.000814	0.0259
Range of Safety Factors on Mean	2.77 < SF < 3.53	1.65 < SF < 2.10

NOTE:

(1)   KEY A in the table above gives the probabilities of exceedance assuming the population mean equals the sample mean.

(2)   KEY B gives the probabilities of exceedance assuming the population mean is at the lower end of the 95% confidence interval.

7.2.3   Discussion of Results

The key results for the confidence intervals are plotted in Figure 7-5, together with the OBE and SSE allowable stresses from the reevaluation criteria.   The confidence intervals for the data are relatively narrow although the sample size was small.   It is seen that the OBE allowable stress lies well below the "mean minus three standard deviations" confidence interval whereas the SSE allowable stress lies within the "mean minus three standard deviations" confidence interval.

It can be stated that criteria specified allowable stresses will exceed the actual tensile strength of the mortar normal to the bed joint in brick about 66 times in 1,000,000 for OBE events and about 44 times in 10,000 for SSE events if the population mean strength is taken at the center of the 95% confidence interval.   If one considers the extreme case where the population mean is taken to be at the lower end of its 95% confidence interval, then these probabilities change to 8 times and 259 times in 10,000 for OBE and SSE events respectively. Based on the extreme nature of this second assumption these probabilities are deemed very satisfactory.

Alternatively, instead of calculating probabilities of exceedence, one may take the same data and calculate factors of safety based on the mean. If this is done for the OBE events, using the full range of the 95% confidence interval for the population mean, the safety factor lies in the range of 2.77 < SF < 3.53.   Similarly, for SSE events, the range is 1.65 < SF < 2.10.

7.2.4 Summary

In view of the above discussion of results, it is concluded that the criteria specified increase factor of 1.67 for tensile stresses normal to the bed joint for brick walls for factored loads is reasonable for the reevaluation of the Indian Point Generating Plant, Unit 2.

7.3 References

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7.7 Mayes, R.L., Omote, Y., and Clough, R.W., "Cyclic Shear Tests of Masonry Piers, Volume 1 : Test Results", EERC Report No. 76-8, May, 1976.

7.8 Monk, C.B., "Transverse Strength of Masonry Walls", Special Publication No. 166, American Society for Testing and Materials, 1954.

7.9 "Research Data and Comments in Support of: Recommended Building Code Requirements for Engineered Concrete Masonry", National Concrete masonry Association.

7.10 Monk, C.B. "Testing High-Bond Clay Masonry Assemblages", ASTM Special Technical Publication No. 320, 1962.

7.11 Brick Institute of America, "Recommended Practice for Engineered Brick Masonry", November, 1969.

7.12 Structural Clay Products Research Foundation, Research Report No. 8, "Compressive and Transverse Tests of Five-inch Brick Walls", May, 1965.

