MIDLAND UNITS 1 & 2
JOB NO. 7220

REVIEW OF U.S. TESTING FIELD AND LABORATORY CONSTRUCTION TEST DATA ON SOILS USED AS FILL

BECHTEL ASSOCIATES PROFESSIONAL CORPORATION
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TEST DATA ON SOILS USED AS FILL

This review of the quality control tests of the earth fill at the Midland Site was made as a result of settlement of the fill supported diesel generator building in excess of that predicted. Soil samples obtained in borings indicated that soil conditions beneath the plant structures are not compatible with the quality of fill that could be expected based on the results of the control tests made by U. S. Testing Company. All fill was accepted as it was being placed based on the results of the field tests performed by U. S. Testing Company.

The review showed many discrepancies in the test results as outlined in the following paragraphs. Review comments are based on the requirements of the technical specifications for fill placement and to subcontract entered into by U. S. Testing Company.

1. <u>Use of Laboratory Test Compaction Curves</u>

Table 9-1 of specification 7220-C-208, Page 14B required one field density and moisture content test be taken for each 500 cubic yards of fill placed. It also required one compaction, grain size, and specific gravity for each 10,000 cubic yards of material. This gives a ratio of 20 field density tests to 1 laboratory compaction test. Although 20:1 is not a strict upper limit, it is a guideline; should density tests be taken more frequently than one per 500 cubic yards of fill the ratio could be higher. The actual ratio is shown in Table A attached. In fact, some of the laboratory compaction tests were used to determine percent compaction for several hundred field density tests taken over a period exceeding two years. Even though no time requirements for the period of use of laboratory tests are specified, it is unlikely that any borrow source in this area would be of such uniform character that such extended use of a compaction curve, truly representative of a large quantity of material, would be applicable. Listed below are selected laboratory test data results indicating the wide range of soil properties that were reported. Such a wide range is typical for soils of the kind used in the fill making prediction of maximum density, based on visual inspection extremely difficult if not impossible without testing.

TEST	MIN. DENSITY (lbs/Ft^3)	MAX. DENSITY (lbs/ft^3)	OPT. MOISTURE (percent)
*BMP269		127.3	10
*BMP278		117.0	15.2
*BMP279		140.8	5.7
**RD24	100.9	119.2	
**RD55	90.2	109.7	
**RD61	109.3	125.3	

*BMP refers to proctor type test.
**RD refers to relative density test run by dry method.

2. Questionable Retests

A field density test that fails to meet requirements of the specification should have been reported to Bechtel who then would have required reworking of the area and retesting.

Of the 668 "failing" tests which were marked "cleared" by another test, in over 10% (72 tests) of the results, the clearing of the "failed" density test was apparently resolved by merely using another laboratory compaction curve with either lower maximum density, which resulted in in the percent compaction being increased sufficiently, or different optimum moisture content which caused the fill to meet the requirements of the specification. The possibility exists that soil was removed after a "failing" test and replaced by different material, but the records do not indicate this and it is not possible from the record to determine if a new density test was made. In other cases, tests labeled "failed" were incorrectly cleared though the same laboratory standard was referenced. For example, in some cases retests to clear a "failed" test were not taken in the same area or at the approximate same elevation. More than 40 retests were over 20 feet from the "failed" test location (as recorded in the test reports) and some were over 200 feet from the original test location. In general, if after a "failing" test the whole area is reworked, the density test location is not too critical assuming that the correct laboratory compaction curve is used for comparison. However, in the plant fill work areas were relatively small, and soil characteristics showed considerable variation necessitating retesting in the immediate vicinity of the "faili-g" test. Retest should be taken in the lift or soil layer that has been reworked. Almost 50 retests were taken at different elevations, some up to 10 ft. from the "failed" test. It should be noted that Bechtel field personnel gave the locations for retesting. This was not a U. S. Testing responsibility. Two retests were dated prior to the time the original test "failed". Over 130 "failing" tests were marked as ("non Q") and never recorded cleared, as they were outside the saftey related area.

Table 3 is a compilation of notes relative to questionable clearing of failed tests.

3. Theoretically Impossible Test Results

Soils cannot be more than 100 percent saturated; therefore, all field density test data points, when plotted as dry density versus moisture content, must be below the zero air voids curve as defined by the specific gravity of the material. Specifications do not require examination of the zero air voids curve, but it is considered common practice relative to compaction plots. There are numerous cases in the U. S. Testing Company data where points plot above the zero air voids curve. Figure 1 attached shows a typical laboratory compaction test curve with field test results plotted on it. Many of the field test results are to determine percent compaction plot above the zero air voids curve. Provided the specific gravity is correct this is not possible so that all such points must represent erroneous data.

The fact that a large number of test results plot above the zero air voids curve tends to make all test results questionable.

Also, referring to Figure 1 it would appear that soil density varied widely. Specifications called for compactive effort results as defined by ASTM D 1557 which is 56,255 ft-lb/ft^3 energy. This was modified to a laboratory test compactive effort of about 20,000 ft-lbs/ft^3 energy, often referred to as Bechtel Modified Proctor (BMP). Laboratory compaction test curves should be related to the same effort as that called for in the field for use in comparing with field density tests to determine percent compaction. According to plots of field data shown on Figure 1, density varied from about 108 lb/ft^3 to about 130 lb/ft^3. It is doubtful that the soil classification or other properties would be similar for such a wide variation in density. It is noted that 100 percent of modified Proctor (ASTM D 1557) which is difficult to obtain, is rated at 56,255 ft-lb/ft^3 energy. The curve plotted on Figura 1 is at about 20,000 ft-lb/ft^3 energy. For comparative purposes it was determined by U. S. Testing in 1974 that 100 percent of specified effort (20,000 ft-lb/ft^3) is approximately equal to 95 percent of the maximum density as determined by ASTM D 1557 (56,255 ft-lb/ft^3) Reference Figure 8.

4. Repeated use of Questionable Laboratory Test Data

Some laboratory compaction test data were used repeatedly even though they continued to show suspect field test results. This could be indicative of questionable laboratory data or the fact that soil was not being placed or compacted according to specifications. Either case is a cause for concern.

Several specific gravity calculations are in error, such as for BMP 273 and 274. In the case of BMP 273, the zero air voids curve passes through the laboratory compaction curve. In another example, BMP 297, the laboratory compaction curve is invalid due to calculation errors, yet was referenced by field density tests 22 times.

Table C is a compilation of notes relative to questionable test data.

5. Limits of Accuracy and Acceptability for Test Data

Figures 1 through 7 attached will be referenced in discussing limits of accuracy of acceptability for field test results as compared to laboratory test data. The figures show plots of compaction data for BMP 278 which are typical of all test results.

- Specified laboratory compactive effort was 20,000 ft-lbs/ft^3 and field compaction effort was originally specified at 56,255 ft-lbs/ft^3 but was changed by Revision 5, dated 7/8/75, specification 7220-C-210, Section 13.7, Page 57 to also be equal to about 20,000 ft-lbs/ft^3.

The specified 20,000 ft-lbs/ft^3 effort establishes a compaction curve relating moisture and density for a specific soil. Moisture was specified for field placed fill to be within $\pm$ 2 percent of optimum moisture as determined by this effort. Density was specified to be greater than 95 percent of the maximum density. As compactive effort is increased in the laboratory test, maximum density will be increased and optimum moisture content will decrease. This change can only occur in the field to the extent that the field moisture content will permit it. Once field compaction is such that the fill density is significantly higher than about 105 percent of maximum, the specified tolerance from optimum moisture content in the laboratory compaction test may no longer be applicable for field control. A + 2 percent numerical value of moisture content acceptable at the specified compactive effort would be too wet at a higher effort since the zero air voids curve defines the absolute maximum that can be achieved, indicating that higher densities for that soil are impossible. Therefore, if the record shows high densities for such material, the data are in error. This was apparently overlooked.

Plots of field data for compaction test BMP 278 are shown on Figures 1 through 6. The title of each figure gives the assumptions made in plotting data for the figure. In comparing figures 3 and 4 it is seen that a majority of field tests were made using the nuclear device. The two test results shown on Figure 4 for the sand cone method indicates one test result on each side of the zero air voids curve. The one falling above the zero air voids curve (shown on Figure 4) is designated by U. S. Testing Company as the only passing sand cone test (shown on Figure 6).

For a field test result to be valid as well as "Passing" it must fall within a well defined area on the plot containing the laboratory compaction curve. This area or window of acceptability is shown for a hypothetical compaction curve on Figure 7a that would meet requirements of Specification 7220-C-210. It is defined by horizontal lines at 95 percent and 105 percent of specified density, vertical lines through $\pm$ 2 percent of optimum moisture content, and a line parallel to the zero voids line indicating saturation about half way between the compaction curve and 100 percent saturation (zero air voids curve). The practical upper limit of 105 percent of specified density is not defined in the specifications. It was arbitrarily chosen as numbers greater than this give increasingly invalid comparisons between field test results and the specified laboratory compaction test curve. Therefore, if all data points fall within the defined window there would be no reason to assume that they are wrong. However, when many data points fall outside the designated area there is something wrong with the information and then all data points become suspect. A review of all data indicates that about 25 percent of the cohesive soil test results fall within this area.

Figure 7B shows an area where field test results would be acceptable, in theory even though not in strict accordance with the specifications. Figure 7B was arrived at by expanding Figure 7a to include test results up to a compactive effort related to ASTM D 1557 (56,255 ft-lb/ft^3) which is considered to be a practical upper limit. About 40 percent of all cohesive soil test results would plot in this area.

6. Accuracy of Test Equipment

Almost all (over 95%) field density tests on cohesive soils were made using the Nuclear Density device. Specification 7220-C-210 section 12.4.2 page 42 indicates this to be acceptable for moisture content determination provided that the results are compatible with those obtained by ASTM D 2216. Similarly, section 12.4.4 says density determined by the nuclear device is acceptable when results are compatible with density as determined by ASTM D 1556.

In a letter from U. S. Testing to Bechtel (dated May 30, 1974), the average deviation of the nuclear device from oven-dry moistures was +.12% for a set of 30 tests. However, the standard error of estimate is 1.8% for the data with the range of differences being from - 3.2% to +3.9%. Thus, accuracy of the nuclear device is questionable, and could translate into errors of about ± 4 pcf in the dry density calculation. (It should be noted that errors in the moisture content tend to shift the position of test results on a moisture density plot approximately parallel to the zero air voids curve, assuming the in-place wet density is correct, and thus do not explain the large number of points which plot outside the zero air voids. Compare Figures 1 and 9).

No reliable correlation between sand cone and nuclear density tests were carried out therefore there is no basis for determining if U. S. Testing would have performed better using the sand cone procedure.

However, it is clear that a large number of the nuclear density tests are wrong. This can be explained by considering the wet unit weight may have been wrong or both the moisture content and unit weight may have been wrong. A reliable correlation with properly conducted sand cone tests should have revealed this, but it was not apparently done.

7. Relative Density Tests

Cases were noted where densities in material classified on the data sheet as zone 3 (sand) were compared to the maximum densities in proctor type tests and other cases where densities in clay soils were compared to the maximum density in relative density tests. An error must exist in the record in such cases either in the classification of the soil on data sheet or in comparing field test results to inappropriate laboratory test data. In general, it appears that relative density tests were used in controlling density of sand fill. There were a significant number of arithmetic errors on calculation sheets even though there are signatures on the sheets indicating they had been checked. Over 100 errors were found in calculations, of relative density from 8/15/79 through 12/78 (not all of these errors change the acceptability of the test results).

ASTM D 2049 section 7.1.2 Wet Method states: "Note 2 - While the dry method is preferred from the standpoint of securing results in a shorter period of time, the highest maximum density is obtained for some soils in a saturated state. At the beginning of a laboratory test program, or when a radical change of materials occurs, the maximum density test should be performed on both wet and dry soil to determine which method results in the higher maximum density. If the wet method produces higher maximum densities (in excess of one percent) it shall be followed in succeeding tests." An example of wet and dry relative density is shown on Figure 10. U. S. Testing Company apparently did not do this frequently enough, or on a broad enough range of non-cohesive soil types. As a consequence many field density test results exceed 100 percent of maximum dry laboratory relative density. As an example, for laboratory test RD55 a total of 566 field tests were made. Of this total, 364 tests were greater than 100 percent compaction. The highest relative density found was 142.2 percent with the majority of tests over 100 percent falling in the range of 100 percent to about 130 percent. Since the difference in maximum density between wet and dry methods is about 4 to 5 lbs/c. ft. (based on recent data) any test result greater than about 115 percent (based on the dry method) is suspect.

Even if the wet laboratory test method data were available for all sands, it appears an unacceptably high number of field test results would greatly exceed 105 percent relative density even based on the wet maximum.

8. Summary

In summary, there are five major faults contained in the Midland Compacted Fill Density Test Reports as follows:

1. erroneous field density test data.
2. incorrect soil identification
3. incorrect (or questionable) laboratory test data.
4. calculation errors
5. improper or incomplete clearing of "failed" tests.

Items 4 and 5 represent existing faults in the data which could be corrected. However, as a result of items 1 through 3, there is no rational means of determining which test results are valid and which are not. Since more than one half of the test results for relative density and percent compaction fall outside the possible theoretical comparison limits, it must be concluded that these test results are suspect and should not be used alone for acceptance of plant area fill. Therefore, other means of testing have been established and employed to determine if the fill in any given area is acceptable.

Also in item 4 it should be noted that on many occassions the inplace density was divided by the maximum density from the relative density test to get percent compaction, these tests were also used to clear other pricing tests.

TABLE A

Listing of All Classifications Referenced in Plant Area Fill Soil Test Records Which were Used for 20 or More Field Density Tests

Classification	No. of Tests
B200	90
B251	31
B252	22
B254	42
B255	57
B260	68
B261	36
B262	165
B269	227
B270	226
B271	141
B274	37
B276	21
B277	158
B278	82
B297	22
R015	20
R016	61
R024	248
R030	54
R035	59
R038	39
R039	28
R040	35
R041	69
R042	103
R043	48
R044	71
R045	43
R049	63
R054	118
R055	566
R059	65
R061	589
R063	42
R065	59

Note: Spec. 7220-C-208 gives a ratio of approximately 20 field tests to each laboratory test.

TABLE B

Notes on Questionable Clearing of Failed Tests

1. Test number MD 245 fails due to high moisture. Cleared by MD 246 which references a proctor with higher optimum moisture content (OMC) such that the ±2% of optimum requirement is met.

2. MD 205 fails with moisture content 6% above the OMC. Cleared by MD 215, which references a relative density lab standard, and is itself still 6% away from the OMC of the proctor referenced by MD 205.

3. MD 223 fails because of high moisture. Cleared by MD 228 which has actually a higher moisture content and lower density, but references a different proctor; the retest passes and clears the failure.

4. Both MD 844 and 886 fail because of high moisture and low density. They are cleared by MD 888 which references a new proctor with lower maximum density and higher OMC than the first.

5. MD 251 fails due to moisture being too high. Cleared by MD 253 which uses a higher OMC proctor.

6. MD 668 clears MDR 634, but the two tests show no correspondence in location, moisture, density, or lab standard.

7. MD 771 failed, being too dry. Cleared by MD 782, which has almost identical moisture content and dry density but uses a new BMP with lower optimum moisture.

8. MD 2384 clears MD 2342, referencing a different proctor with an OMC which fits the in-situ conditions. However, the dry density of MD 2384 is way too high to fit the original soil classification, and in addition, it falls outside of the zero air voids curve for the classification which it has been changed to.

9. MD 556 clears MD 554 by using a BMP with lower moisture requirements. The field densities differ by 24 pcf and would seem to be different material.

10. MD 558 clears MD 555 but has too high a density to be the same soil as MD 555. It also uses a different proctor.

11. MD 566 and 568, classified as BMP 262 cohesive soils, are cleared by MD 569 which is classified as RD 33 and has totally different soil properties than the two failures.

12. MD 1317, 18, 19 and 20 fail and are all cleared by MD 1477 taken over 5 weeks later. There is poor correspondence in the soil properties and the proctor is different from failing to passing test.

13. MD 2965 clears MD 2963 with a different proctor through the test results would have been passing with the original BMP.

14. MD 1388, classified as BMP 278, is cleared by MD 1461, classified as RD 55.

15. MD 170, classified as RD 24 is cleared by MD 173, classified as BMP 234.

16. MDR 287 fails with a relative density of 77%. Cleared by MDR 291 which has .1 pcf lower density but arbitrarily rounds up the relative density to 80%; it passes and clears the failure.

17. In all of the following field density tests on sand, the passing test has approximately the same or lower density than the failures, but references a lower maximum density RD lab standard:

MDR 343	clears	MDR 339
MDR 514	clears	MDR 507
MDR 513	clears	MDR 508
MDR 515	clears	MDR 509
MDR 516	clears	MDR 510
MDR 522A	clears	MDR 521
MDR 558	clears	MDR 556, 557
MDR 480	clears	MDR 473
MDR 555	clears	MDR 525, 527, 534
MDR 533	clears	MDR 526, 530, 531

18. MD 2384 clears MD 2342, but is at 7' lower elevation.

19. MD 123 clears MD 122, but is at 10.5' lower elevation.

20. MD 149 clears MD 142, but is at 10' higher elevation.

21. MD 1694 clears MD 1693 but is 43' away from the site of the first test.

22. MD 3114 clears MD 3102, but the two tests are 68' apart.

23. MD 186 clears MD 183 though it is 110' away.

24. MD 1209 clears MD 1207 and MD 1205, yet is 183 ft. away from the failures.

25. MD 1097, dated August 4, 1977, cleared by MD 1048 dated July 16, 1977.

Note: This table gives typical observations and is not meant to be all-inclusive.

TABLE C

Notes on Questionable Test Data

1. The first field density test to reference RD 24 (5/75) has a relative density of 170.6%. The standard continued to be used, however, with relative densities greater than 100% occuring repeatedly.

2. Similarly for RD 30, the first two tests (9/75) have 114% and 122% relative densities, yet the standard was used for 10 months, 54 tests, with 52% of the results over 100%.

3. During the first two weeks of use (7/76), RD 41 was referenced 22 times with 12 tests over 100% relative density (6 tests over 110% and 3 over 120%). The standard was used for 5 months, however, with over 40% of the results over 100%.

4. The first test using RD 55 (8/76) has a relative density of 119%, with the field test being made the same day as the standard and, thus, assumedly the same material. These results would throw doubt on the lab standard, yet it was used for two full years and 566 tests, with 64% of the results over 100% relative density.

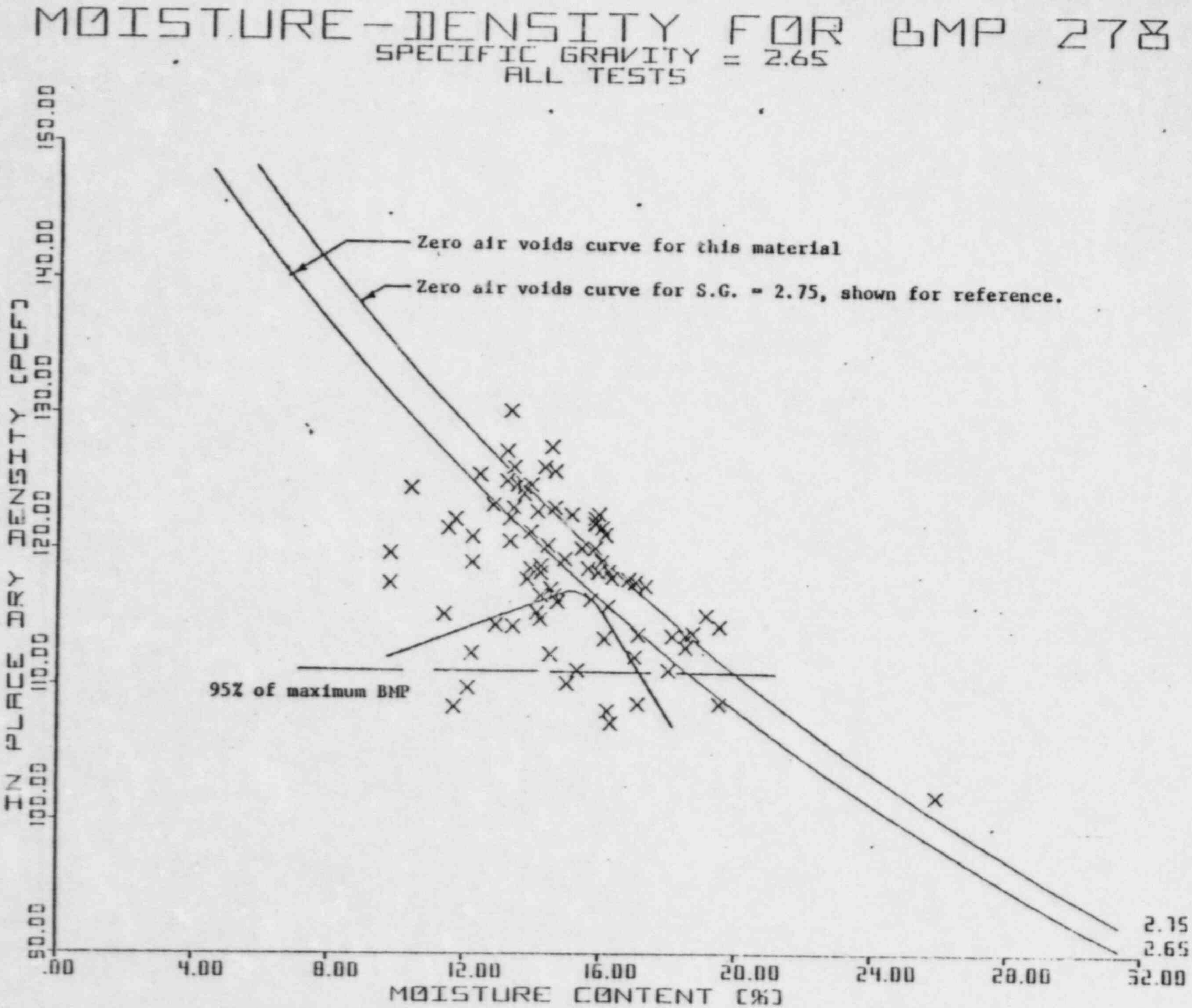
5. Even high density structural backfill standards such as RD 61 (maximum density of 125.3 pcf), used 593 times, show over 25% of the tests having greater than 100% relative density.

6. The first seven tests referencing BMP 269 (scattered over a two month period around 7/76) all fall outside the zero air voids curve. This classification was used for 1 1/2 years, referenced 227 times.

7. The first two tests referencing BMP 270 (7/76) fall 6 pcf above the zero air voids curve. Continued use of this proctor for over 2 years resulted in 226 tests with 82 outside the theoretical maximum.

8. For the first month (4/77) all BMP 278 tests fell on or outside the zero air voids curve. For the next month, over half the tests did the same, or have greater than 105% compaction. The standard was used over half a year, with 43 out of a total of 82 tests outside the zero air voids curve.

Note: This table gives typical observations and is not meant to be all-inclusive.



MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
ALL TESTS
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
Zero air voids curve for this material
Zero air voids curve for S.G. = 2.75, shown for reference.
95% of maximum BMP
2.75
2.65
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MØISTURE CONTENT (%)

FIGURE 1

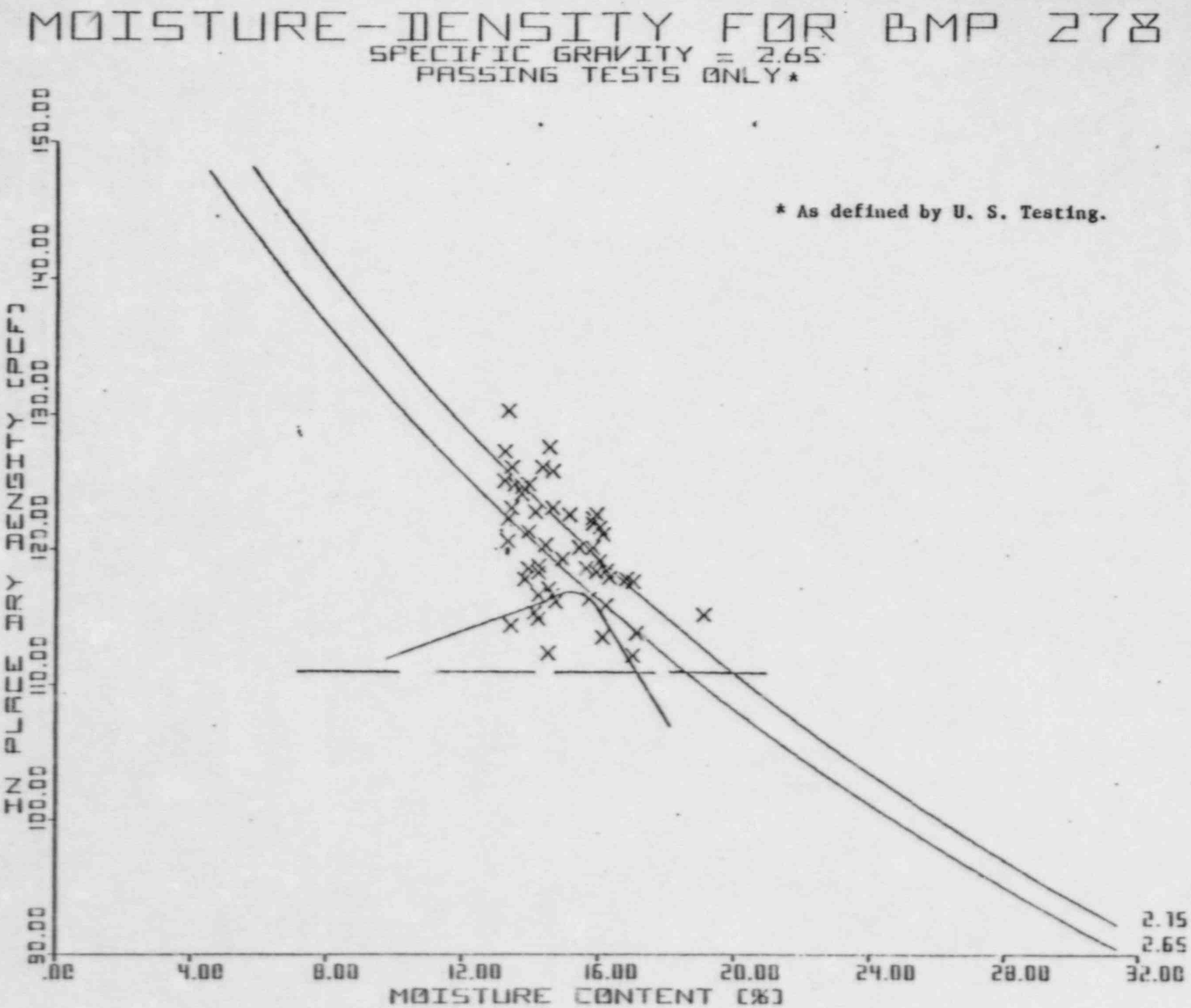

MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
PASSING TESTS ØNLY*
* As defined by U. S. Testing.
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MØISTURE CØNTENT (%)
2.15
2.65

FIGURE 2

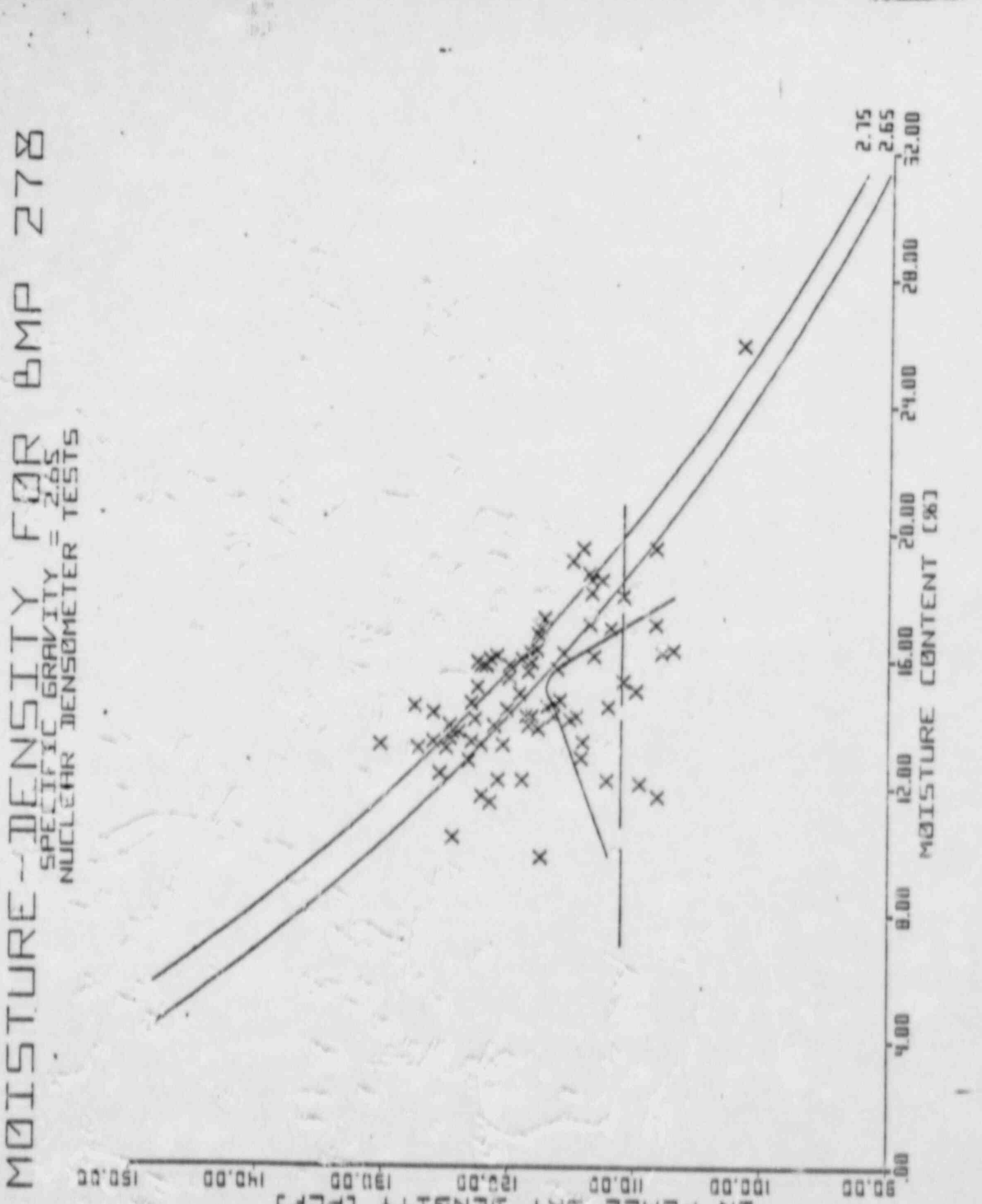

MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
NUCLEAR DENSØMETER TESTS
IN PLACE DRY DENSITY (PCF)
MØISTURE CONTENT (%)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
2.75
2.65

FIGURE 3

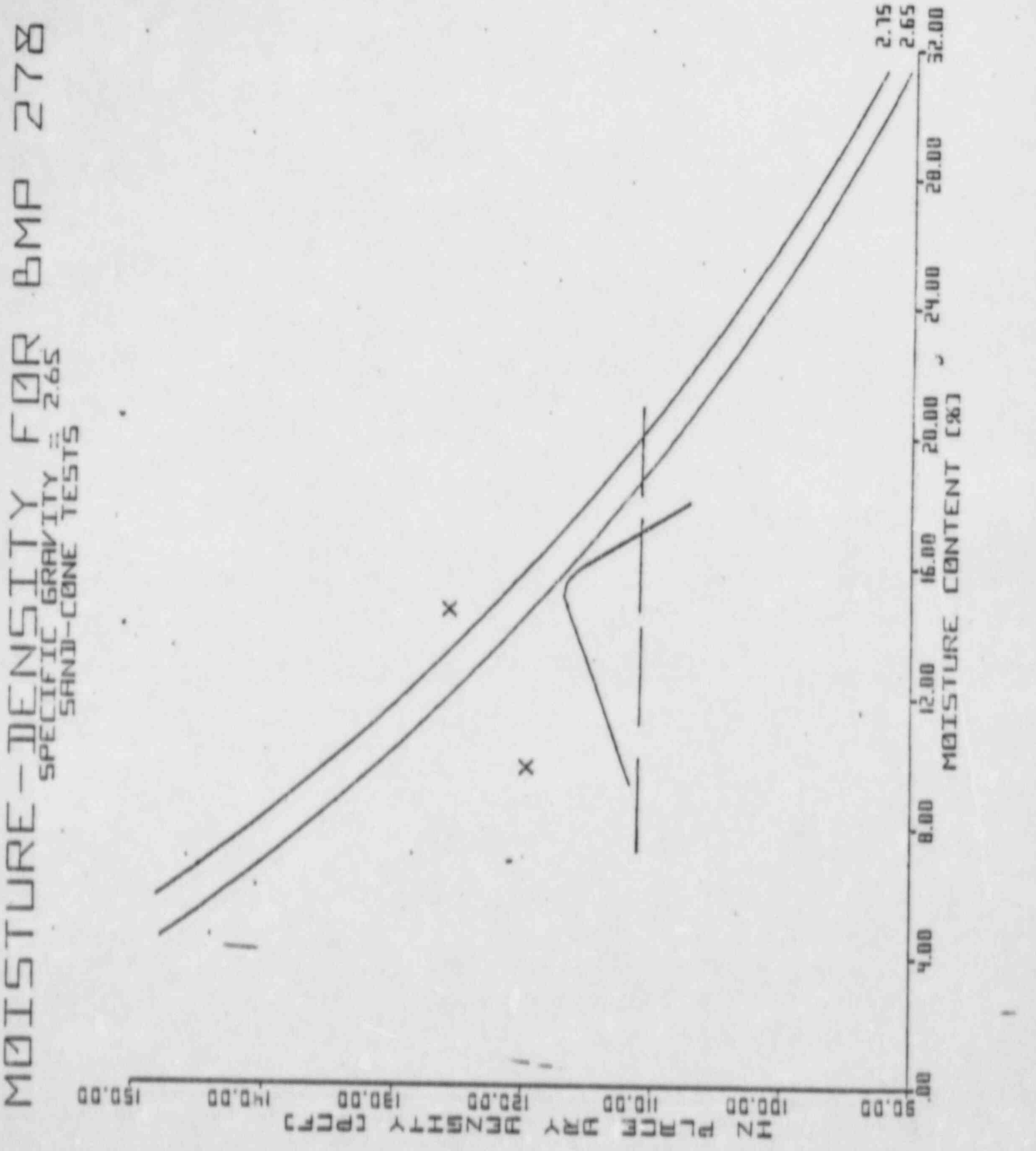
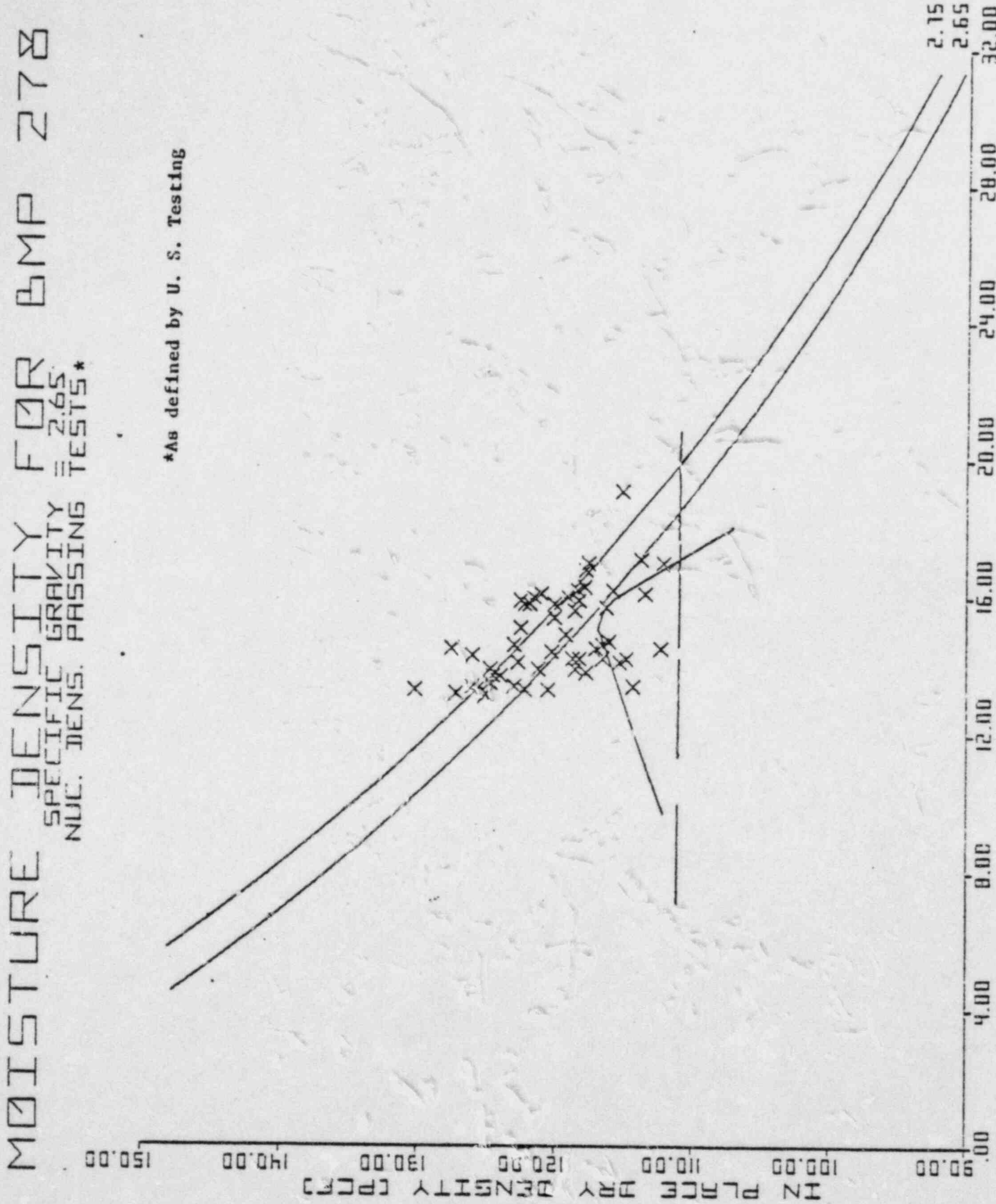

MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
SAND-CONE TESTS
2.75
2.65
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MØISTURE CONTENT (%)
90.00
100.00
110.00
120.00
130.00
140.00
150.00
IN PLACE DRY DENSITY (PCF)

FIGURE 4



MØISTURE DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
NUC. DENS. PASSING TESTS*
*As defined by U. S. Testing
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
MØISTURE CONTENT (%)
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
2.15
2.65