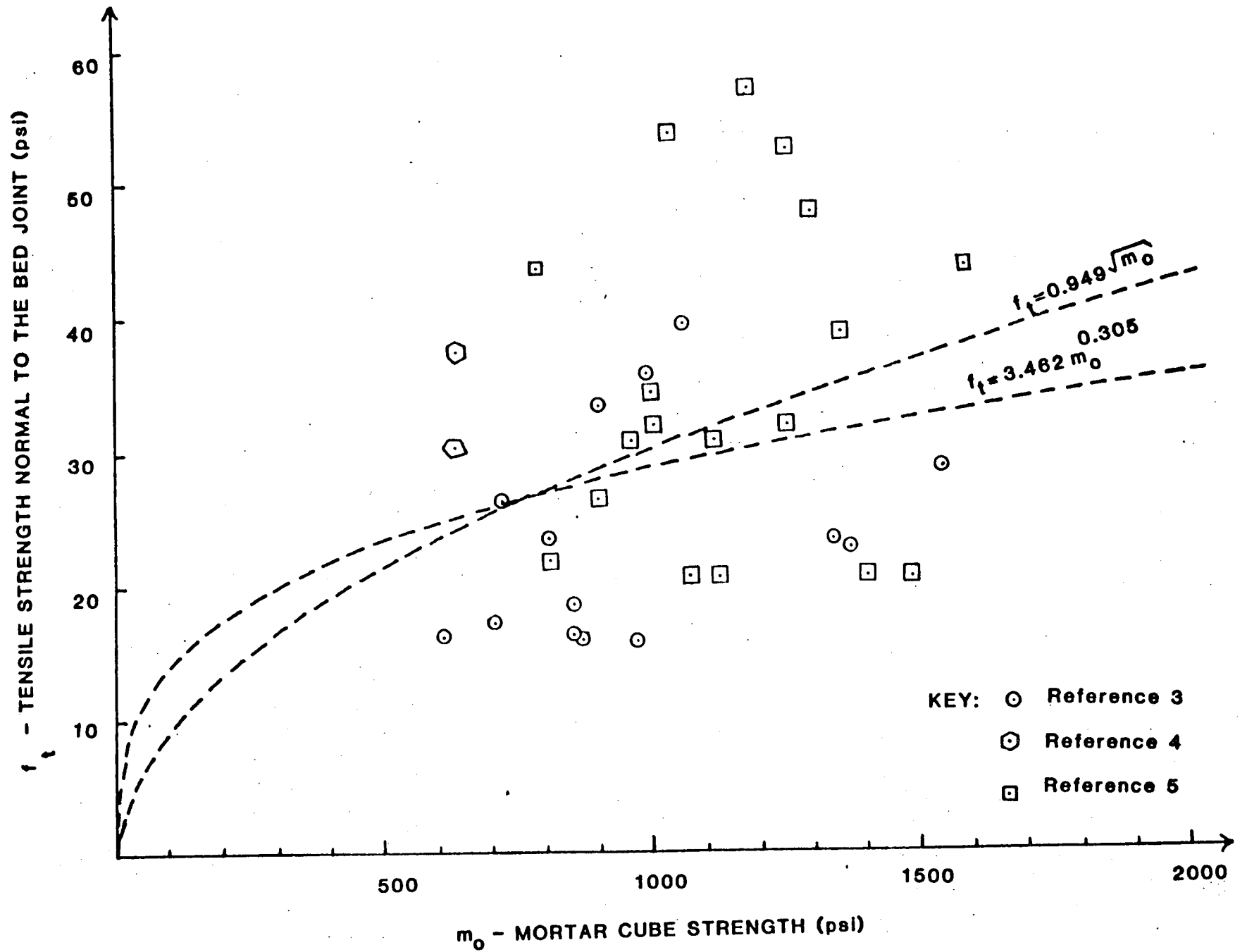7.13 Structural Clay Products Research Foundation, Research Report No. 9, "Compressive, Transverse and Racking Strength Tests of Four-inch Brick Walls", August, 1965.

7.14 Structural Clay Products Research Foundation, Research Report No. 10, "Compressive and Transverse Tests of Eight-inch Brick Walls", October, 1966.