FIGURE 5

MOISTURE-DENSITY FOR BMP 278

SPECIFIC GRAVITY = 2.65
SAND-CONE PASSING TESTS *

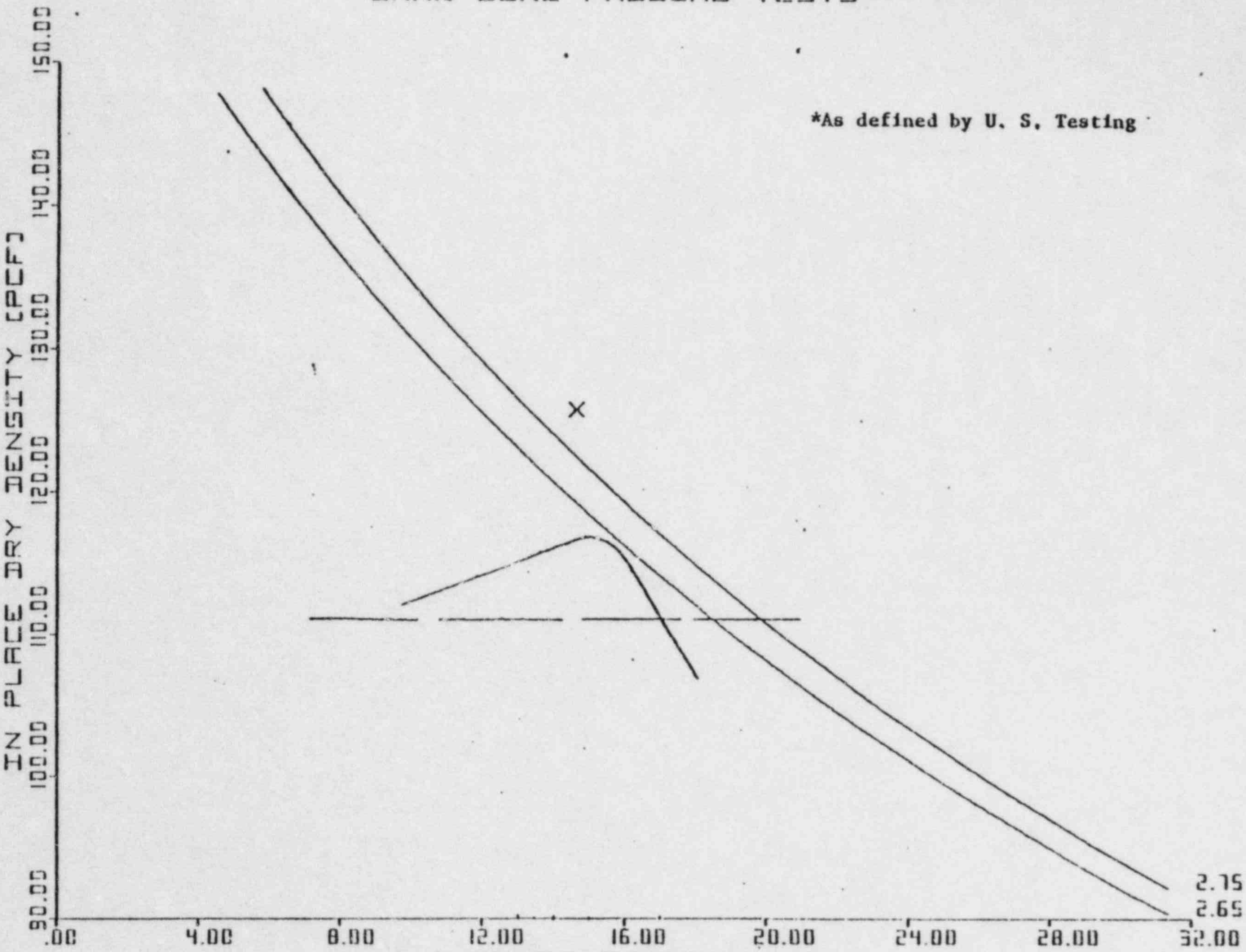


FIGURE 6

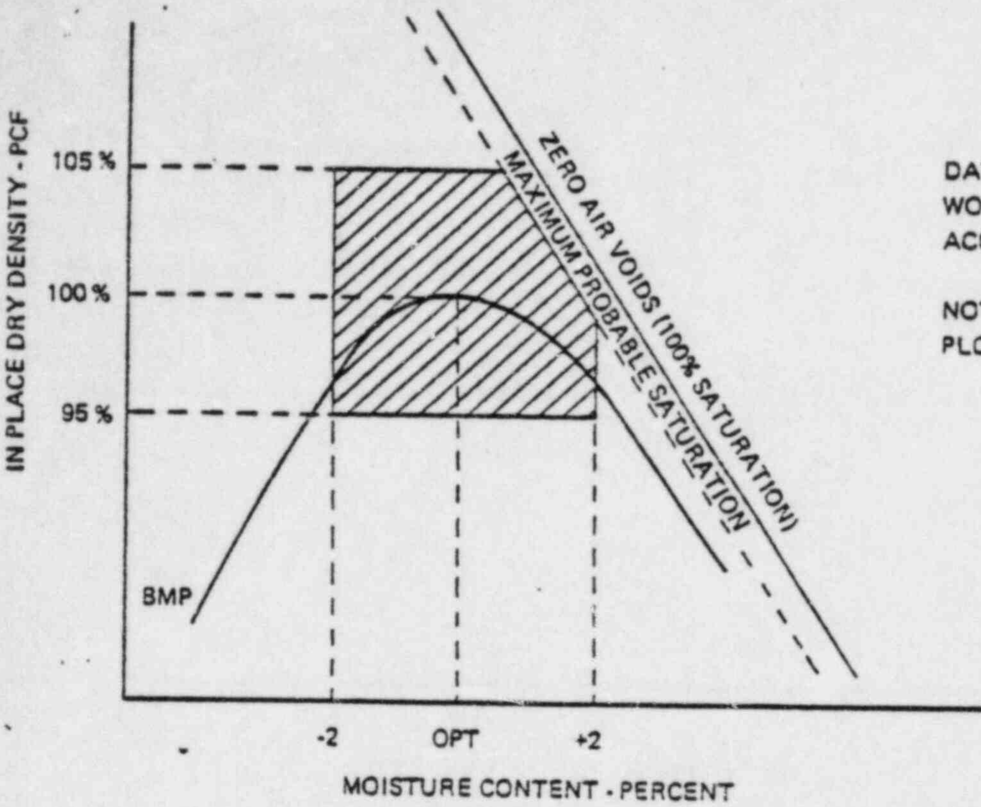


FIGURE 7-A-

DATA POINTS THAT PLOT IN SHADED AREA WOULD BE GENERALLY ACCEPTABLE ACCORDING TO SPECIFICATIONS

NOTE: ABOUT 25% OF ALL FIELD DATA PLOTS IN THE SHADED AREA

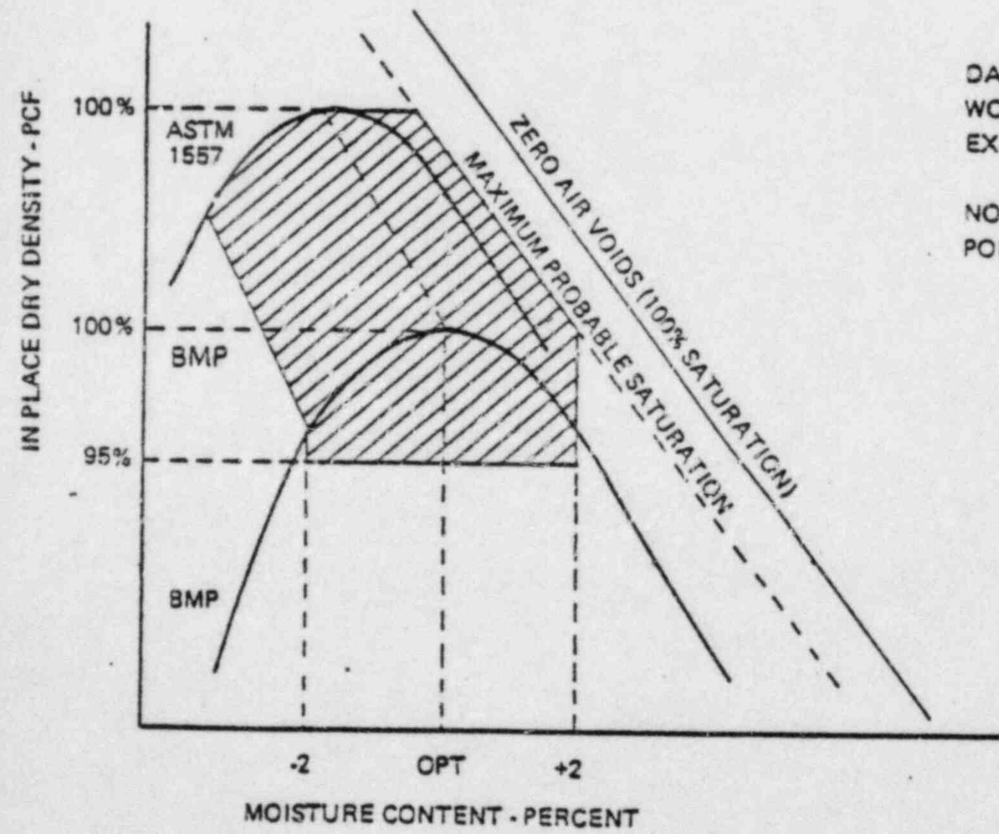
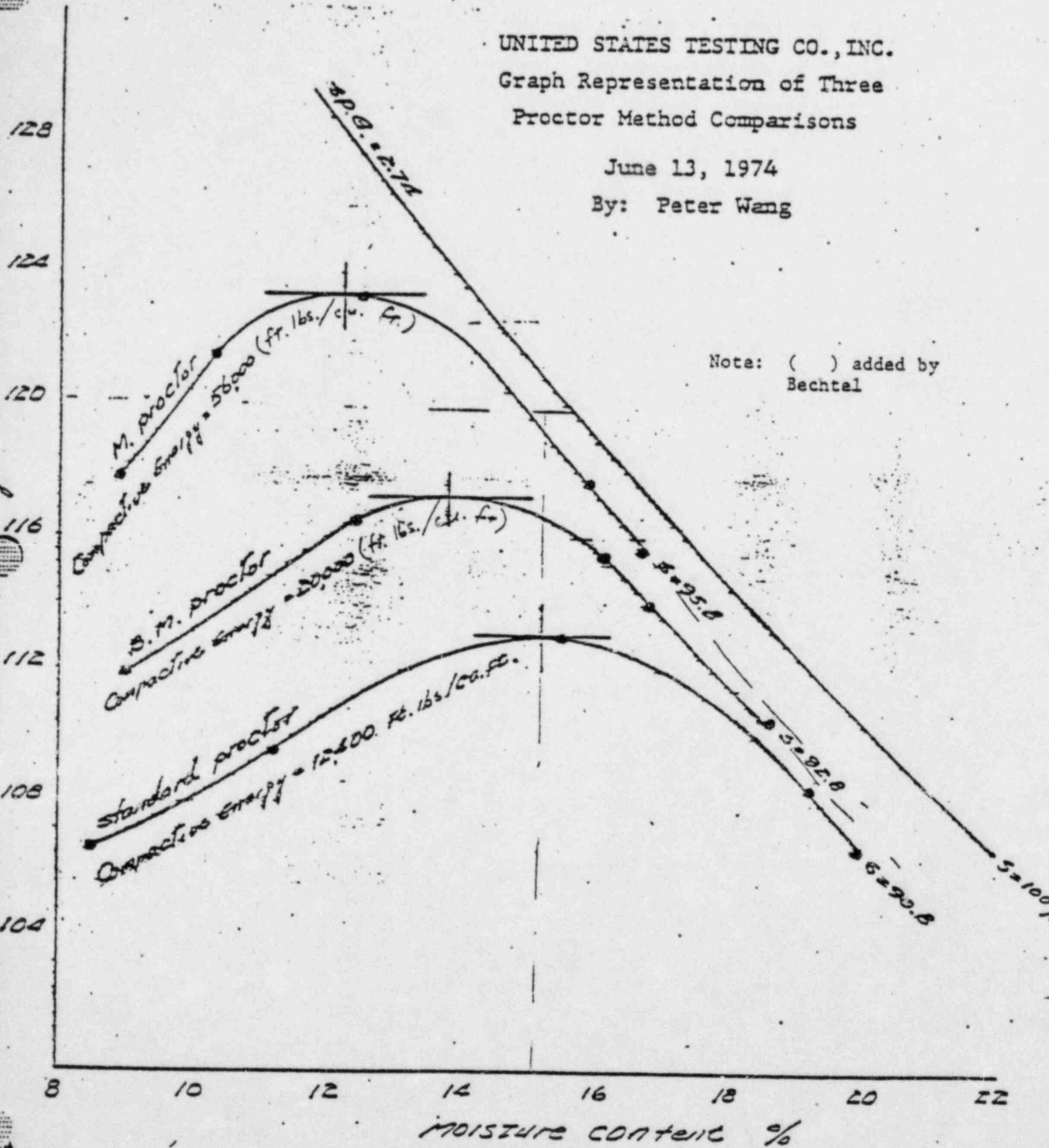


FIGURE 7-B-

DATA POINTS THAT PLOT IN SHADED AREA WOULD BE ACCEPTABLE REGARDLESS OF EXACT SPECIFICATION WORDING

NOTE: ABOUT 40% OF ALL FIELD DATA POINTS PLOT IN THE SHADED AREA

FIGURE 7: WINDOWS OF ACCEPTABILITY (A) BASED ON BMP SPECIFICATION (B) REGARDLESS OF EXACT WORDING OF SPECIFICATION



UNITED STATES TESTING CO., INC.
Graph Representation of Three
Proctor Method Comparisons
June 13, 1974
By: Peter Wang
Note: (   ) added by Bechtel
M. proctor
Compactive Energy = 56,000 (ft. lbs./cu. ft.)
B.M. proctor
Compactive Energy = 20,000 (ft. lbs./cu. ft.)
standard proctor
Compactive Energy = 12,400. ft. lbs/cu. ft.
128
124
120
116
112
108
104
8
10
12
14
16
18
20
22
moisture content %

FIGURE 8

MOISTURE-DENSITY FOR BMP 278

SPECIFIC GRAVITY = 2.65

ALL TESTS

3.5% Subtracted from Moisture Content, Dry Density Recalculated

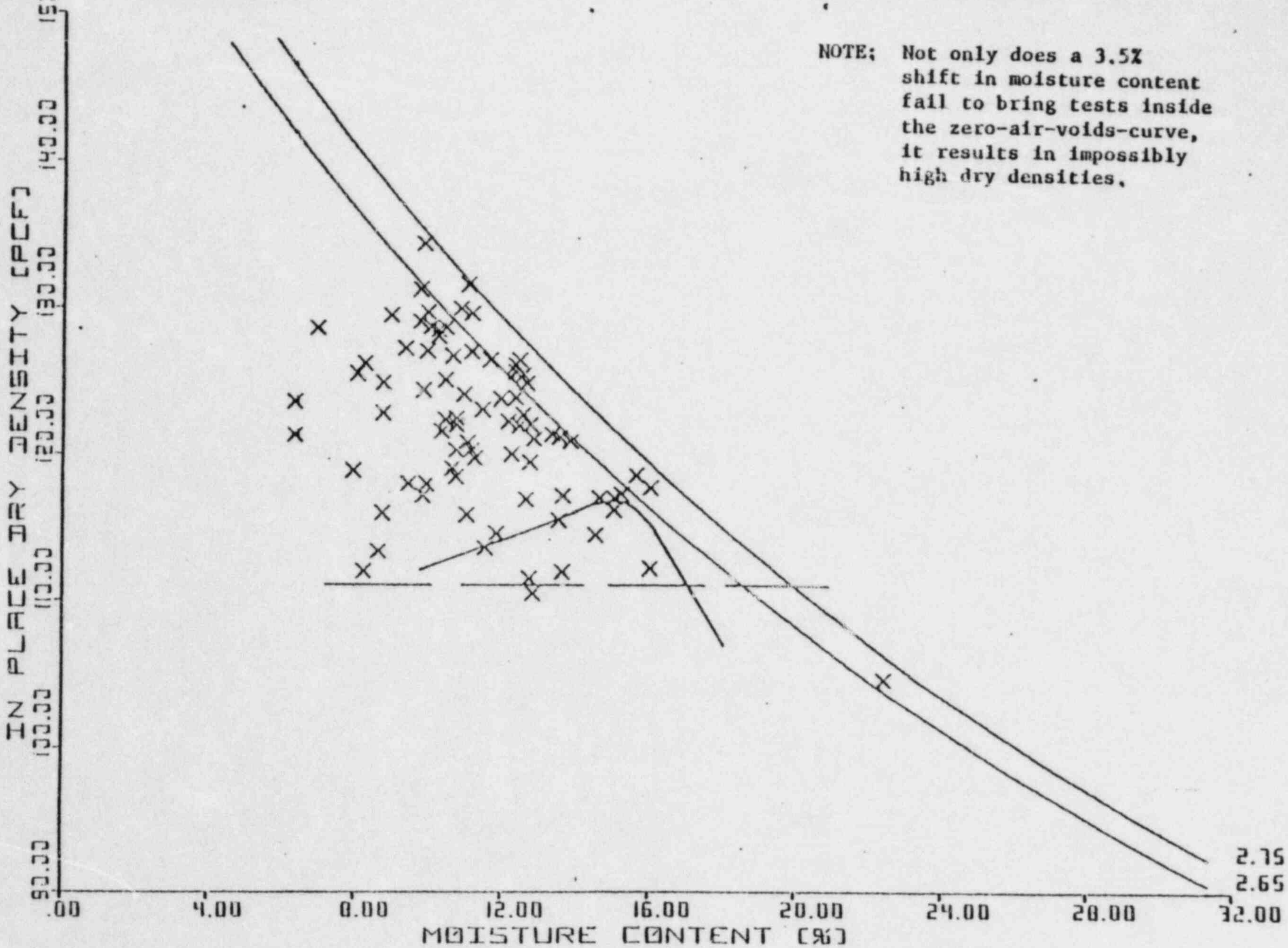
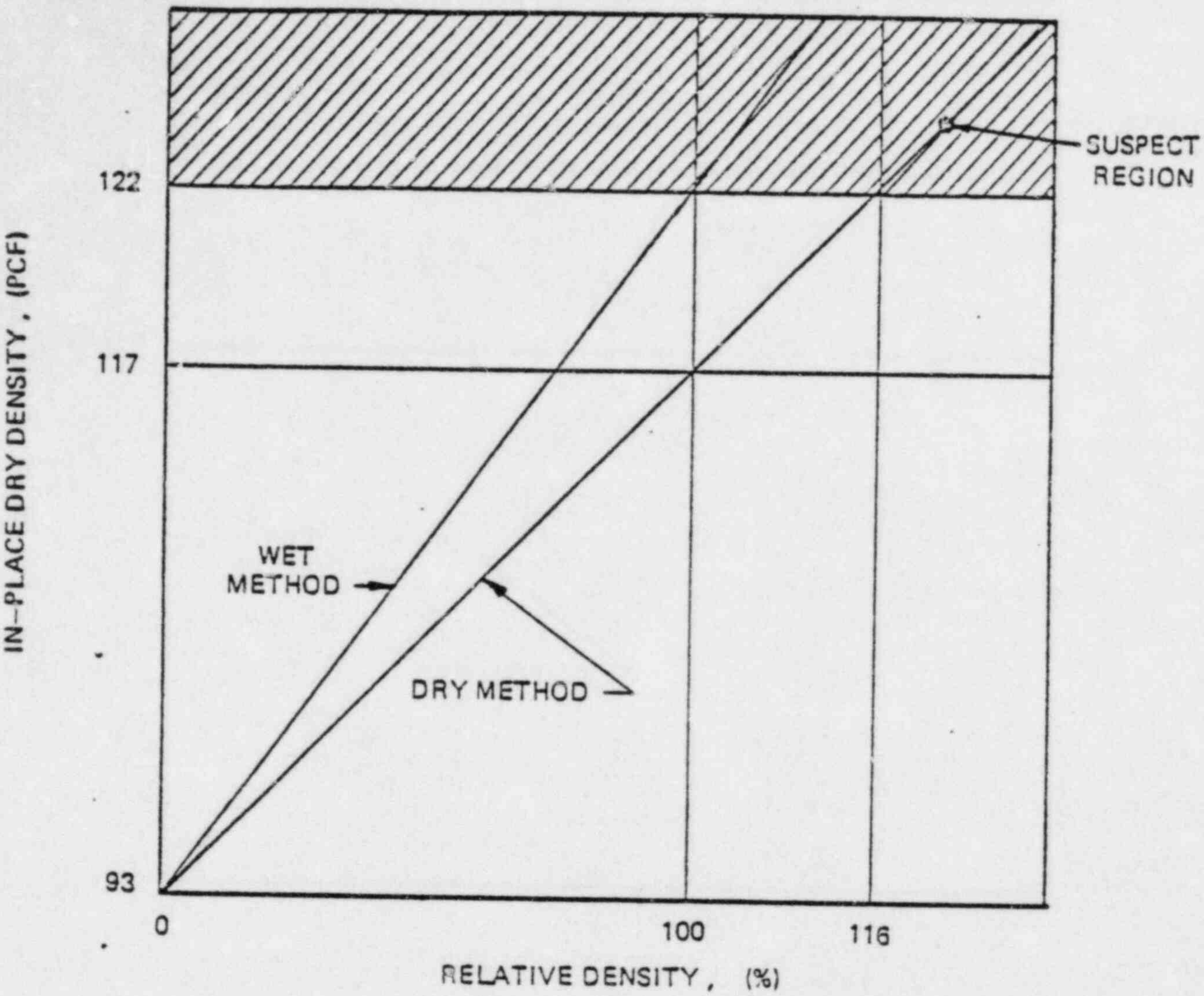


FIGURE 9



NOTE: VALUES FOR DRY DENSITY ARE TYPICAL OF A RANDOM FILL SAND. ANY TESTS SHOWING MORE THAN 117% RELATIVE DENSITY WOULD BE SUSPECT IN THIS EXAMPLE. STRUCTURAL SANDS TEND TO SHOW ONLY 2 OR 3 PCF INCREASE IN MAXIMUM DENSITY AND THUS RESULTS AT MUCH LOWER RELATIVE DENSITY WOULD BE SUSPECT, SAY 105 – 110 PERCENT

FIGURE 10

CHANGE IN RELATIVE DENSITY SCALE FROM DRY TO WET METHODS OF OBTAINING MAXIMUM DENSITY , BASED ON RECENT LAB RESULTS

~~Rough Draft~~ Rough Draft 175

The titles should read:

REVIEW OF U.S. TESTING FIELD AND LABORATORY CONSTRUCTION TEST DATA ON SOILS USED AS PLANT AREA FILL

DEHorn
7-6-79

First Paragraph on Page 1 states in part, "soil samples obtained in borings indicated that soil conditions beneath the plant structures are not compatible with the quality of fill that would be expected based on the result of the control tests made by U. S. Testing Company". I don't know how this statement can be made when no correlation has been made between questionable material and actual tests taken at that locatin.

Item 1 on Page 1 states in part, "although 20:1 is not a strict upper limit it is a guideline. Should density tests be taken more frequently than 1 per 500 cubic yards of fill, the ratio could be higher". This is misleading. C-211 for Plant Area Fill in Confined Areas the frequency of testing could be as frequent as 1 per 10 cubic yards of material to 1 per 100 cubic yards of material. This could give you a ratio of 1000:1 to 100:1 ratios respectively.

Item 1 goes on to state, "The actual ratio is shown in Table A attached" Does Table A include North Plant Dike, Northeast Dike and West Plant Dike data?

Item 2 on Page 2, 2ND para. states in part "In general, if after a 'failing' test the whole area is reworked." This may be a cause of the problem also that the whole area was not reworked, but just the failing lift.

Item 2 on Page 2, Second Paragraph, Last quarter of paragraph states in part, "it should be noted that Bechtel field personnel gave the location for retesting." This should state, "it should be noted that Bechtel field personnel gave the locations for testing and retesting".

Item 2, on Page 2, 2ND para. states in part "Retest should be taken in the lift or soil layer that has been reworked." Retest should be representative of the entire soil placement the test represents.

Item 3 on Page 2 states in part, "Figure 1 attached shows a typical laboratory compaction test curve with field test results plotted on it" Is this a typical laboratory compaction test with respect to the number of tests plotted to the right of the zero air voids curve or just a typical plot of a compaction curve?

First Paragraph on Page 3 states"the fact that a large number of test results plot above the aero air voids curve tends to make all test results questionable". I find this statement hard to believe. What is the large number of tests we are talking about in comparison with total number of tests?

Second Paragraph on Page 3 states in part, "Specifications called for compactive effort results as defined by ASTM-D-50.57 which is 56,255 feet-pound/cubic foot energy. For Method D, this value should be 56,000 foot-lbs/cu ft.

Page 3, Second Paragraph (except for the first sentence) the remaining portion of this paragraph does not seem pertinent.

Item 4, Page 3, First Paragraph, Last Sentence -- What is the reason for this sentence being placed here?

Item 4, Page 3, Second Paragraph states in part, "several specific gravity calculations are in error, such as for BMP273 and 374." This should state that the plottingsof the zero air voids curve on BMP273 and 274 are in error. What is the basis for this statement. The calculations for specific gravities on the calculation machine seem to be correct.

Item 5, Page 3, First Paragraph, last sentence states, "The figures show plots of compaction data for BMP278 which are typical for all test results." This statement is misleading. Is this plot showing the number of tests above the zero air voids curve typical for all tests using various BMP's.

Item 5, Page 3, Second Paragraph references 56,255 ft-lbs/ct ft. This should be 56,000 ft-lbs/ct ft.

Page 4, First Paragraph states in part, "this change can only occur in the field to the extent that field moisture content will permit it. Once field compaction is such that the field density is significantly higher than about 105% of maximum, the specified tolerance for optimum moisture content in laboratory compaction tests may no longer be applicable for field control." What is meant by this statement?