FIGURE 7-1 TENSILE STRENGTH VS MORTAR CUBE STRENGTH
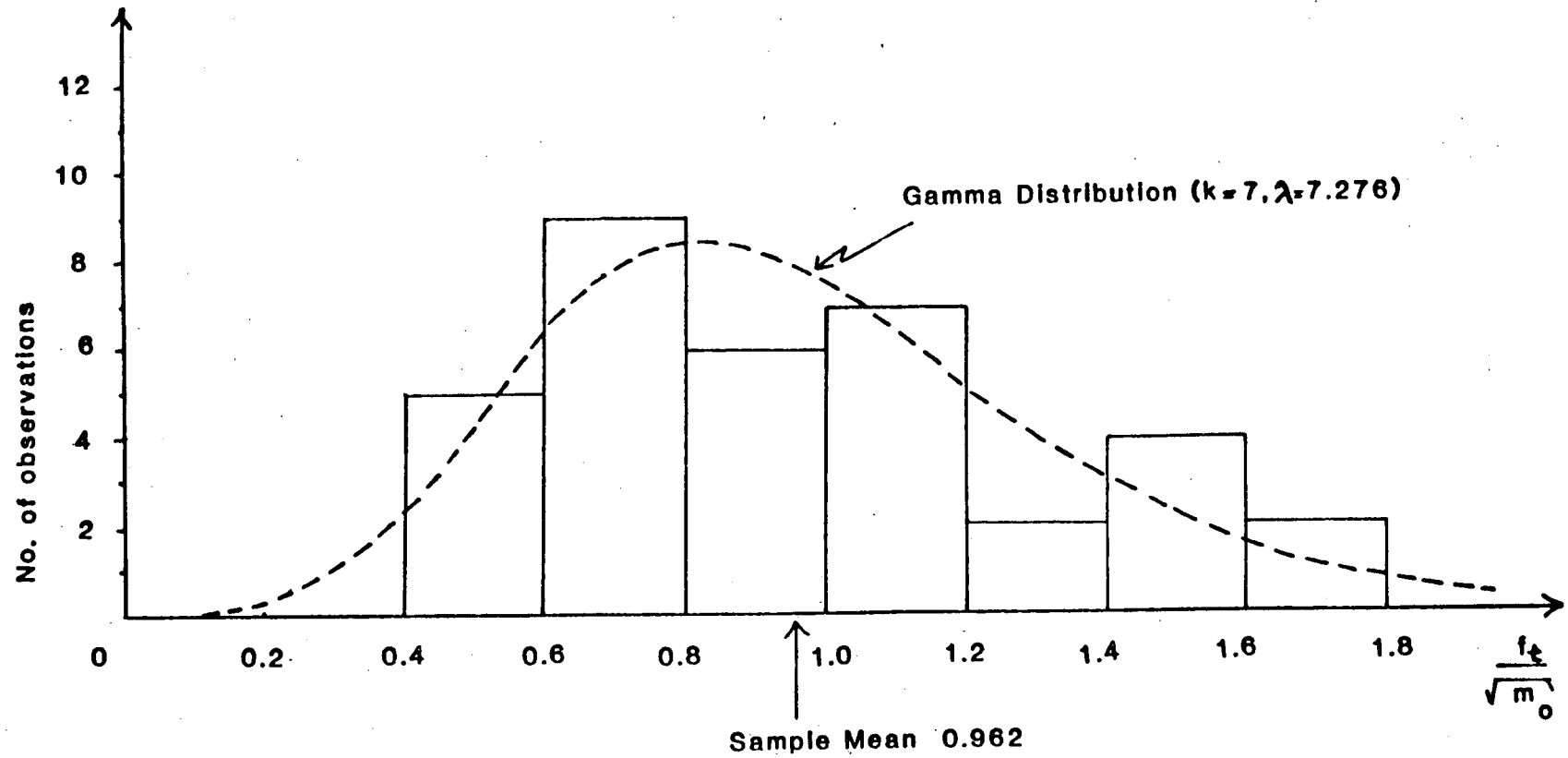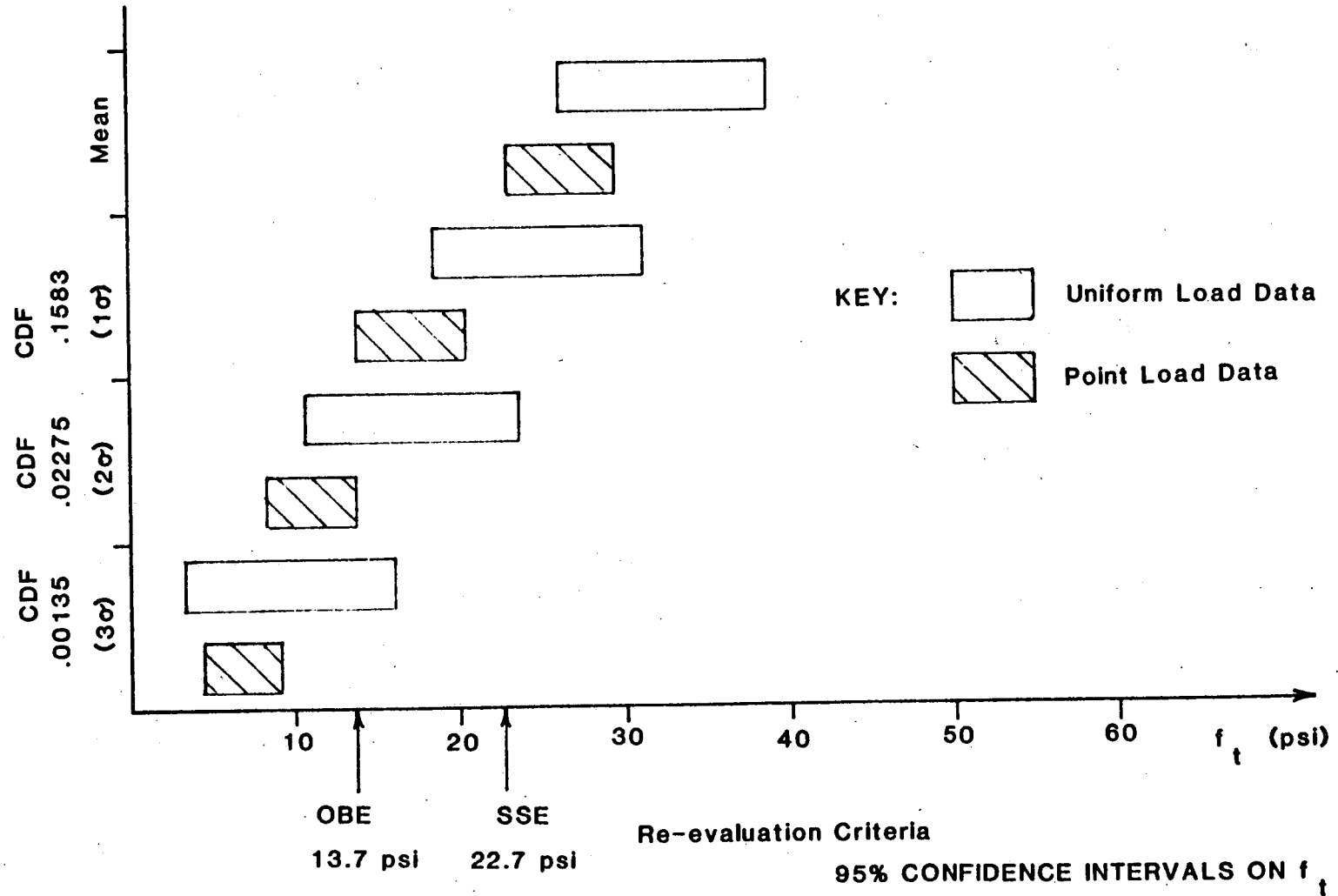
FIGURE 7-2   NORMALIZED POINT LOAD DATA