Page 4, 1st para. states in part greater than 95. Should be equal to or greater than 95.

Page 4, Second Paragraph, Last sentence states, "The 'one' following above the zero air voids curve ~~ixxqxxxxxxxhxxxx~~ (shown on Figure 4) is designated by U. S. Testing Company as the only passing sand cone test (shown on Figure 6)." What is the relevance of this statement?

Page 4, Third Paragraph and Fourth Paragraph reference Figure 7a. This should be Figure 7A.

Page 4, Last Paragraph states in part, "Figure 7B was arrived at by expanding Figure 7A to include test results up to a compactive effort related to ASTM D15.57 (56,255 ft-lbs/cu ft) which is considered to be a practical upper limit." I don't feel that this is a practical limit based on lift thicknesses and compaction equipment. Bechtel modified proctor is more applicable than practical.

Item 6 on Page 5 states, "almost all (over 95%) field density tests on cohesive soils were made using the nuclear density device." What are the actual numbers nuclear density device vs total test?

Item 6, Page 5, Second Paragraph, Second Sentence states, "However, the stand air of the estimated is 1.8% for the data with the range of differences mean from negative 3.2% to positive 3.9%." What is meant by the "stand air"

Same paragraph - further on - states "(in should be noted that errors in the moisture content tend to shift the position of test results on a moisture density plot approximately parallel to the zero air voids curve assuming the in-place wet density is correct and thus do not explain the large number of points which plot outside the zero air voids. Compare Figures 1 and 9)". Is the assumption that the in-place wet density is correct/valid based on the results of this report.

Item 7 on Page 5, last sentence states, "over 100 errors were found in the calculations on the relative density from 8-15-75 through 12-78. (not all of these errors changed the acceptability of the test results)." What were the actual numbers?

First Paragraph on Page 6, second-to-last sentence states in part, "the highest relative density found was 142.2%." This contradicts Item 1 in Table C which indicates relative density of 170.6%. Table A - Does this include tests in the west plant dike, north plant dike and northeast plant dike. Also does this include Q and Non-Q areas in the plant area fill?

Table B - Item 16 — is not clear. Is this stating that the retest had a density of 76.9 and it was rounded up to 80% using a different layup density standard

Figure 2 and 5. -- there are approximately four tests that are shown to the right of the ~~upper~~ optimum moisture content plus 2% line which should be failing tests but are shown as passing tests.

Figure 9 has subtracted 3.5% from the moisture content and recalculated dry density. Couldn't the nuclear densometer also be giving incorrect wet densities?

COMMENTS ON THE REVIEW OF U.S. TESTING FIELD AND LABORATORY CONSTRUCTION TEST DATA ON SOILS USED AS FILL

First Paragraph, Last line on first page - This sentence ~~might not be~~ is not completely correct. Some of ~~it~~ the fill may have been rejected and ~~cleared later~~ removed or ~~maybe~~ later cleared by a retest after reworking the material ~~never cleared.~~ This could be stated differently. Fill ~~was~~ is tested for acceptance at, ..., but not conclude what the results were.

Page Two, Second Paragraph, Last line - States, "over 130 failing tests were marked as Non-Q and never recorded cleared by a passing test." Does that mean that these tests were really taken in Non-Q areas and, because they were in Non-Q areas, were they just disregarded? Better put in words to indicate that they were marked Non-Q — in parenthesis - because they were taken outside the safety-related area.

Item 3, Second sentence - States that specifications do not require examination of the zero air voids curve but it is considered fundamental soil mechanics relative to compaction plots. If that's true, why didn't they require it and why did not Bechtel use this method years ago. It's a little late to be picking on the tester when Bechtel is supposed to head up all controls on testing.

Item 4, First sentence - States "some laboratory compaction test data were used repeatedly even though they continued to show suspect field test results." We do not understand that sentence. Are they saying that for some period of time, a long time ago, either Bechtel or US Testing recognized that they had data suspect results and continued to use these results? As stated, it is invalid conclusion because suspect is a recent event.

Page 4, Second Paragraph, Last sentence - ~~(Don this is very difficult to catch)~~ They ought to complete the sentence—that the only ~~Sand~~ passing one test shown - Fitz which in fact should have been unacceptable or should have been labeled failing test. Instead of making ~~It evades~~ the point — it's hinting around. ~~One should have been passing, right? But it passed the one that was above and in fact that can't be. (DEH — and they didn't say that.)~~

Page 4, Third Paragraph, Last sentence - States, "a review of all data indicates that about 25% of the cohesive soil test results fall within this area." The last paragraph, last sentence on Page 4 states, "about 40% of all cohesive soil test results ~~were~~ would plot in this area". Are we saying 75%and 60% respectively of our data is outside these areas. Therefore, it's overwhelming proof that it's invalid data?

AR

It would seem to me if they were going to really make a strong point, they should reverse the numbers and ~~AVAA~~ about 75% fell out and therefore must be invalid.

Item 6, Second Paragraph - ~~The real question is, does this get down to this could be one of the real causes for our settlement. It would seem just to have these questions, It would seem~~ since we know It would seem that they could carry this paragraph a little further and provide the end result error that could exist because of this. An then I could read it and say, ~~that~~ It weighs this much into the cause of the excessive settlement."

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Page 5, Item 6, Third Paragraph, Last sentence - States, "in most cases were the test result plots outside the acceptable zone defined in Section 5, the difference between nuclear and sand cone methods would not have made the test results acceptable had a sand cone method been used. Does this really confirm our poor compaction?

Page 5, Item 7 has "comparing" spelling "compring"

Page 6, First Paragraph states in part 364 tests "~~whost~~" "whose" should be "~~whoses~~" -where

~~I don't like their summary.~~ In their summary there are five major faults. They are either ~~eight~~ anamolies or nonconformances. They are unexplained anamolies or they are five major nonconformances. If you find fault with something it means that you are placing the blame there. But these look like nonconformances to me, so they might as well call them what ~~where~~ they are.

Page 6, Item 8, Last Paragraph. It must be concluded that these test results are suspect and should not be used alone for acceptance of plant area fill. So that is their ~~this~~ conclusion too. The next thing is somewhat editorial therefore other means have been established. I don't think that belongs in this. It has nothing to do with the review of these reports. It might be a follow-up action because of our other conclusion. It really doesn't have any subsistence — this is the wrong place for that. I think we ought to just drop that sentence.

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One last thing - for a final conclusion is this enough information that you can say, despite the absolute accuracy of this thing, is there enough information unexplained or what have you here, to conclude that we really did have placements that did not meet compaction criteria. All the stuff we did on K-T and all that kind of stuff — the bottom line is still that we now know that soil was not placed compacted to the specification report. Our testing not only failed to catch that, our testing led us to believe what we had for the duration wws that the stuff was really good. It seems to me that is the root cause of the Diesel Generator falling.

7-5-79

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MIDLAND UNITS 1 & 2
JOB NO. 7220

REVIEW OF U.S. TESTING
FIELD AND LABORATORY CONSTRUCTION
TEST DATA ON SOILS USED AS FILL

BECHTEL ASSOCIATES PROFESSIONAL CORPORATION
JUNE 1979

General Comment: Comparisons of field tests to test pits may make the results and conclusions of this report more believeable. (Similar to what was done with the grade beam problem) suggest this be incorporated in this report.

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REVIEW OF U. S. TESTING FIELD AND LABORATORY CONSTRUCTION TEST DATA ON SOILS USED AS FILL

This review of the quality control tests of the earth fill at the Midland Site was made as a result of settlement of the fill supported diesel generator building in excess of that predicted. Soil samples obtained in borings indicated that soil conditions beneath the plant structures are not compatible with the quality of fill that would be expected based on the results of the control tests made by U. S. Testing Company. All fill was accepted as it was being placed based on the results of the field tests preformed by U. S. Testing Company.

The review showed many discrepancies in the test results as outlined in the following paragraphs. Review comments are based on the requirements of the technical specifications for fill placement and the subcontract entered into by U. S. Testing Company.

who's responsible for flagging when Proctor is Required?

1. Use of Laboratory Test Compaction Curves

Table 9-1 of specification 7220-C-208, Page 14B required one field density and moisture content test be taken for each 500 cubic yards of fill placed. It also required one compaction, grain size, and specific gravity for each 10,000 cubic yards of material. This gives a ratio of 20 field density tests to 1 laboratory compaction test. Although 20:1 is not a strict upper limit, it is a guideline; should density tests be taken more frequently than one per 500 cubic yards of fill the ratio could be higher. The actual ratio is shown in Table A attached. In fact, some of the laboratory compaction tests were used to determine percent compaction for several hundred field density tests taken over a period exceeding two years. Even though no time requirements for the period of use of laboratory tests are specified, it is unlikely that any borrow source in this area would be of such uniform character that such extended use of a compaction curve, truly representative of a large quantity of material, would be applicable. Listed below are selected laboratory test data results indicating the wide range of soil properties that were reported. Such a wide range is typical for soils of the kind used in the fill making prediction of maximum density, based on visual inspection extremely difficult if not impossible without testing.

Is this an Air tight case?

TEST	MIN. DENSITY (lbs/Ft^3)	MAX. DENSITY (lbs/ft^3)	OPT. MOISTURE (percent)
*BMP269		127.3	10
*BMP278		117.0	15.2
*BMP279		140.8	5.7
**RD24	100.9	119.2	
**RD55	90.2	109.7	
**RD61	109.3	125.3	

*BMP refers to proctor type test.
**RD refers to relative density test run by dry method.

if so, why ~~does~~ did spec only Require A 20:1 Ratio?

2. Questionable Retests

A field density test that fails to meet standards dictated by the selected laboratory test data must normally be cleared by another field test made in the same area after corrective action has been taken. In the procedure adopted by U. S. Testing Company, this test result would be compared to the appropriate laboratory compaction curve. Bechtel QC determined which "failing" tests had been cleared by subsequent retest.

Of the 668 "failing" tests which were marked "cleared" by another test, in over 10% (72 tests) of the results, the clearing of the "failed" density test was apparently resolved by using another laboratory compaction curve with either lower maximum density, which resulted in the percent compaction being increased sufficiently, or different optimum moisture content which caused the fill to meet the requirements of the specification. The possibility exists that soil was removed after a "failing" test and replaced by different material, but the records do not indicate this. In other cases, tests labeled "failed" were incorrectly cleared though the same laboratory standard was referenced. For example, in some cases retests to clear a "failed" test were not taken in the same area or at the approximate same elevation. More than 40 retests were over 20 feet from the "failed" test location (as recorded in the test reports) and some were over 200 feet from the original test location. In general, if after a "failing" test the whole area is reworked, the retest location is not too critical assuming that the correct laboratory compaction curve is used for comparison. However, in the plant fill work areas were relatively small, and soil characteristics showed considerable variation necessitating retesting in the immediate vicinity of the "failing" test. Retest should be taken in the lift or soil layer that has been reworked. Almost 50 retests were taken at different elevations, some up to 10 ft. from the "failed" test. It should be noted that Bechtel field personnel gave the locations for retesting. This was not a U. S. Testing responsibility. Two retests were dated prior to the time the original test "failed". Over 130 "failing" tests were marked as "non Q" and never recorded cleared by a passing test.

We should make same statement in 1. who is responsib

Table B is a compilation of notes relative to questionable clearing of failed tests.

3. Theoretically Impossible Test Results

Soils cannot be more than 100 percent saturated; therefore, all field density test data points, when plotted as dry density versus moisture content, must be below the zero air voids curve as defined by the specific gravity of the material. Specifications do not require examination of the zero air voids curve, but it is considered fundamental soil mechanics relative to compaction plots. There are numerous cases in the U. S. Testing Company data where points plot above the zero air voids curve. Figure 1 attached shows a typical laboratory compaction test curve with field test results plotted on it. Many of the field test results plot above the zero air voids curve. Provided the specific gravity is correct this is not possible so that all such points must represent erroneous data.

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The fact that a large number of test results plot above the zero air voids curve tends to make all test results questionable.

Also, referring to Figure 1 it would appear that soil density varied widely. Specifications called for compactive effort results as defined by ASTM D 1557 which is 56,255 ft-lb/ft^3 energy. This was modified to a laboratory test compactive effort of about 20,000 ft-lbs/ft^3 energy, often referred to as Bechtel Modified Proctor (BMP). Laboratory compaction test curves should be related to the same effort as that called for in the field for use in comparing with field density tests to determine percent compaction. According to plots of field data shown on Figure 1, density varied from about 108 lb/ft^3 to about 130 lb/ft^3. It is doubtful that the soil classification or other properties would be similar for such a wide variation in density. It is noted that 100 percent of modified Proctor (ASTM D 1557) which is difficult to obtain, is rated at 56,255 ft-lb/ft^3 energy. The curve plotted on Figure 1 is at about 20,000 ft-lb/ft^3 energy. For comparative purposes it was determined by U. S. Testing in 1974 that 100 percent of specified effort (20,000 ft-lb/ft^3) is approximately equal to 95 percent of the maximum density as determined by ASTM D 1557 (56,255 ft-lb/ft^3) Reference Figure 8.

Please clarify and amplify

what does this mean

4. Repeated use of Questionable Laboratory Test Data

Some laboratory compaction test data were used repeatedly even though they continued to show suspect field test results. This could be indicative of questionable laboratory data or the fact that soil was not being placed or compacted according to specifications. Either case is a cause for concern. Subcontract 7220-C-208 Exhibit C, Page 17 of 47 No. 2 states "You (U.S. Testing) are to immediately report data that indicates material that does not comply to specifications or procedures."

Several specific gravity calculations are in error, such as for BMP 273 and 274. In the case of BMP 273, the zero air voids curve passes through the laboratory compaction curve. In another example, BMP 297, the laboratory compaction curve is invalid due to calculation errors, yet was referenced by field density tests 22 times.

How do we know this was not done? were these result reviewed by Bechtel

Table 6 is a compilation of notes relative to questionable test data.

5. Limits of Accuracy and Acceptability for Test Data

Figures 1 through 7 attached will be referenced in discussing limits of accuracy of acceptability for field test results as compared to laboratory test data. The figures show plots of compaction data for BMP 278 which are typical of all test results.

Specified laboratory compactive effort was 20,000 ft-lbs/ft^3 and field compaction effort was originally specified at 56,255 ft-lbs/ft^3 but was changed by Revision 5, dated 7/8/75, specification 7220-C-210, Section 13.7, Page 57 to also be equal to about 20,000 ft-lbs/ft^3.

The specified 20,000 ft-lbs/ft^3 effort establishes a compaction curve relating moisture and density for a specific soil. Moisture was specified for field placed fill to be within ± 2 percent of optimum moisture as determined by this effort. Density was specified to be greater than 95 percent of the maximum density. As compactive effort is increased in the laboratory test, maximum density will be increased and optimum moisture content will decrease. This change can only occur in the field to the extent that the field moisture content will permit it. Once field compaction is such that the fill density is significantly higher than about 105 percent of maximum, the specified tolerance from optimum moisture content in the laboratory compaction test may no longer be applicable for field control. A + 2 percent numerical value of moisture content acceptable at the specified compactive effort would be too wet at a higher effort since the zero air voids curve defines the absolute maximum that can be achieved, indicating that higher densities for that soil are impossible. Therefore, if the record shows high densities for such material, the data are in error. This was apparently overlooked.

Plots of field data for compaction test BMP 278 are shown on Figures 1 through 6. The title of each figure gives the assumptions made in plotting data for the figure. In comparing figures 3 and 4 it is seen that a majority of field tests were made using the nuclear device. The two test results shown on Figure 4 for the sand cone method indicates one test result on each side of the zero air voids curve. The one falling above the zero air voids curve (shown on Figure 4) is designated by U. S. Testing Company as the only passing sand cone test (shown on Figure 6).

For a field test result to be valid as well as "Passing" it must fall within a well defined area on the plot containing the laboratory compaction curve. This area or window of acceptability is shown for a hypothetical compaction curve on Figure 7a that would meet requirements of Specification 7220-C-210. It is defined by horizontal lines at 95 percent and 105 percent of specified density, vertical lines through ± 2 percent of optimum moisture content, and a line parallel to the zero voids line indicating saturation about half way between the compaction curve and 100 percent saturation (zero air voids curve). The practical upper limit of 105 percent of specified density is not defined in the specifications. It was arbitrarily chosen as numbers greater than this give increasingly invalid comparisons between field test results and the specified laboratory compaction test curve. Therefore, if all data points fall within the defined window there would be no reason to assume that they are wrong. However, when many data points fall outside the designated area there is something wrong with the information and then all data points become suspect. A review of all data indicates that about 25 percent of the cohesive soil test results fall within this area.

Figure 7B shows an area where field test results would be acceptable, in theory even though not in strict accordance with the specifications. Figure 7B was arrived at by expanding Figure 7a to include test results up to a compactive effort related to ASTM D 1557 (56,255 ft-lb/ft^3) which is considered to be a practical upper limit. About 40 percent of all cohesive soil test results would plot in this area.

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6. <u>Accuracy of Test Equipment</u>

Almost all (over 95%) field density tests on cohesive soils were made using the Nuclear Density device. Specification 7220-C-210 section 12.4.2 page 42 indicates this to be acceptable for moisture content determination provided that the results are compatible with those obtained by ASTM D 2216. Similarly, section 12.4.4 says density determined by the nuclear device is acceptable when results are compatible with density as determined by ASTM D 1556.

In a letter from U. S. Testing to Bechtel (dated May 30, 1974), the average deviation of the nucelar device from oven-dry moistures was +.12% for a set of 30 tests. However, the stand error of estimate is 1.8% for the data with the range of differences being from - 3.2% to +3.9%. Thus, accuracy of the nuclear device is questionable, and could translate into errors of about ± 4 pcf in the dry density calculation. (It should be noted that errors in the moisture content tend to shift the position of test results on a moisture density plot approximately parallel to the zero air voids curve, assuming the in-place wet density is correct, and thus do not explain the large number of points which plot outside the zero air voids. Compare Figures 1 and 9).

Even with the range of possible error for nuclear-determined moisture values shown above, it appears that the controlling factors resulting in erroneously reported degrees of compaction were selection of the appropriate laboratory test curve as well as erroneous test data (revealed by points plotted right of the zero air voids curve indicating specific gravities in excess of 2.80, 2.90, and even 3.00) rather than the type of field test method used. In most cases where the test result plots outside the acceptable zone defined in section 5, the difference between nuclear and sand cone methods would not have made the test result acceptable had a sand cone method been used.

7. <u>Relative Density Tests</u>

Cases were noted where densities in material classified on the data sheet as zone 3 (sand) were compared to the maximum densities in proctor type tests and other cases where densities in clay soils were compared to the maximum density in relative density tests. An error must exist in the record in such cases either in the classification of the soil on the data sheet or in compring field ~~test~~ results to inappropriate laboratory test data. In general, it appears that relative density tests were used in controlling density of sand fill. There were a significant number of arithmetic errors on calculation sheets even though there are signatures on the sheets indicating they had been checked. Over 100 errors were found in calculations, of relative density from 8/15/75 through 12/78 (not all of these errors change the acceptability of the test results).

Fig 1. shows many tests outside the 2.75 curve while figure 9 shows none - therefore this conclusion is not supported by these two figures.
(The real accuracy of the instruments should be determined.

ASTM D 2049 section 7.1.2 Wet Method states: "Note 2 - While the dry method is preferred from the standpoint of securing results in a shorter period of time, the highest maximum density is obtained for some soils in a saturated state. At the beginning of a laboratory test program, or when a radical change of materials occurs, the maximum density test should be performed on both wet and dry soil to determine which method results in the higher maximum density. If the wet method produces higher maximum densities (in excess of one percent) it shall be followed in succeeding tests." An example of wet and dry relative density is shown on Figure 10. U. S. Testing Company apparently did not do this frequently enough, or on a broad enough range of non-cohesive soil types. As a consequence many field density test results exceed 100 percent of maximum dry laboratory ralative density. As an example, for laboratory test RD55 a total of 566 field tests were made. Of this total, 364 tests whosed greater than 100 percent compaction. The highest relative density found was 142.2 percent with the majority of tests over 100 percent falling in the range of 100 percent to about 130 percent. Since the difference in maximum density between wet and dry methods is about 4 to 5 lbs/c. ft. (based on recent data) any test result greater than about 115 percent (based on the dry method) is suspect.

Even if the wet laboratory test method data were available for all sands, it appears an unacceptably high number of field test results would greatly exceed 105 percent relative density even based on the wet maximum.

8. Summary

In summary, there are five major faults contained in the Midland Compacted Fill Density Test Reports as follows:

1. erroneous field density test data.
2. incorrect soil identification
3. incorrect (or questionable) laboratory test data.
4. calculation errors
5. improper or incomplete clearing of "failed" tests.

Items 4 and 5 represent existing faults in the data which could be corrected. However, as a result of items 1 through 3, there is no rational means of determining which test results are valid and which are not. Since more than one half of the test results for relative density and percent compaction fall outside the possible theoretical comparison limits, it must be concluded that these test results are suspect and should not be used alone for acceptance of plant area fill. Therefore, other means of testing have been established and employed to determine if the fill in any given area is acceptable.

TABLE A

Listing of All Classifications Referenced in Plant Area Fill Soil Test Records Which were Used for 20 or More Field Density Tests

Classification	No. of Tests
B200	90
B251	31
B252	22
B254	42
B255	57
B260	68
B261	36
B262	165
B269	227
B270	226
B271	141
B274	37
B276	21
B277	158
B278	82
B297	22
R015	20
R016	61
R024	248
R030	54
R035	59
R038	39
R039	28
R040	35
R041	69
R042	103
R043	48
R044	71
R045	43
R049	63
R054	118
R055	566
R059	65
R061	589
R063	42
R065	59

Note: Spec. 7220-C-208 gives a ratio of approximately 20 field tests to each laboratory test.

TABLE B

Notes on Questionable Clearing of Failed Tests

1. Test number MD 245 fails due to high moisture. Cleared by MD 246 which references a proctor with higher optimum moisture content (OMC) such that the ±2% of optimum requirement is met.

2. MD 205 fails with moisture content 6% above the OMC. Cleared by MD 215, which references a relative density lab standard, and is itself still 6% away from the OMC of the proctor referenced by MD 205.

3. MD 223 fails because of high moisture. Cleared by MD 228 which has actually a higher moisture content and lower density, but references a different proctor; the retest passes and clears the failure.

4. Both MD 844 and 886 fail because of high moisture and low density. They are cleared by MD 888 which references a new proctor with lower maximum density and higher OMC than the first.

5. MD 251 fails due to moisture being too high. Cleared by MD 253 which uses a higher OMC proctor.

6. MD 668 clears MDR 634, but the two tests show no correspondence in location, moisture, density, or lab standard.

7. MD 771 failed, being too dry. Cleared by MD 782, which has almost identical moisture content and dry density but uses a new BMP with lower optimum moisture.

8. MD 2384 clears MD 2342, referencing a different proctor with an OMC which fits the in-situ conditions. However, the dry density of MD 2384 is way too high to fit the original soil classification, and in addition, it falls outside of the zero air voids curve for the classification which it has been changed to.

9. MD 556 clears MD 554 by using a BMP with lower moisture requirements. The field densities differ by 24 pcf and would seem to be different material.

10. MD 558 clears MD 555 but has too high a density to be the same soil as MD 555. It also uses a different proctor.

11. MD 566 and 568, classified as BMP 262 cohesive soils, are cleared by MD 569 which is classified as RD 33 and has totally different soil properties than the two failures.

12. MD 1317, 18, 19 and 20 fail and are all cleared by MD 1477 taken over 5 weeks later. There is poor correspondence in the soil properties and the proctor is different from failing to passing test.

13. MD 2965 clears MD 2963 with a different proctor through the test results would have been passing with the original BMP.

14. MD 1388, classified as BMP 278, is cleared by MD 1461, classified as RD 55.

15. MD. 170, classified as RD 24 is cleared by MD 173, classified as BMP 234.

16. MDR 287 fails with a relative density of 77%. Cleared by MDR 291 which has .1 pcf lower density but arbitrarily rounds up the relative density to 80%; it passes and clears the failure.

17. In all of the following field density tests on sand, the passing test has approximately the same or lower density than the failures, but references a lower maximum density RD lab standard:

MDR 343	clears	MDR 339
MDR 514	clears	MDR 507
MDR 513	clears	MDR 508
MDR 515	clears	MDR 509
MDR 516	clears	MDR 510
MDR 522A	clears	MDR 521
MDR 558	clears	MDR 556, 557
MDR 480	clears	MDR 473
MDR 555	clears	MDR 525, 527, 534
MDR 533	clears	MDR 526, 530, 531

18. MD 2384 clears MD 2342, but is at 7' lower elevation.

19. MD 123 clears MD 122, but is at 10.5' lower elevation.

20. MD 149 clears MD 142, but is at 10' higher elevation.

21. MD 1694 clears MD 1693 but is 43' away from the site of the first test.

22. MD 3114 clears MD 3102, but the two tests are 68' apart.

23. MD 186 clears MD 183 though it is 110' away.

24. MD 1209 clears MD 1207 and MD 1205, yet is 183 ft. away from the failures.

25. MD 1097, dated August 4, 1977, cleared by MD 1048 dated July 16, 1977.

Note: This table gives typical observations and is not meant to be all-inclusive.

TABLE C

Notes on Questionable Test Data

1. The first field density test to reference RD 24 (5/75) has a relative density of 170.6%. The standard continued to be used, however, with relative densities greater than 100% occurring repeatedly.

2. Similarly for RD 30, the first two tests (9/75) have 114% and 122% relative densities, yet the standard was used for 10 months, 54 tests, with 52% of the results over 100%.

3. During the first two weeks of use (7/76), RD 41 was referenced 22 times with 12 tests over 100% relative density (6 tests over 110% and 3 over 120%). The standard was used for 5 months, however, with over 40% of the results over 100%.

4. The first test using RD 55 (8/76) has a relative density of 119%, with the field test being made the same day as the standard and, thus, assumedly the same material. These results would throw doubt on the lab standard, yet it was used for two full years and 566 tests, with 64% of the results over 100% relative density.

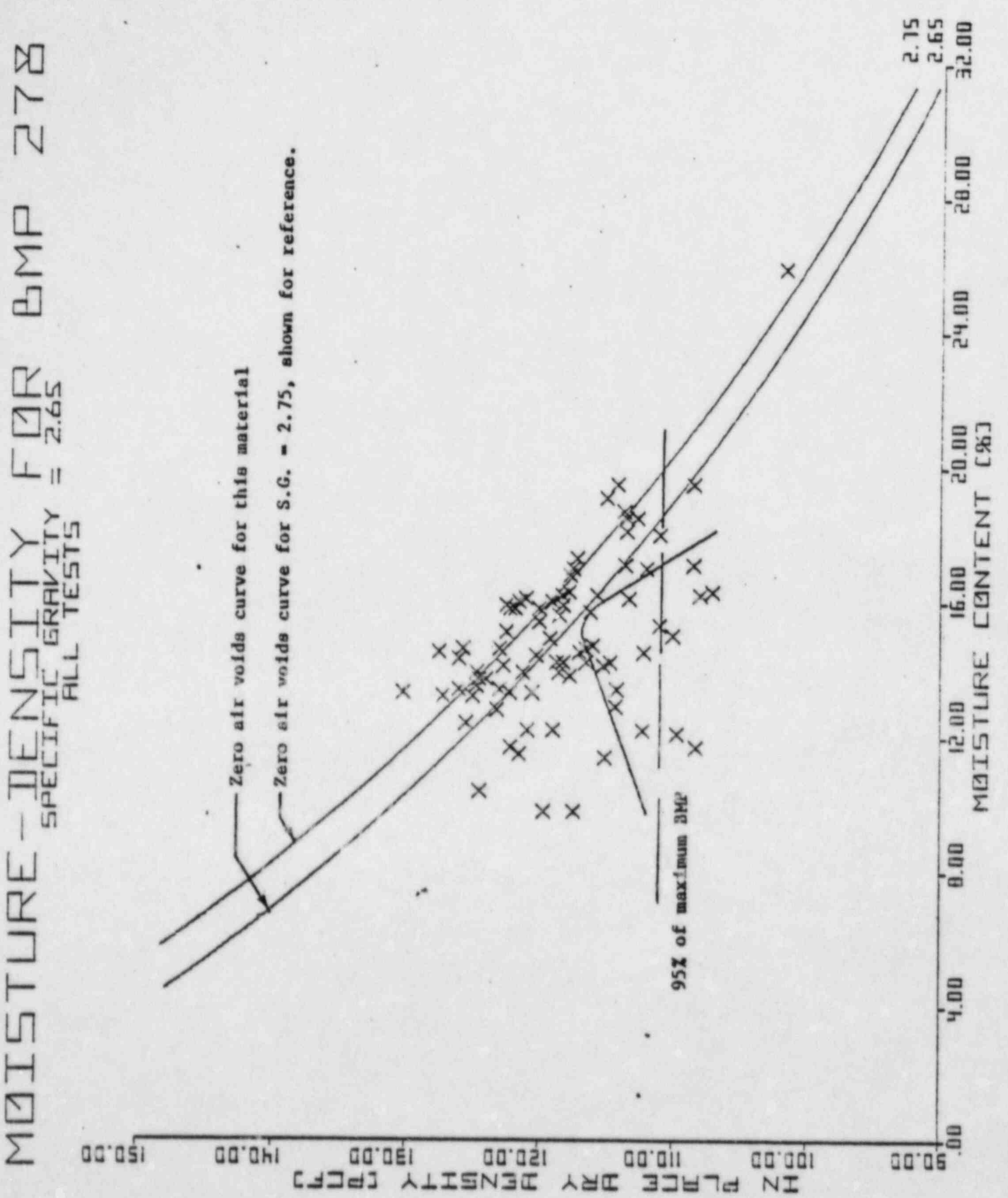
5. Even high density structural backfill standards such as RD 61 (maximum density of 125.3 pcf), used 593 times, show over 25% of the tests having greater than 100% relative density.

6. The first seven tests referencing BMP 269 (scattered over a two month period around 7/76) all fall outside the zero air voids curve. This classification was used for 1 1/2 years, referenced 227 times.

7. The first two tests referencing BMP 270 (7/76) fall 6 pcf above the zero air voids curve. Continued use of this proctor for over 2 years resulted in 226 tests with 82 outside the theoretical maximum.

8. For the first month (4/77) all BMP 278 tests fell on or outside the zero air voids curve. For the next month, over half the tests did the same, or have greater than 105% compaction. The standard was used over half a year, with 43 out of a total of 82 tests outside the zero air voids curve.

Note: This table gives typical observations and is not meant to be all-inclusive.



MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
ALL TESTS
Zero air voids curve for this material
Zero air voids curve for S.G. = 2.75, shown for reference.
95% of maximum BMP
2.75
2.65
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
MØISTURE CØNTENT (%)
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00

FIGURE 1

MOISTURE-DENSITY FOR BMP 278

SPECIFIC GRAVITY = 2.65
PASSING TESTS ONLY*

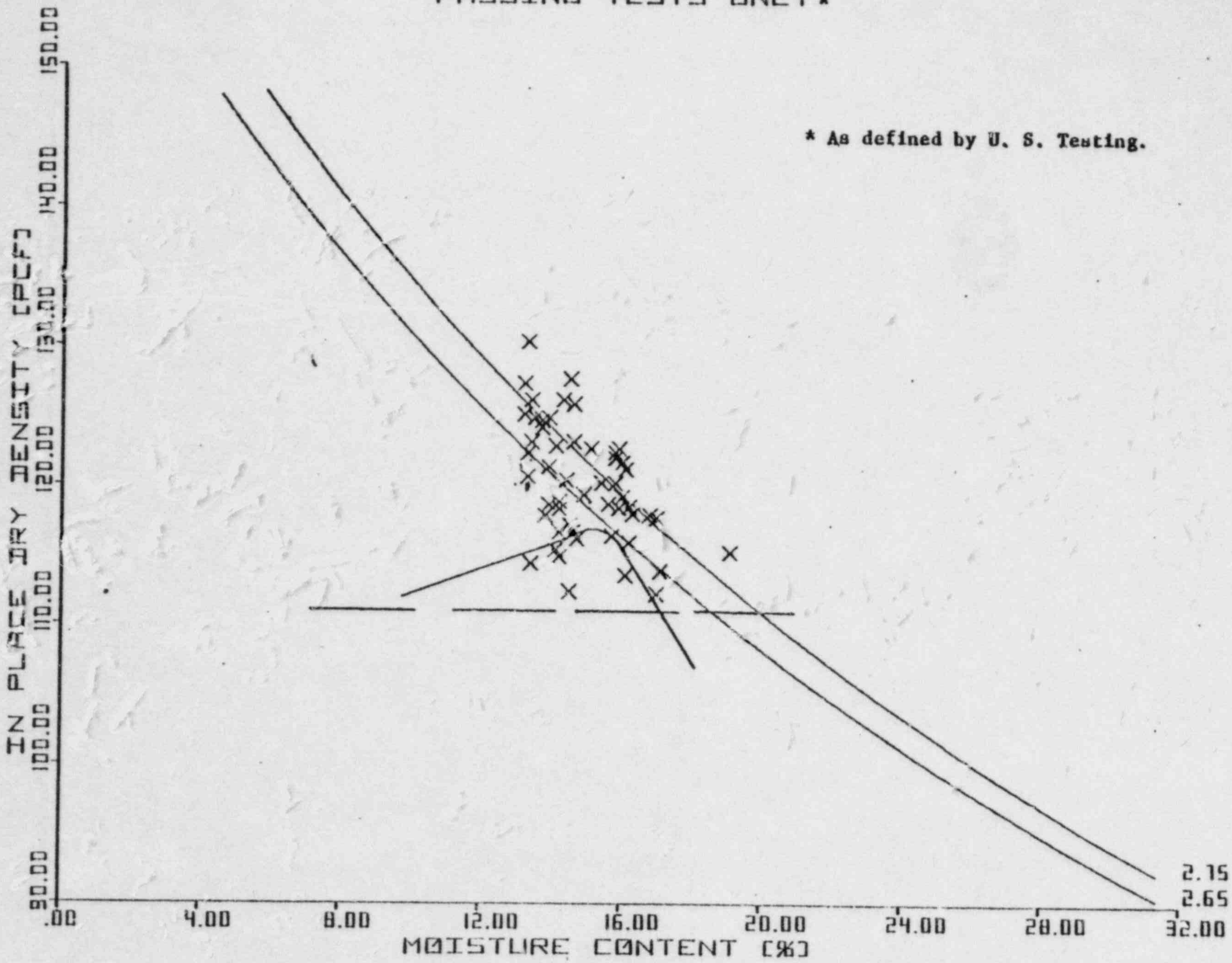


FIGURE 2

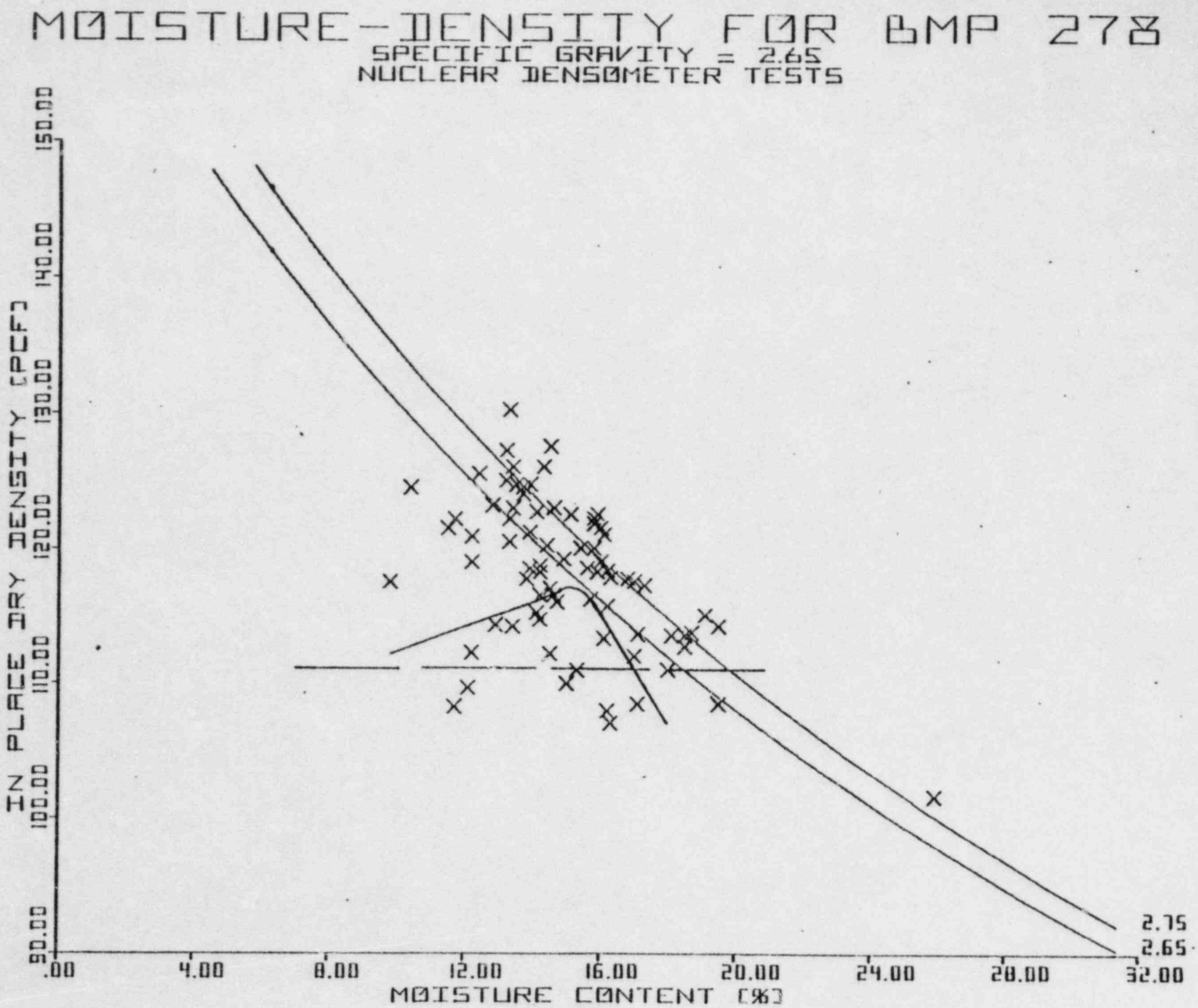

MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
NUCLEAR DENSØMETER TESTS
IN PLACE DRY DENSITY [PCF]
MØISTURE CONTENT [%]
90.00
100.00
110.00
120.00
130.00
140.00
150.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
2.75
2.65

FIGURE 3

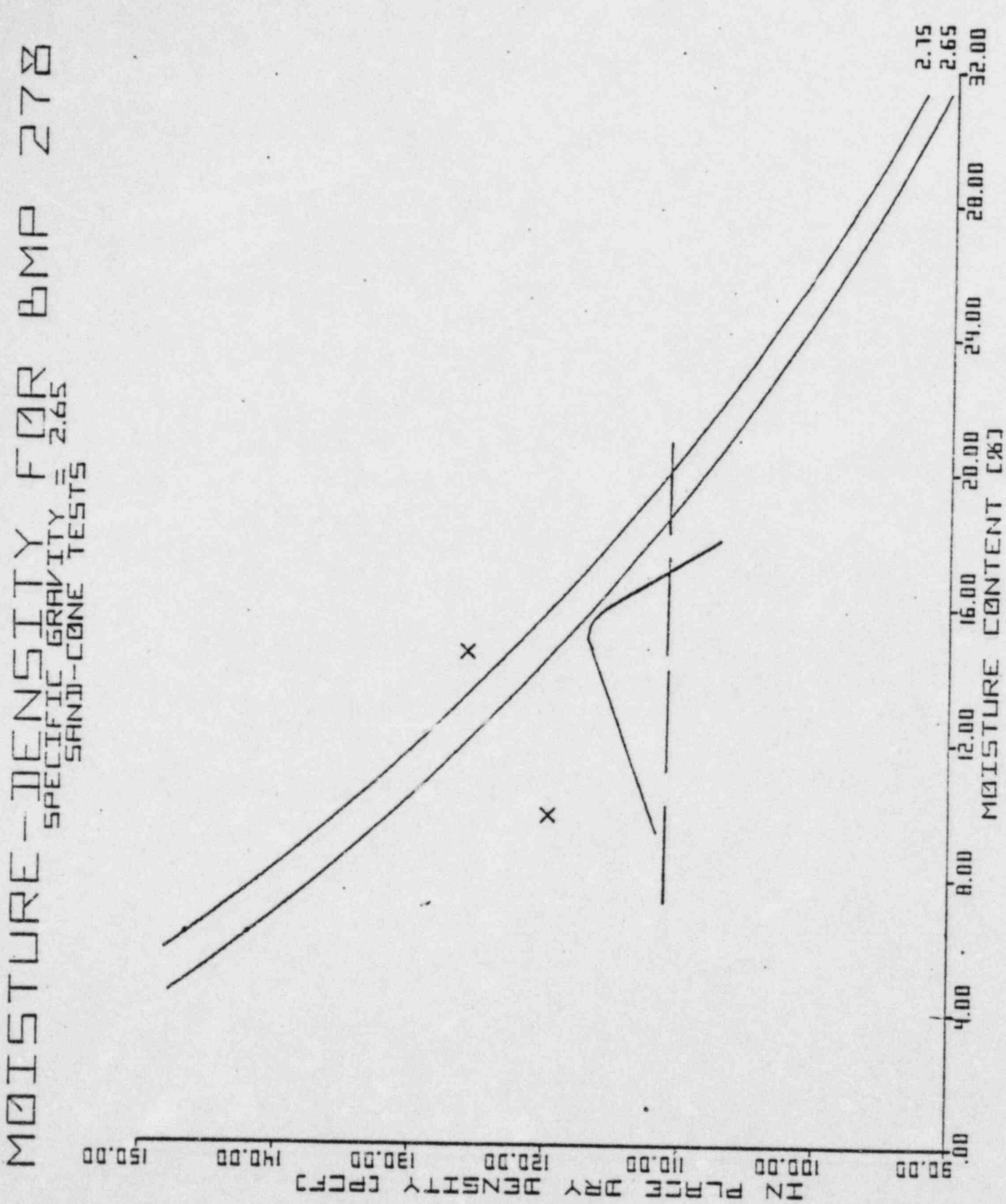

MOISTURE-DENSITY FOR BMP 27B
SPECIFIC GRAVITY = 2.65
SAND-CONE TESTS
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MOISTURE CONTENT (%)
2.15
2.65