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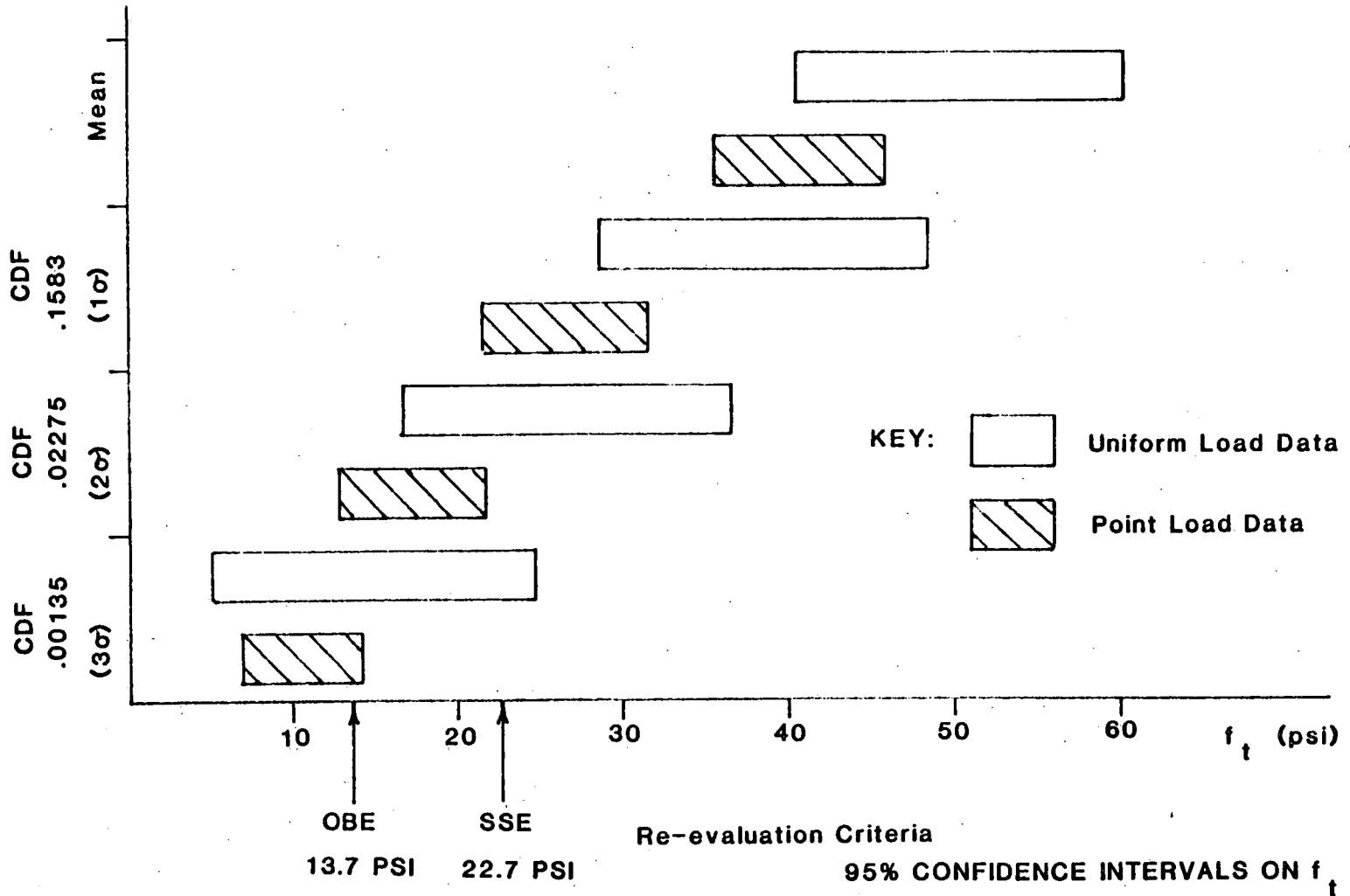FIGURE 7-3 CONFIDENCE INTERVALS FOR POPULATION STATISTICS Mo = 750 PSI

FIGURE 7-4 CONFIDENCE INTERVALS FOR POPULATION STATISTICS, Mo = 1800 PSI

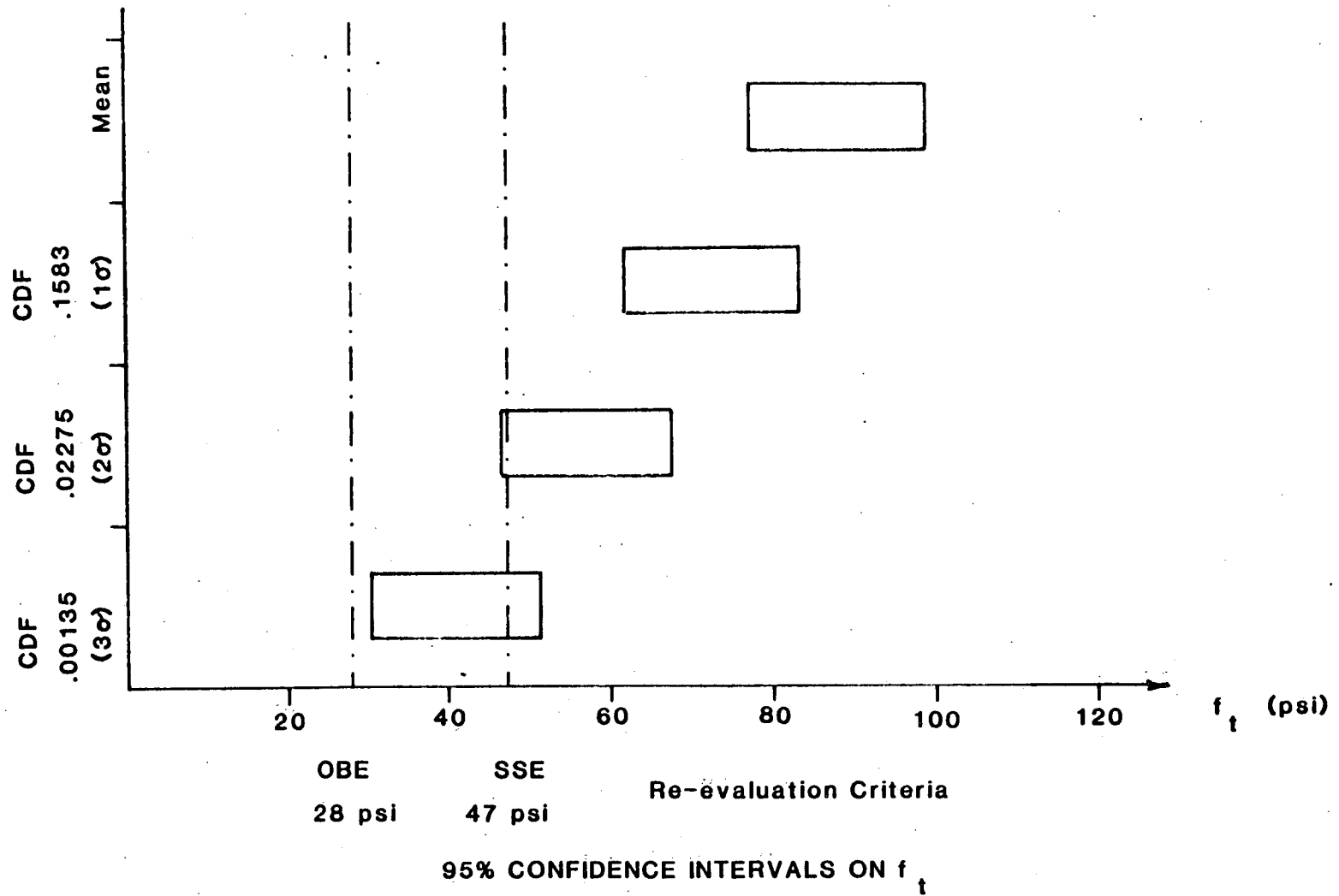FIGURE   7-5   CONFIDENCE INTERVALS FOR POPULATION STATISTICS

8 SPECIAL INSPECTION STRESS CATEGORY

Item 10 of the NRC review required that:

> "The licensee should justify using allowable stresses
> applicable to the Special Inspection category and
> indicate whether quality assurance/quality control
> information is available to support the categorization."