FIGURE 4

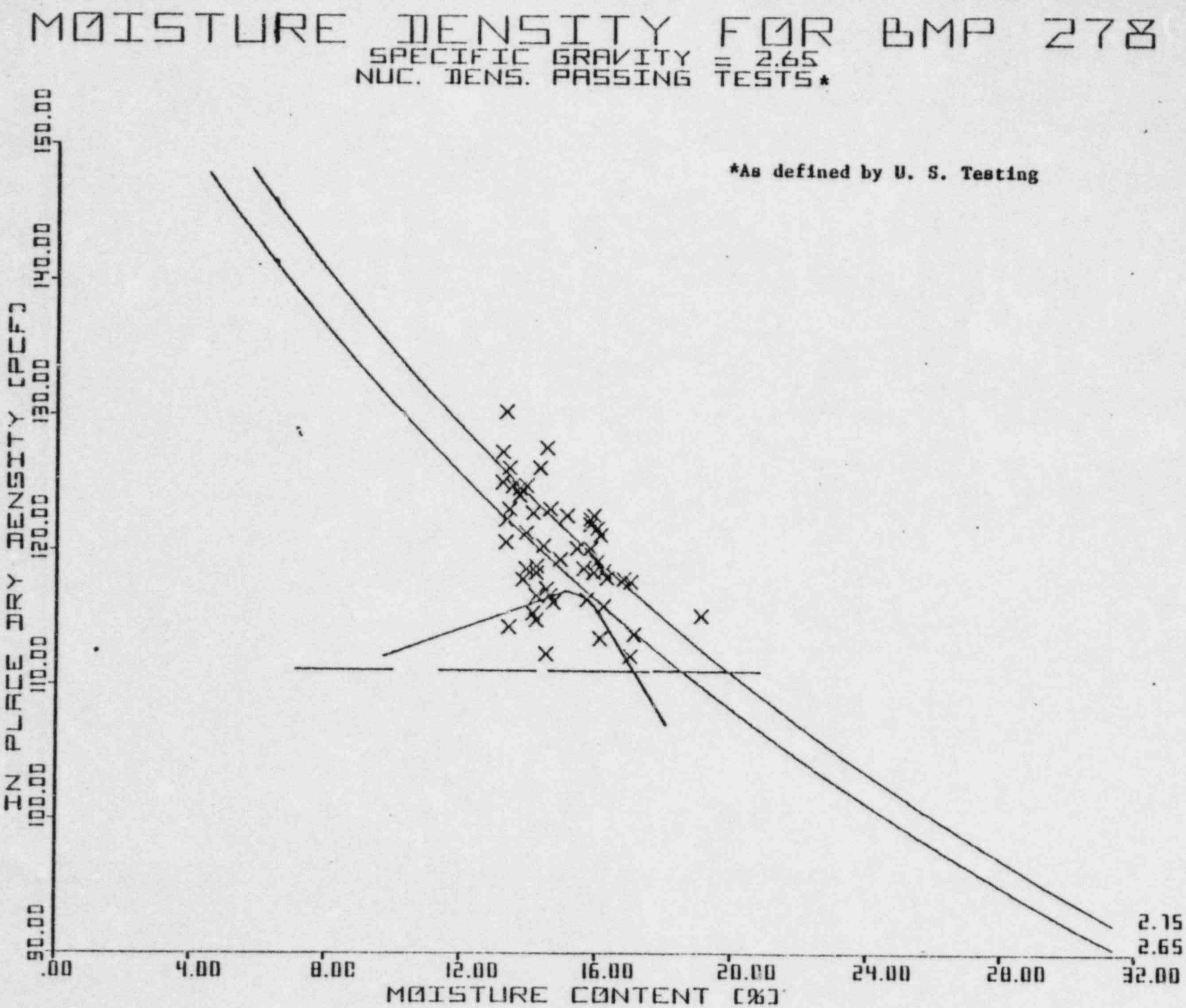

MØISTURE DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
NUC. DENS. PASSING TESTS*
*As defined by U. S. Testing
IN PLACE DRY DENSITY [PCF]
150.00
140.00
130.00
120.00
110.00
100.00
90.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MØISTURE CØNTENT [%]
2.15
2.65

FIGURE 5

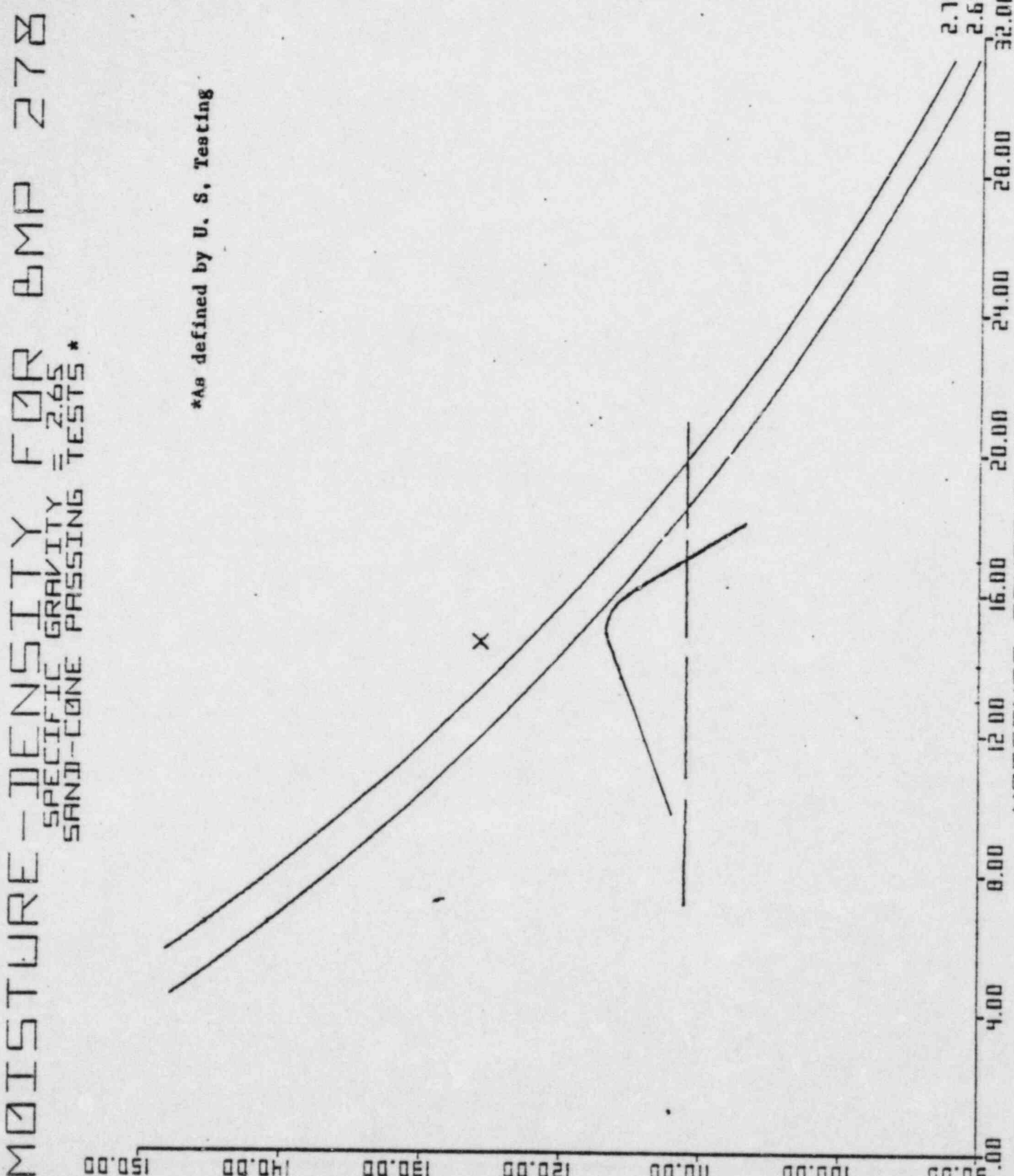

MOISTURE-DENSITY FOR BMP 278
SPECIFIC GRAVITY = 2.65
SAND-CONE PASSING TESTS *
*As defined by U. S. Testing
IN PLACE DRY DENSITY (PCF)
150.00
140.00
130.00
120.00
110.00
100.00
90.00
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
MOISTURE CONTENT (%)
2.75
2.65

FIGURE 6

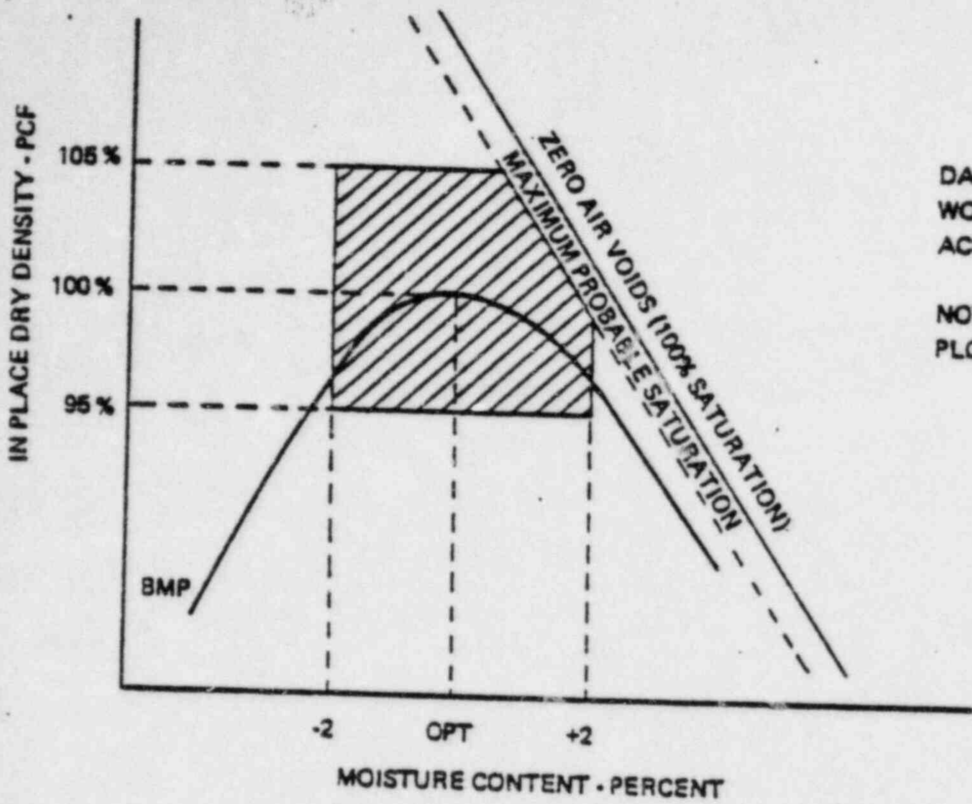


FIGURE 7-A-

DATA POINTS THAT PLOT IN SHADED AREA WOULD BE GENERALLY ACCEPTABLE ACCORDING TO SPECIFICATIONS

NOTE: ABOUT 25% OF ALL FIELD DATA PLOTS IN THE SHADED AREA

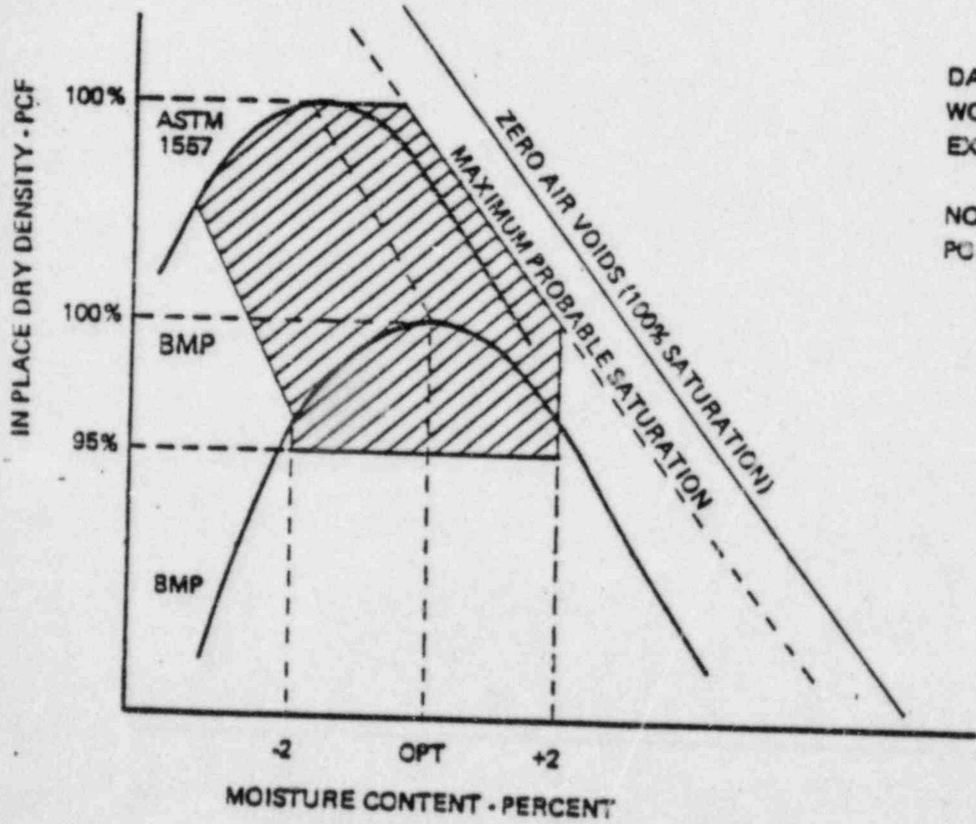


FIGURE 7-B-

DATA POINTS THAT PLOT IN SHADED AREA WOULD BE ACCEPTABLE REGARDLESS OF EXACT SPECIFICATION WORDING

NOTE: ABOUT 40% OF ALL FIELD DATA POINTS PLOT IN THE SHADED AREA

FIGURE 7: WINDOWS OF ACCEPTABILITY (A) BASED ON BMP SPECIFICATION (B) REGARDLESS OF EXACT WORDING OF SPECIFICATION

UNITED STATES TESTING CO., INC.
Graph Representation of Three
Proctor Method Comparisons

June 13, 1974
By: Peter Wang

Note: ( ) added by Bechtel

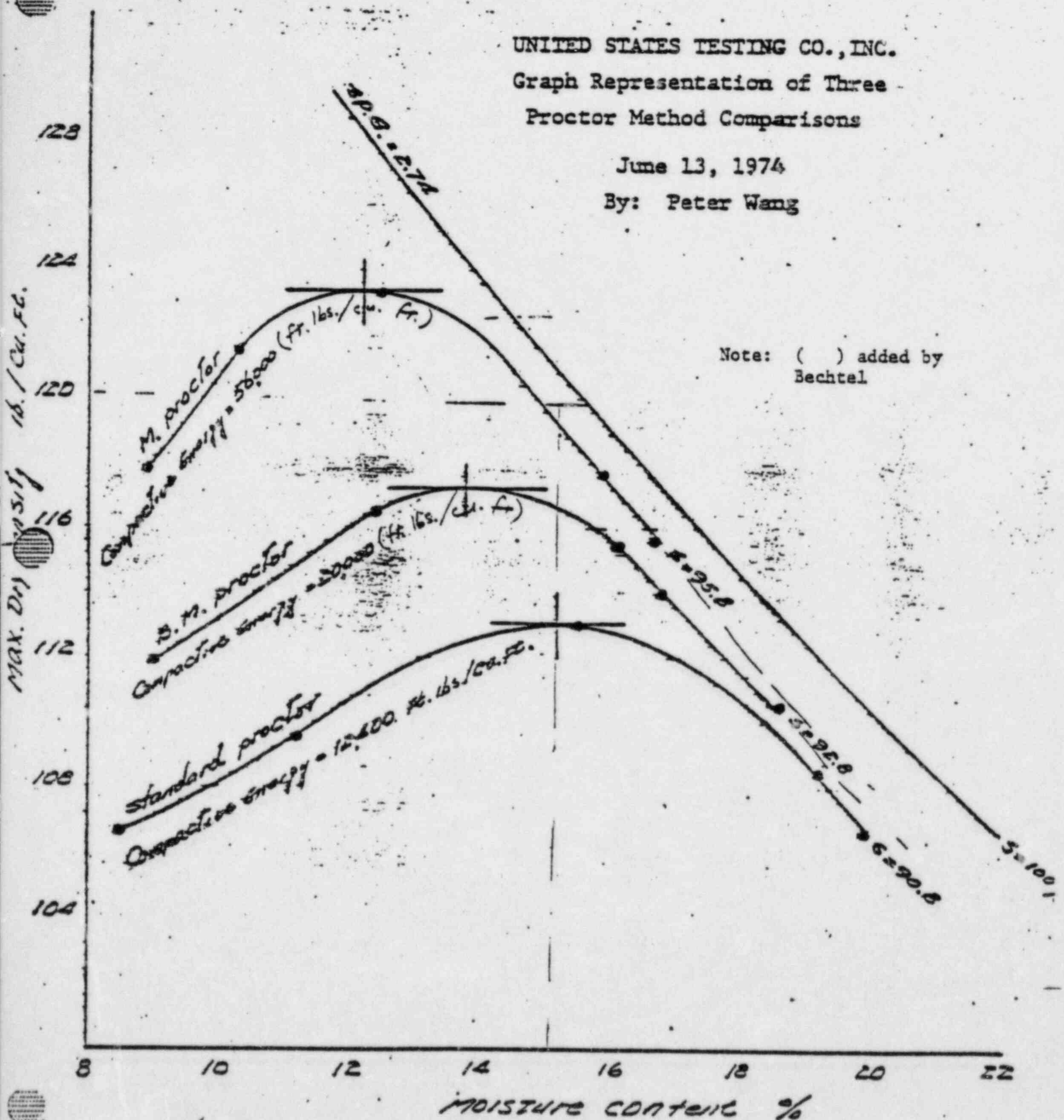
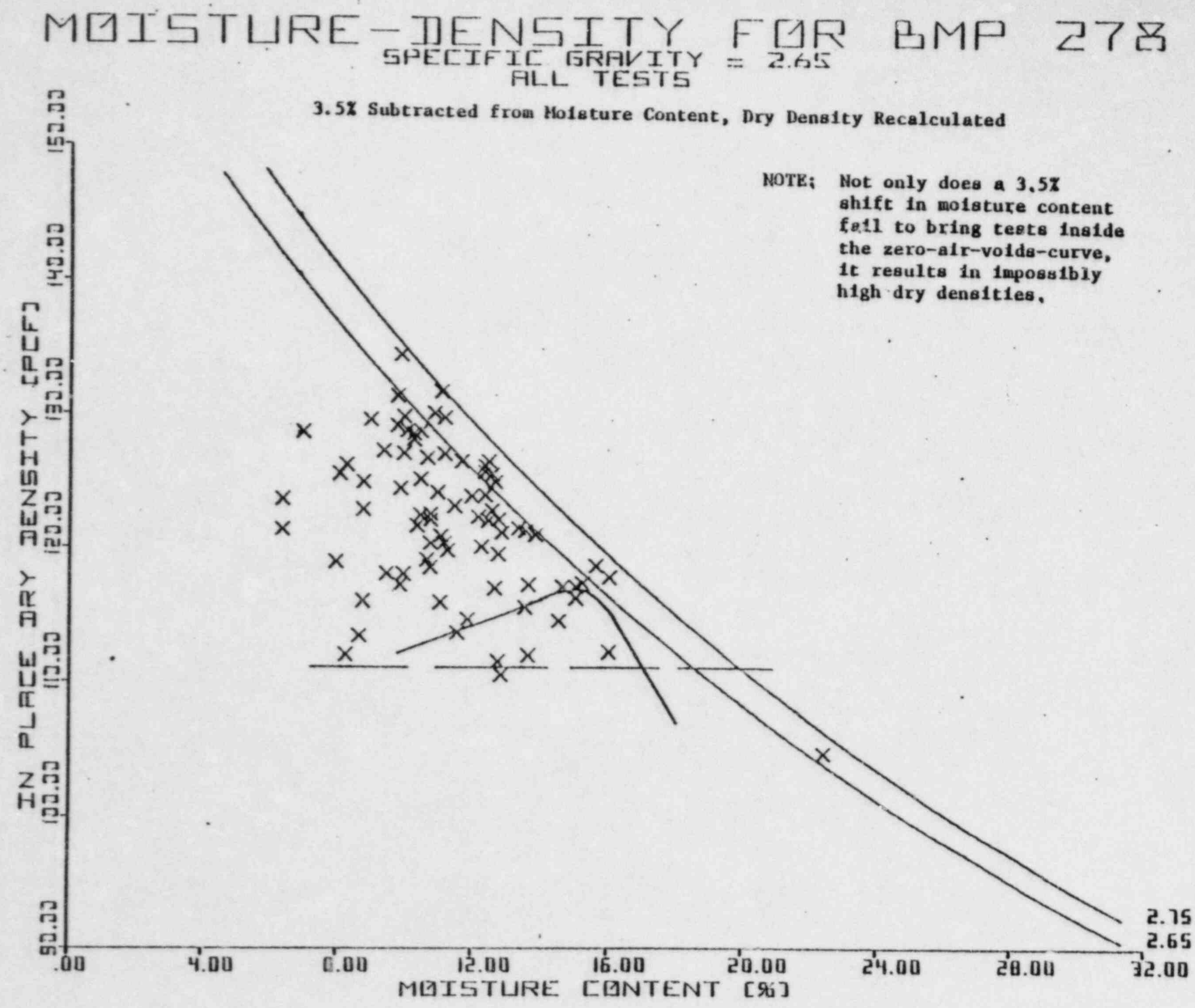
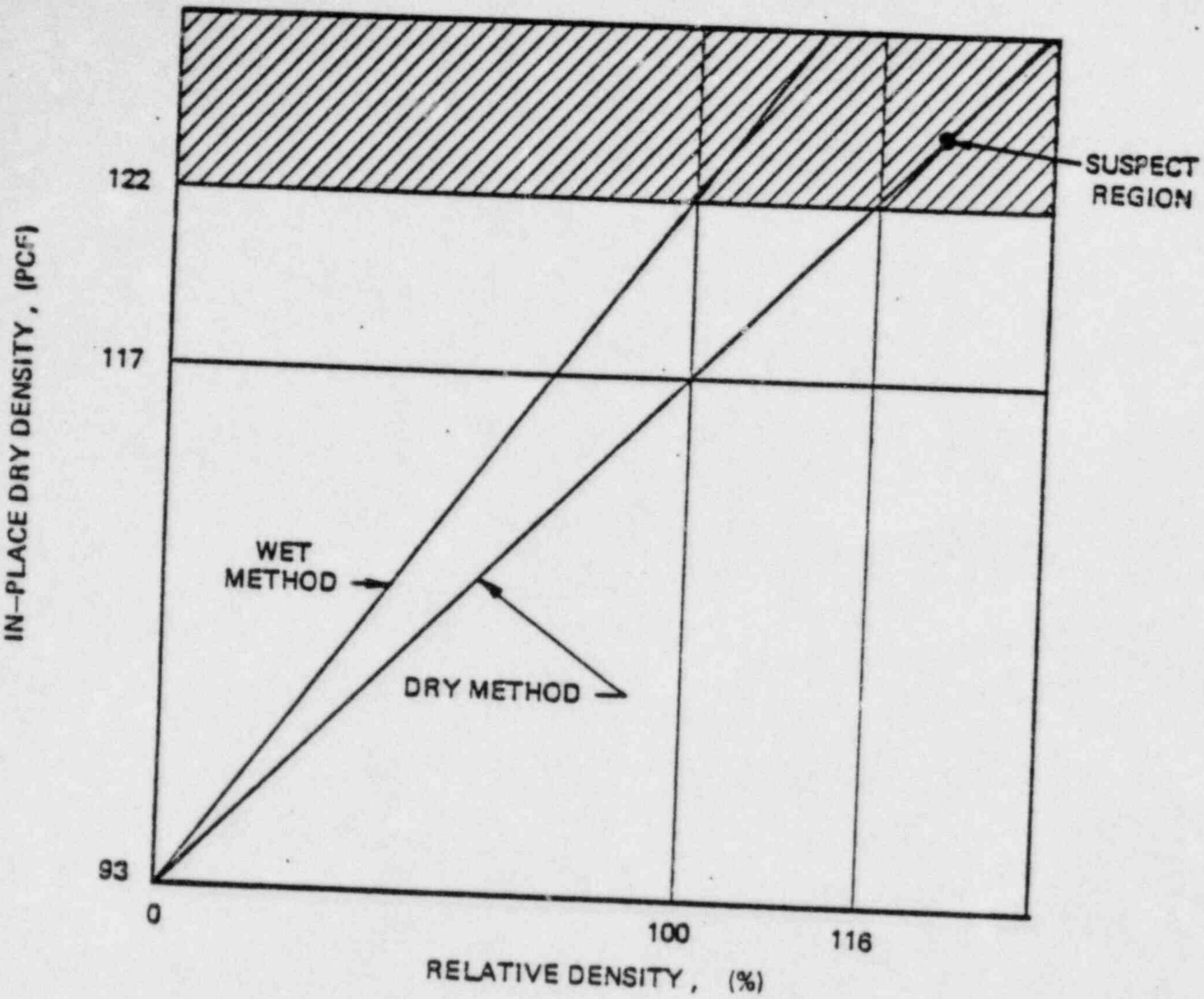