All the relevant files and records from the construction period have been researched and no information on the implementation of quality assurance/quality control procedures is available.

The masonry walls at Indian Point, Unit 2 are single wythe hollow concrete block, multi wythe solid block and multi wythe brick. The specification for the masonry required Dur-o-Wall horizontal joint reinforcing but no account was taken of this in the evaluation, which considered all walls as unreinforced. Therefore the only physical characteristics of the construction important to the evaluation were the block, brick and mortar properties. Specified minimum strengths were used for each of these components, even though experience has shown that walls tested several years after construction invariably have strengths well above the minimum specified strength.

All of the walls were inspected by personnel from Computech Engineering Servies, Inc. during site visits on December 8 to 11, 1980, on June 15, 1981 and September 16, 1981. This visual inspection of the walls indicated that, with two exceptions, the condition of the walls was very good. The walls had good vertical alignment, there were no visible cracks, the masonry units were of good quality and the mortar joints were well constructed and showed no signs of deterioration.

The only exceptions were in the Fan House, where some cracking of mortar joints had occurred over restricted portions of the South and West walls. These cracks generally occurred in the region where the walls were supported by embedded plates. It is proposed to fix these regions by the application of an epoxy surface treatment. The common wall between the Fan House and the Fuel Storage Building has some separation at the interface of the masonry blocks and the adjacent steel columns. Proposed modifications include the addition of clip angles around the periphery of these effected walls.

9 CONCLUSIONS

A detailed response has been presented for each of the items from the NRC review which were not fully replied to in the previous submittal. Following is a brief summary of the conclusions reached on each item:

a. Single vs Multi-Wythe Analysis Four walls were analyzed for both single and multi-wythe action to demonstrate that the single wythe assumption is conservative. The frequencies for the multi-wythe walls were increased to the rigid range and the stress ratios reduced several times if composite action was assumed.

b. Allowable Shear Strains The applicability of 35 tests to the Indian Point Unit 2 masonry walls was demonstrated and the results evaluated to justify the allowable shear strains used in the criteria. From a statistical analysis the range of safety factors on the criteria values compared with the mean test values was 2.15 to 2.90 for an OBE event and 2.13 to 2.61 for the SSE. These values are considered satisfactory.

c. Allowable Shear Stresses The criteria limit for in-plane shear stresses was revised in accordance with the SEB criteria, i.e. with a 30% increase for factored loads.

d. Applicability of Test Results The applicability of the test results used to justify the allowable stresses was discussed.

e. Tension Parallel to Bed Joint Statistical analysis was carried out on the results of tests for tension parallel to the bed joint for both concrete and brick masonry. Safety factors on the mean were 6.16 to 7.81 and 3.67 to 4.65 for the OBE and SSE events respectively for brick masonry. For the concrete masonry the computed safety factors had a mean of 6.40 for unfactored loads and 3.83 for factored loads. These results are considered to fully justify the criteria values and the 1.67 increase factor for factored loads.

f. Tension Normal to Bed Joint Similar statistical analyses were performed for brick and concrete masonry for tension normal to the bed joint. Factors of safety on the mean for concrete block were 1.68 to 4.39 for OBE and 1.01 to 2.64 for SSE events. For the brick the corresponding ratios were 2.77 to 3.53 for OBE and 1.65 to 2.10 for SSE loads. The lower bound values for the concrete masonry were adversely influenced by a number of tests using point loading on samples cured for only 15 days. In view of this the analyses are considered to justify the criteria stresses.

g. Special Inspection Stress Categories Although no quality assurance or quality control information is available from the time of construction the results of recent inspections of the wall showed good quality construction except for two areas which are being corrected.

The responses contained in this report plus those of the earlier submittal are considered to fully reply to all the NRC requests and to confirm the validity of the criteria for the Indian Point, Unit 2 masonry wall evaluation.