FIGURE 8

FIGURE 9


MØISTURE-DENSITY FØR BMP 278
SPECIFIC GRAVITY = 2.65
ALL TESTS
3.5% Subtracted from Moisture Content, Dry Density Recalculated
NOTE; Not only does a 3.5% shift in moisture content fail to bring tests inside the zero-air-voids-curve, it results in impossibly high dry densities.
IN PLACE DRY DENSITY [PCF]
MØISTURE CØNTENT [%]
2.75
2.65
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
90.00
100.00
110.00
120.00
130.00
140.00
150.00

1?4



NOTE: VALUES FOR DRY DENSITY ARE TYPICAL OF A RANDOM FILL SAND. ANY TESTS SHOWING MORE THAN 117% RELATIVE DENSITY WOULD BE SUSPECT IN THIS EXAMPLE. STRUCTURAL SANDS TEND TO SHOW ONLY 2 OR 3 PCF INCREASE IN MAXIMUM DENSITY AND THUS RESULTS AT MUCH LOWER RELATIVE DENSITY WOULD BE SUSPECT, SAY 105 – 110 PERCENT

FIGURE 10

CHANGE IN RELATIVE DENSITY SCALE FROM DRY TO WET METHODS OF OBTAINING MAXIMUM DENSITY , BASED ON RECENT LAB RESULTS

NONCONFORMANCE REPORT

1. Drawing/Part No.	PSK-C-12 & C-109 Q
Rev.	6
2. Item Description	Earthwork Zone 1 & 2
4. Serial Number	NA
5. Purchase Order No.	C-210 Rev. 2
6. Contractor/Location	Canonie Const. Co. South Haven, Michigan
7. Project No.	7220
8. Item Location	Q Listed Dikes
9. Startup System No.	NA
10. QC Field Inspection Plan No.	C-210 18, 28, 24, 31, 32, 17, 33 and 34
11. ASME Code Item	☐ Yes ☒ No
12. Reported by	L. Shively
Date	1/28/74
13. Originated by	Carroll
Date	7-4-74
15. Replacement No.	NA
16. Replacement Serial No.	NA
17. Source	Sub-Contractor
Page	1 of 5
NCR No.	55

25. Disposition Concurrence: 5-10-74; 5/3/74; 5/9/74

18. Routing Instructions: ☐ Route to Field Engineering ☐ Route to Material Supervisor

19. Nonconforming Condition:

Spec. C-210-Rev. 2, section 12.6.1 states in part "The water content during compaction shall not be more than 2 percentage points below optimum moisture content and shall not be more than 2 percentage points above optimum moisture content..."

Contrary to the above, compaction test records indicate that material with out-of-specification moisture content was placed as shown in the following list:

20. ☐ Field Disposition ☒ Field Recommendation/Route to Project Engineering

As per IOM DEDC-104 dated Nov. 7, 1973, the optimum moisture content range was relaxed to 2% dry to 5% wet on zone 2 material in the Bullock Creek area and the other selected areas of the dike as specified by the Bechtel representative. The following data from block 19 is on zone 2 material and within 2% dry to 5% wet of optimum. It is listed separately for project engineering's evaluation to DEDC-104

21. Field Disposition Results:

22. Engineering Disposition

Based on a review of test results listed on pages 2 thru 5 of this NCR and also on various test results submitted as a response to NCR C-26, Engineering concludes that the in-place material is satisfactory. A summary of the results of our evaluation will be forwarded under separate cover.

Engineering recommends proceeding with the Plant Area Fill work, and also recommends to "use-as-is" the in-place material described in this NCR. 5-3-74

23. Engineering Disposition Results:

24. Design Change Required ☒ No ☐ Yes, see attached:

Drawing ____ Rev. ____ DCN ____

Spec. ____ Rev. ____ ADD ____

26. Rejected Material Disposition ☐ Return to Supplier ☐ Scrap

Remarks

27. QC Acceptance: Shively 5/15/74

NONCONFORMANCE REPORT (CONT'D)

NCR NO. 55

TEST NO.	DATE	STATION	ELEV.	MOISTURE CONTENT	OPTIMUM MOISTURE	CURVE NO.	RIGHT OR LEFT ℄	
				North Plant Dike				
WOD 8- 1	9-12-73	7 + 32	610	10.4	7.1	COP 2Q	215' R	No Action Taken
3	9-12-73	8 + 93	610	9.6	12.6	COL 12	215' R	No Action Taken
4	9-14-73	10 + 50	612	6.8	11.8	COB 2	200' L	
5	9-14-73	9 + 50	612	6.6	11.8	COB 2	250' L	
8	9-18-73	9 + 19	610	4.5	7.1	COP 2Q	238' L	
9	9-18-73	6 + 52	610	13.0	10.3	COD-1	92' R	
12	9-25-73	4 + 09	608	10.0	7.4	COL-11	80' R	Reworked-No Retest
13	9-25-73	6 + 08	609	14.2	9.4	COD-8	105' R	Reworked-No Retest
14	9-25-73	9 + 08	612	17.2	9.4	COD-8	200' R	Reworked-No Retest
15	9-25-73	5 + 10	609	12.1	8.6	COF-2	80' R	Reworked-No Retest
19	9-25-73	8 + 59	611	15.9	11.2	COD-5	82' R	
23	10-06-73	8 + 92	613	20.7	10.8	COL-15	212' R	Material Replaced-No Retes
24	10-06-73	6 + 90	613	24.0	10.8	COL-15	212' R	Material Replaced-No Retes
29	10-08-73	4 + 25	613	18.3	14.7	COD-7	92' R	No Retest
32	10-12-73	1 + 00	615	6.9	9.4	COD-8	50' L	
36	10-17-73	8 + 82	615	15.2	10.8	COL-15	40' R	
37	10-17-73	7 + 82	615	14.2	10.8	COL-15	40' R	
38	10-19-73	8 + 99	615	19.4	10.8	COL-15	110' R	Reworked Area-No Retest
39	10-19-73	9 + 52	615	17.3	10.8	COL-15	110' R	No Retest
40	10-19-73	8 + 49	615	15.2	10.8	COL-15	210' R	
41	10-24-73	3 + 00	617	13.8	11.2	COD-4	150' R	Reworked Area
46	10-24-73	8 + 03	624	19.5	16.4	COD-2	℄	
47	10-25-73	6 + 03	621	10.0	7.4	COL-11	70' R	
48	10-25-73	6 + 03	621	12.6	9.4	COD-8	150' R	
54	11-08-73	4 + 00	624	14.6	11.2	COD-5	20' R	Retested Not Passed-See W(

10099-2

8-

NONCONFORMANCE REPORT (CONT'D)

55

TEST NO.	DATE	STATION	ELEV.	MOISTURE CONTENT	OPTIMUM MOISTURE	CURVE NO.	RIGHT OR LEFT	
WOD 8-55	11-08-73	4 + 00	623	18.3	11.2	COD-5	℄	
59	11-10-73	5 + 00	624	16.5	11.8	COB-2	20' R	
61	11-10-73	4 + 00	623	14.3	10.3	COD-1	20' L	Moisture Too High
64	11-13-73	5 + 50	622	10.5	8.0	?	200' R	No Action Until Spring
							50' L	"Start Up"
				West Plant Dike				
WOD 1- 1	9-11-73	5 + 30	610	4.8	12.0	COB 1		
2	9-11-73	3 + 85	610	4.7	12.0	COB 1	80' R	No Retest
3	9-11-73	2 + 70	610	5.0	12.0	COB 1	80' R	No Retest
14	10-24-73	3 + 52	624	13.7	11.2	COB 1	80' R	Reworked Area No Retest
16	11-08-73	5 + 00	633	10.3	8.0	COD-5	75' R	
						COD-11	25' R of shoulder	
				North East Dike				
WOD 7-35	9-12-73	27 + 00	608	10.3	7.4	COL 11	?	
46	9-25-73	33 + 00	616	19.5	16.4	COD 2	10' R	
47	9-25-73	31 + 00	616	19.6	12.7	COB 3	10' R	Reworked-Retested (see belo
58	10-02-73	28 + 45	612	20.1	16.4	COD 2	85' R	Reworked-Retested
64	10-11-73	32 + 00	614	18.3	14.2	COL 8	90' R	
69	10-12-73	31 + 00	616	21.0	12.7	COB 3	12' R	
74	10-20-73	28 + 00	617	23.0	20.5	COD 6	10' R	Retest-See WOD 7-47
78	11-14-73	30 + 00	622	16.3	11.2	COD-5	10' R	
80	11-13-73	31 + 00	616	17.1	12.6	COD 12	20' R	Retest of 47 & 69 (failed)

10098-2

BECHTEL

NONCONFORMANCE REPORT (CONT'D)

NCR NO. 55

BLOCK 20 CONTINUED

TEST NO.	ZONE	MOISTURE CONTENT	OPTIMUM MOISTURE	DIFFERENCE FROM OPTIMUM	% COMPACTION
WOD8-9	2	13.0	10.3	+2.7	97.6
WOD8-12	2	10.0	7.4	+2.6	100.2
WOD8-13	2	14.2	9.4	+4.8	97.7
WOD8-19	2	15.9	11.2	+4.7	96.0
WOD8-29	2	18.3	14.7	+3.6	99.6
WOD8-36	2	15.2	10.8	+4.4	103.2
WOD8-37	2	14.2	10.8	+3.4	101.9
WOD8-40	2	15.2	10.8	+4.4	100.4
WOD8-41	2	13.8	11.2	+2.6	100.2
WOD8-47	2	10.0	7.4	+2.6	99.4
WOD8-48	2	12.6	9.4	+3.2	100.0
WOD8-59	2	16.5	11.8	+4.7	102.4
WOD8-61	2	14.3	10.3	+4.0	96.8
WOD1-14	2	13.7	11.2	+2.5	99.3
WOD7-35	2	10.3	7.4	+2.9	97.1
WOD7-46	2	19.5	16.4	+3.1	98.4
WOD7-58	2	20.1	16.4	+3.7	96.2
WOD7-64	2	18.3	14.2	+4.1	95.3
WOD7-74	2	23.0	20.5	+2.5	96.6

On the remainder of tests the field submits the following supplemental data for Project Engineering review & evaluation.

WOD8-1	2	10.4	7.1	+3.3	84.9
WOD8-3	2	9.6	12.6	-3.0	103.3
WOD8-4	1	6.8	11.8	-5.0	100.7
WOD8-5	1	6.6	11.8	-5.2	101.1

QC-61

ORIGINAL

EIGHT

BLOCK 20 CONTINUED

NONCONFORMANCE REPORT (CONT.)

NO. 55

TEST NO.	ZONE	MOISTURE CONTENT	OPTIMUM MOISTURE	DIFFERENCE FROM OPTIMUM	% COMPACTION
WOD8-8	2	4.5	7.1	-2.6	97.7
WOD8-14	2	17.2	9.4	+7.8	86.3
WOD8-15	2	12.1	8.6	+3.5	94.9
WOD8-23	2	20.7	10.8	+9.9	94.9
WOD8-24	2	24.0	10.8	+13.2	100.8
WOD8-32	1	6.9	9.4	-2.5	95.7
WOD8-38	2	19.4	10.8	+8.6	95.2
WOD8-39	2	17.3	10.8	+6.5	95.6
WOD8-46	1	19.5	16.4	+3.1	97.8
WOD8-54	1	14.6	11.2	+3.4	97.4
WOD8-55	1	18.3	11.2	+7.1	90.5
WOD8-64	1	10.5	8.0	+2.5	100.5
WOD1-1	1	4.8	12.0	-7.2	95.9
WOD1-2	1	4.7	12.0	-7.3	98.7
WOD1-3	1	5.0	12.0	-7.0	93.7
WOD1-16	1	10.3	8.0	+2.3	99.6
WOD7-47	2	19.6	12.7	+6.9	87.3
WOD7-69	2	21.0	12.7	+8.3	93.3
WOD7-78	2	16.3	11.2	+5.1	97.3
WOD7-80	2	17.1	12.6	+4.5	93.9

If necessary, field recommends evaluation of affected in-place material be done at the same time that evaluation of areas affected by NCR C-26 are conducted.

Gary W. Knull 3-6-78

10033-2

QC-1-3

ORIGINAL

BEBC - 104

Bechtel Associates Professional Corporation

Inter-office Memorandum

GEK N

Gantz

Jeffers

Hudson

To	E. E. Felton	Date	November 7, 1973
Subject	Midland Plant Units 1 & 2 Job No. 7220 Earthwork Moisture Content File: C-210, C-208, 0274	From	P. A. Martinez
		Of	Engineering
		At	Ann Arbor
Copies to	J. H. Allen J. C. Hink R. L. Rixford L. F. Wilcox		

Reference: a) FCR-C-18 dated November 2, 1973

In response to your FCR (ref. a) and based on laboratory test data, compaction data, and location of the material being placed, specification C-210 can be relaxed with the following stipulations:

The optimum moisture content range can be specified as 2% dry to 5% wet of optimum provided that if the moisture content exceeds 2% wet of optimum the fill shall be placed with a compactive effort equal to at least 95% of the Bechtel modified proctor test result (20,000 foot pounds effort). This will be done at no additional cost to Bechtel. This also applies only to zone 2 material which is placed in the Bullock Creek area and in other selected areas of the dike as specified by the Bechtel representative. The moisture control specifications originally written for zone 1 material still apply to zone 1 material. That is, zone 1 material must be placed within a moisture content range of 2% dry to 2% wet.

The above change in allowable range of optimum moisture content for the zone 2 material may result in more than four passes of compaction equipment. However, as pointed out above, this additional effort will not be at the expense of Bechtel since it is being done to allow construction to continue and give the contractor the best utilization of his equipment and people.

P. A. Martinez

RLR/rc

Teletype Message

TYPE DOUBLE SPACE - BE BRIEF

CHECK APPROPRIATE BOX:

Night Lett | Full Rate | Report Delivery: YES / NO | CHARGE ACCT. | NUMBER TO BE CALLED

ADDRESSED TO	ADDRESSEE	ADDRESS	LOCATION (CITY, STATE OR CO.)
	Bechtel Power corporation	2500 E. Miller Road	Midland, Michigan
	Attn: R. E. Felton		

MESSAGE SECTION - If additional addresses are required continue to list below:

March 22, 1974

BEBC - 249

Subject: Midland Plant Units 1 & 2, Job No. 7220

Soil Boring Program

File: C-210, C-208, 0274

Reference: 1) BEBC-238 attach. Sht. 1

In response to telecon request from R. Greta on 3/20/74, this is to clarify that the "X-tests" referred to in ref. 1) consist of moisture content determination (ASTM D 2216) and dry density.

The dry density is to be determined by the following procedure:

Samples will be extracted in Shelby tubes. A representative four to six inch sample shall be cut from the Shelby tube. The cuts must be uniform and perpendicular to the axis of the tube. The sample should then be carefully extruded from the cut portion of the Shelby tube, using a jack with a diameter equal to the inside diameter of the Shelby tube. The extrusion should be

cc: R. Greta, J. H. Allen, S. S. Afifi

040

LOCATION & EXT.

ORIGINATOR'S COPY:

Teletype Message

TYPE DOUBLE SPACE • BE BRIEF

CHECK APPROPRIATE BOX:

Night Ltr. | Full Rate | Report Delivery YES | NO | CHARGE ACCT. NUMBER

ADDRESSEE	ADDRESS	LOCATION (CITY, STATE OR COUNTRY)

MESSAGE SECTION - If additional addresses are required continue to list below:

made vertically with care taken to assure an undeformed sample.

The inside diameter of the Shelby Tube will be used to express the sample diameter and the volume of the sample computed from the cross-sectional area and the average of four height measurements.

The wet density is the ratio between the weight of the extruded sample and the calculated volume of the sample.

The dry density is the ratio between the weight after oven drying at 105°C for 24 hours and the calculated volume of the sample.

Extreme care is required in handling these samples between the boring location and the laboratory to assure obtaining an undeformed sample. The extrusion and weight and height measurements should be made by a qualified technician under the supervision of the engineer.

SIGNATURE

041

LOCATION & EXT.

ORIGINATOR'S COPY

04605

Teletype Message

TYPE DOUBLE SPACE • BE BRIEF

MESSAGE NUMBER	OPR. INL.	

DSG	TELTEX	TWX	TELEX

CHECK APPROPRIATE BOX:

Night Ltr: | Full Rate: | Report Delivery: YES | NO

CHARGE ACCT. CODE:

NUMBER TO BE CALLED

MESSAGE ADDRESSED TO

ADDRESSEE	ADDRESS	LOCATION (CITY, STATE OR COUNTRY)

MESSAGE SECTION - If additional addresses are required continue to list below:

Weight measurements are to be to the nearest 0.1gm and dimensions to the nearest 0.01 inch.

BECHTEL

NONCONFORMANCE REPORT

1. PAGE 1 OF	1
14. NCR NO.	88
2. DRAWING/PART NO.	Spec 7220-C-208
REV.	2
7. PROJECT NO.	7220
12. REPORTED BY	J.C. Chud
DATE	4/17-74
3. ITEM DESCRIPTION	Earthwork Zone 1 & 2
8. ITEM LOCATION	"Q" Listed Dikes
13. VALIDATED BY	J. Connelly
DATE	4-17-74
4. SERIAL NUMBER	N/A
9. STARTUP SYSTEM NO.	N/A
15. REPLACEMENT PART NO.	N/A
5. PURCHASE ORDER NO.	Subcontract C-210 Rev. 2
10. QC FIELD INSPECTION PLAN NO.	C-210 17, 18, 24, 28, 31, 32, 33 & 34
16. REPLACEMENT SERIAL NO.	N/A
6. CONTRACTOR/LOCATION	Canonie Constr. Co. South Haven Mich.
11. ASME CODE ITEM	☐ YES ☒ NO
17. SOURCE	Subcontract

25. DISPOSITION CONCURRENCE: REWORK / REJECT / REPAIR / USE AS IS ☒

[signature] 5/3/
PROJECT FIELD ENGINEER

[signature] 4/30
PROJECT ENGINEER

J. Connelly 5/2/
PROJECT FIELD QC ENGINEER

AUTHORIZED INSPECTOR DATE

18. ROUTING INSTRUCTIONS: ☒ ROUTE TO FIELD ENGINEERING ☐ ROUTE TO MATERIAL SUPERVISOR

19. NONCONFORMING CONDITION: Spec 7220-C-208, Table 9-1, page 14a states in part: field densities, moisture content test freque will be one per every 500 cubic yards of fill. Actual test taken was one per every 2300 cubic yards. Ref: NCR #C-26 & NCR #55 It was recommended by Project Engineering borings be taken to evaluate the in place density of affected areas. Approx. 500 samples were taken in areas as designated by Project Engineering (work done under subcontract FSC-60, Raymond Internation) Out of all samples taken 5% are actually failures.

20. ☐ FIELD DISPOSITION ☒ FIELD RECOMMENDATION/ROUTE TO PROJECT ENGINEERING

Disregard failures as they are widley spread out and not far out of spec. A large percentage of these failures are also in the top one to two feet of dike and would have to be reconditioned before placement of embankment anyway. We recommend leaving dike as is with reconditioning of top lift as required. J.C. [signature] 4/17-74

21. FIELD DISPOSITION RESULTS:

22. ENGINEERING DISPOSITION

Based upon evaluation of data from the boring program initiated in response to NCR-C-26, Engineering recommends the Plant Area fill be used-as-is. A summary of the results of the boring program is being completed and will be forwarded under separate cover. Earthwork may proceed on the plant Area Fill. RFR 4-30-74 JGH 4-30-74

23. ENGINEERING DISPOSITION RESULTS:

RECEIVED
MAY 3 1974
BECHTEL POWER CORP.
JOB 7220
PER M. H.

24. IS DESIGN CHANGE REQUIRED ☒ NO ☐ YES, SEE ATTACHED:

DRAWING ______ REV. ______ DCN ______

SPEC. ______ REV. ______ ADD. ______

26. REJECTED MATERIAL DISPOSITION ☐ RETURN TO SUPPLIER ☐ SCRAP

REMARKS

27. QC ACCEPTANCE [signature] 5/15/
QC ENGINEER DATE

AUTHORIZED INSPECTOR DATE

ORIGINAL

Forecast Date 5-10-74

NONCONFORMANCE REPORT

C-208	REV. 2	7. PROJECT NO. 7220	DATE 4-17-74
2 & 2		8. ITEM LOCATION Q Area Dikes	DATE 4-17-74
		9. SYSTEM NO. N/A	
C-208 Rev 2		10. QC FIELD INSPECTION PLAN NO. C-208, 7-18, 24, 27, 31, 32, 34	N/A
Co. Sand Grans Mach.		11. ASME CODE ITEM — NO	17. SOURCE Subcontract

ROUTE TO FIELD ENGINEERING

ROUTE TO MATERIAL SUPERVISOR

NONCONFORMING CONDITION:

Spec 7220-C-208, Table 9-1, page 14a states in part: field densities, moisture content test freque will be one per every 500 cubic yards of fill. Actual test taken was one per every 2300 cubic yards. Ref: NCR #C-26 & NCR 455. It was recommended by Project Engineering borings be taken to evaluate the in place density of affected areas. Approx. 600 samples were taken in areas as designated by Project Engineering. Work done under subcontract FSC-60, (Raymond International) Out of all samples taken 5% are actually failures.

FIELD DISPOSITION

FIELD RECOMMENDATION/ROUTE TO PROJECT ENGINEERING

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21. FIELD DISPOSITION RESULTS

ENGINEERING DISPOSITION

Based upon evaluation of data from the boring program initiated in response to NCR C-26, Engineering recommends the Plant Area fill be used-as-is. A summary of the results of the boring program is being completed and will be forwarded under separate cover. Earthwork may proceed on the plant area fill. RLC 4-30-74 [signature] 4-30-74

23. ENGINEERING DISPOSITION RESULTS

24. IS DESIGN CHANGE REQUIRED — NO / YES, SEE ATTACHED

DRAWING ___ REV. ___ DCN

SPEC. ___ REV. ___ ADD.

26. REJECTED MATERIAL DISPOSITION — RETURN TO SUPPLIER / SCRAP

27. QC ACCEPTANCE

ORIGINAL

Bechtel Associates Professional Corporation

Inter-office Memorandum

BEBC - 376

To E. E. Felton

Date June 10, 1974

Subject Midland Plant Units 1 and 2
Job No. 7220
Report of Soils Boring Program
File: C-210, 1700, 0274

From P. A. Martinez

Of Engineering

At Ann Arbor

Copies to
J. H. Allen w/o
S. S. Afifi w/o
J. C. Hink w/a (less appendices)

References: a) NCR 26
b) NCR 88

Transmitted herewith is the report of the Soils Boring Program initiated as a result of NCR 26 and required to complete action on NCR 88.

This report completes Engineering action on the two referenced NCRs.

P. A. Martinez

RLR/slv

Enclosure

RECEIVED
JUN 10 1974
BECHTEL POWER CORP.
JOB 7220
PER ___

REPORT FOR NCR 88

On March 26, 1974, a sampling and testing program for additional moisture and density checks was started under the supervision of a Geotech representative as requested by engineering to respond to NCR 26. Drilling and sampling was started March 26, 1974 and completed on April 5, 1974. Laboratory testing was completed April 11, 1974. The tests were compiled and since 5 percent compaction values fell below 95 percent, NCR 88 was initiated.

The data pertinent to NCR 88 in connection with the existing fill in the west plant dike, north plant dike, and northeast plant dike are discussed herein. The intent of this report is to assist engineering in evaluating and documenting NCR 88.

A total of 58 borings were drilled in the west plant dike, north plant dike, and northeast plant dike. These borings penetrated Zone 1 material and Zone 2 material as indicated on Figure 1 by solid symbols and open symbols, respectively. Boring ground surface elevation, coordinates and depth are shown in Table 1.

From these borings, a total of 356 Shelby tube samples were taken. The samples were cut in the laboratory to lengths of about 6 inches resulting in a total of approxiamtely 451 specimens suitable for testing (338 in the north plant dike, 53 in the west plant dike, and 60 in the northeast plant dike). Another 84 specimens were not considered suitable for testing because of tube damage or excessive stone content, as indicated in the remarks columns of the tables in the attached Appendix A, which contains a tabulation of laboratory test data. Appendix B contains laboratory data worksheets.

Moisture determinations were made according to ASTM Designation D 2216, density determination according to Chapter 1, page 37 of Earth Manual, U.S. Department of Interior.

Test Results

Figures 2 and 3 show plots of percent Bechtel modified compaction (BMC) versus depth for the borings wherein percent compaction below 95 percent were encountered. Test results which were judged unacceptable by the soils engineer on the job were not included in these plots. These were results from samples which came from the sand drain (Zone 3 material), contained stones, or were disturbed. In the case of sand drain or excessive rock, it was judged that samples volume measurements were inaccurate. See remarks, column, Appendix A.

RECEIVED
JUN 10 1974
BECHTEL POWER COR
JOB 7220
PER R

Figure 2 contains data where the percent compaction below 95 percent was either above 94 percent or the samples taken were near the surface (TB 24, 21, and 4, NPD). Data between 94 percent and 95 percent, when occurring in the infrequent manner shown in Figure 2, is considered acceptable. The difference between 94 percent and 95 percent is not significant when considering the accuracy range inherent in sampling and testing procedures used in practical soil mechanics. Furthermore, these data were not a part of a trend of reducing density within the fill, as can be seen from Figure 2. This is substantiated further the lines of average percent compaction (Figure 2), which shows that the degree of compaction was above the 95 percent value. Averaging of soil properties, within a reasonable depth range which does not contain significant scatter is a commonly accepted tool exercised by soils engineers. Therefore, all data between 95 percent and 94 percent are considered within the intent of 95 percent BMC compaction and will not be further discussed.

Data near the surface fell within the zone where removal and reconditioning will be required before placement of new fill (only 3 cases: TB 24, TB 21, and TB 2, NPD). The degree of compaction should increase after reconditioning and passage of the 50-ton roller equipment.

Figure 3 shows plots where occasional percent compaction less than 94 percent were encountered. The plots also show the 95 percent compaction line and the average percent compaction line. These same borings are indicated with a hexagon on Figure 1 and amount to 10 borings.

All the above 10 cases in Figure 3 were between 90 percent and 95 percent compaction. The values below 95 percent occurred in the form of spikes in the percent compaction versus depth correlation. Further, they represent one value between 90 percent and 95 percent per 5000 cubic yards for northeast dike, 3200 cubic yards for west plant dike, 6350 cubic yards in north plant dike. These occurred at scattered locations as can be seen from hexagons in Figure 1.

Furthermore, lines of average percent compaction for the holes show percent compaction above 95 percnet (Figure 3). Except when soil properties vary within a large range, the soil behavior is more determined by the average pertinent property than by the absolute maximum or the absolute minimum.

It can, therefore, be concluded that the in-place fill tested meets the intent of a 95 percent degree of compaction by the Modified Bechtel Method.

6-10-79

Bechtel Associates Professional Corporation

Inter-office Memorandum

BEBC – 810

To J. Connolly

Date June 9, 1975

Subject Midland Plant Units 1 & 2
Job No. 7220
Documentation of Change to
NCR 88 Report
File: C-210, C-1700, 0294

From R. L. Castleberry

Of Engineering

At Ann Arbor

Copies to

J. F. Newgen
W. F. Holub

Enclosure: 1) IOM, S. S. Afifi to R. L. Castleberry, 5-6-75

This is to transmit enclosure 1, officially confirming the information given verbally by J. O. Wanzeck on 6-19-74.

R. L. Castleberry

RLR/jeh

Enclosure.

RECEIVED
JUN 12 1975
QUALITY CONTROL
BECHTEL JOB. 7220
SIGNATURE

ROUTE QC 07220 INIT.
PFOGE
A. PFOGE
CIVIL
2
OPEN
☐ YES X
DATE

C-1700

Bechtel Associates Professional Corporatio

Inter-office Memorandum

To. R. L. Castleberry

Date 8 May 1975

Subject NCR 88 Text Correction
Midland Units 1 & 2
Job 7220-001

From S. S. Afifi

Of Geotechnical Services

Copies to R. O. Anderson
S. L. Blue
R. L. Rixford
1320,2530,3410

At Ann Arbor 10(D)5

This memo transmits a memo from J. O. Wanzeck to me regarding a correction required in the text of NCR 88. This correction was given verbally by J. O. Wanzeck to field personnel.

Don Horn of Consumers Power Company QA requested today that this correction be transmitted officially to the field.

S. S. Afifi

SSA:lab

Attachment

MAY 12 1975
A.A. 7220

civil

0294,0545

Bechtel Associates Professional Corporation

Inter-office Memorandum

To S. S. Afifi

Date 10 January 1975

Subject Midland Units 1 & 2
NCR 88 - Documentation of Verbal Changes

From J. O. Wanzeck

Of Geotechnical Services

Copies to S. L. Blue w/o
R. L. Castleberry w/a
R. Rixford w/a
1310,2530,3410

At Ann Arbor - E

The following memo is to document my response to a question raised by Midland QC and Consumers QC on June 19, 1974, as noted in my trip report dated June 21, 1974.

On Report NCR 88 (attached), see Paragraph 1, second page which should read ". . . (TB24 NPD & 21, 4 NED)."

Paragraph 2 of the second page should read ". . . (TB24 NPD & TB21,2 NED)."

This information was given to the field verbally; therefore, this memo is intended for documentation.

J O Wanzeck
J. O. Wanzeck

JOW:lab

Attachment

REPORT FOR NCR 88

On March 26, 1974, a sampling and testing program for additional moisture and density checks was started under the supervision of a Geotech representative as requested by engineering to respond to NCR 26. Drilling and sampling was started March 26, 1974 and completed on April 5, 1974. Laboratory testing was completed April 11, 1974. The tests were compiled and since 5 percent compaction values fell below 95 percent, NCR 88 was initiated.

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Figures 2 and 3 show plots of percent Bechtel modified compaction (BMC) versus depth for the borings wherein percent compaction below 95 percent were encountered. Test results which were judged unacceptable by the soils engineer on the job were not included in these plots. These were results from samples which came from the sand drain (Zone 1 material), contained stones, or were disturbed. In the case of sand drain or excessive rock, it was judged that sample volume measurements were inaccurate. See remarks, column, Appendix A.

Figure 2 conta[illegible] data where the percent compaction below 95 percent was either above 94 percent or the samples taken were near the surface ~~(TB-24, 21, and 4, NPD)~~. Data between 94 (TB24 NPD & 21, 4 NE) percent and 95 percent, when occurring in the infrequent manner shown in Figure 2, is considered acceptable. The difference between 94 percent and 95 percent is not significant when considering the accuracy range inherent in sampling and testing procedures used in practical soil mechanics. Furthermore, these data were not a part of a trend of reducing density within the fill, as can be seen from Figure 2. This is substantiated further the lines of average percent compaction (Figure 2), which shows that the degree of compaction was above the 95 percent value. Averaging of soil properties, within a reasonable depth range which does not contain significant scatter is a commonly accepted tool ~~exercised~~ by soils engineers. Therefore, all data between 95 percent and 94 percent are considered within the intent of 95 percent BMC compaction and will not be further discussed.

Data near the surface fell within the zone where removal and reconditioning will be required before placement of new fill (only 3 cases: ~~TB-24, TB-21, and TB-2, NPB)~~. The degree of (TB24 NPD & TB21,2 [illegible]) compaction should increase after reconditioning and passage of the 50-ton roller equipment.

Figure 3 shown plots where occasional percent compaction less than 94 percent were encountered. The plots also show the 95 percent compaction line and the average percent compaction line. These same borings are indicated with a hexagon on Figure 1 and amount to 10 borings.

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It can, therefore, be concluded that the in-place fill tested meets the intent of a 95 percent degree of compaction by the Modified Bechtel Method.

4 [illegible]

To	BWMarguglio, JSC-220A
From	DEHorn, Midland DEH
Date	October 31, 1978
Subject	MIDLAND PROJECT - NRC EXIT INTERVIEW OF OCTOBER 27, 1978 File: 0.4.2 Serial: 280FQA78

Consumers Power Company

Internal Correspondence

CC	SAfifi, Bechtel - Ann Arbor WRBird, JSC-216B RLCastleberry, Bechtel - Ann Arbor TCCooke, Midland	JLCorley, Midland GSKeeley, P14-408B DBMiller, Midland JFNewgen, Bechtel

The following people were in attendance at the subject exit interview which was conducted at the end of G. J. Gallagher's inspection of October 24-27, 1978:

CPCo	Bechtel	NRC
RCBauman	WLBarclay	RJCook
TCCooke	ABoos	GJGallagher
JLCorley	RLCastleberry	
DEHorn	LADreisbach	
GSKeeley	PAMartinez	
DBMiller		
BHPeck		
RMWheeler		

Mr. Gallagher stated that the visit was a follow-up on 50.55(e) report of the diesel generator settlement and that it was also a fact finding visit. The inspection consisted of a review of past data, activities in progress and planned activities for future work. Inspection was performed by review of the FSAR commitments; Specification C-210; Specification C-211; PQCI/IR C-1.02; Dames and Moore Report of Foundation Investigation and Preliminary Explorations for Borrowed Materials dated June 28, 1968 and supplement to this report dated March 15, 1969; preliminary data on diesel generator settlement problem including boring plan, cross sections of fill, blow count versus the elevation graphs, lab data, settlement data, boring logs, dutch cone logs, weather data and penetrameter readings in test pits; design drawings C-45, C-109, C-117 and C-1001; soil tests taken in the diesel generator building area during construction compiled by B. T. Cheek, Bechtel QC; observation of soil testing at the test lab and in the field; and discussions with Bechtel Geo-Tech, Project Engineering, Field Engineering, Quality Control Engineering, U.S. Testing, Consumers Power Company, PMO and QA personnel. Mr. Gallagher stated that he would not handle the findings as noncompliances, however, they could become items of noncompliance when they are reviewed by his management.

His findings/observations were as follows:

1. The FSAR states that during operation, settlement readings will be taken every 90 days. Because of the diesel generator settlement problem, this frequency should be re-evaluated for adequacy.

2. FSAR Table 2.5-14 "Summary of Foundation Supporting Seismic Category I Structures" identifies the supporting soil materials under the diesel generator building as being controlled, compacted cohesive soils. However, construction drawing C-109, Rev. 9 and C-117, Rev. 6 identifies the material in this area as Zone 2 material. Zone 2 material is identified as random fill described as any material free of organic or other deleterious materials. In the field a variety of materials have been used for the diesel generator foundation material, in particular, sands, clay, and lean concrete, silty sands and clayey sands. The apparent conflict is that Table 2.5-14 identifies cohesive soils where, in actuality, cohesionless sands have been utilized. A review of the records indicate that sands have been used between elevation 594'-608', areas of elevation 611'-613' and areas between 616'-268'. This indicates the extent of the variability of the material placed under the diesel generator building foundation. Mr. Gallagher did not feel it was good judgement to use random material under the support of a structure.

3. FSAR Table 2.5-21 "Summary of Compaction Requirements" identify random fill to require a compaction effort of a minimum of 4 passes with the specified equipment in this table. This requirement has not been an imposed requirement of Bechtel Specification C-210 nor an inspection requirement of Bechtel Quality Control Instruction C-1.02 for backfill.

4. FSAR section 3.8.5.5 states that settlements of shallow spread footings founded on compacted fill are estimated to be on the order of ½" or less. Site Survey Program has identified settlements in the diesel generator building foundation on spread footings to range from 0.55 inches to 2.30 inches and in excess of 3.0 inches for the diesel generator pedestal.

5. FSAR figure 2.5-47 indicates the foundation of the diesel generator building to be at elevation 634', according to design drawings C-1001, Rev. 5 it is indicated for the diesel generator spread footings and pedestal foundation to be at 628'.

6. A. Specification C-210, section 13.7.1 requires all cohesive backfill in the plant area to be compacted to not less than 95% maximum density as determined by ASTM D1557 method D which requires an effective compactive effort of 56,000 foot-pounds of energy per cubic foot of soil. However, section 13.4 Testing requires testing of the materials placed in the plant area to be performed in accordance with tests listed in section 12.4. This section, in particular section 12.4.5.1, "Cohesive Soils," requires maximum lab densities to be determined using ASTM D1557 Method D provided a compactive energy equal to 20,000 foot-pounds per cubmic foot is applied (Bechtel Modified Proctor Density). To date, the Bechtel Modified Proctor Density for determining maximum proctor density versus optimum moisture content has been utilized. This conflict results in an unconservative method of determining the maximum proctor density and method of assuring that the required percent compaction is achieved. In particular, the actual in-place compaction would be less using the Bechtel Modified Proctor Density as a reference than using the standard ASTM D1557 method D. This is due to the fact that the compactive energy exerted using the Bechtel Modified Method is less than the effort exerted by the standard method D - example: 20,000 foot-pounds versus 56,000 foot-pounds.

6. B. Bechtel Quality Control Instruction C-1.02 section 2.4 testing identifies the applicable inspection criteria and includes Specification C-210, section 13.7 and 12.4 which includes the apparent conflict as described in detail in Part A above.

   C. A further review of the original subsurface investigation performed by Dames and Moore and documented in report supplement dated March 15, 1969 page 16 indicates that the recommended minimum compaction criteria for support of structures be 100% of maximum density using a compactive effort of 20,000 foot-pounds (resulting from Bechtel Modified Proctor determination). However, this 100% of Bechtel Modified Proctor corresponds to 95% compaction according to the standard ASTM D1557 method D and not 95% compaction according to Bechtel Modified Proctor method which has been utilized for the entire plant fill area to date. Furthermore, Dames and Moore Report, page 15 states that all fill and backfill material should be placed at or near the optimum moisture content in near horizontal lifts approximately 6-8" in loose thickness. Bechtel specification permits a maximum of 12 inches which affects the compactability of the material.

7. Piping, condensate lines, duct banks, and other utilities under the diesel generator building may also be affected and must be evaluated.

8. Mr. Gallagher stated he was leaving not having seen design calculations and will be discussing design calculations, assumptions made, and conflicts with the FSAR with Licensing.

9. The inspector observed the structural concrete crack that has developed in the east exterior wall. The crack was observed with members from Bechtel Geo-Tech and Consumers Power Company. The crack extended full height of the wall and continued down through the spread footing as seen from the inside of the building. The crack is expected to have been induced flexurally caused by differential settlement. Discussion with Bechtel design staff has indicated that this crack is under study and is currently being evaluated. ACI-318-71 in the commentary section 10.6.4 limits flexural crack exposed to the outside to 0.013". Corrective action may be required if this limit is exceeded.

10. The following tests were observed to be performed in accordance with the applicable tests standards by U.S. Testing:

    A. Lab Test ASTM D1557-70

    B. Field Test ASTM D/1556-64

11. Calculations should be evaluated on the increase and the rate of increase of the pond fill and the effects of the water in other areas.

12. Mr. Gallagher stated that the NRC does not view preloading of the structure to be a fix or resolution of the problem at this time.

13. Seismic loading calculations should be determined for the type of material existing in its present condition.

Question 6

You propose to fill the borated water storage tanks and measure the resulting structure settlements.

(a) On what basis do you conclude a surcharge no greater than the tank loading will achieve compaction to the extent intended by the criteria stated in the PSAR? What assurance is provided by the technique that residual settlement for the life of the plant will not be excessive?

(b) A similar procedure is proposed for other tanks, including the diesel fuel oil storage tanks, and should also be addressed.

(c) The borated water storage tanks have not yet been constructed and are to be located upon questionable plant fill of varying quality. Provide justification why these safety-related tanks should be constructed prior to assuring the foundation material is suitable for supporting these tanks for the life of the plant. For example, can the tanks be removed with reasonable effort without significant impact?

Response (to 6a)

The results of field explorations in the borated water storage tanks area generally indicate satisfactory fill. To date, 18 borings have been taken in this area. Three of these borings indicate some soft materials. However, based on three borings per tank, there has been no identified unsatisfactory material directly beneath the borated water tanks.

MUST HAVE BACKUP / DEH

(CAH) TO DEMONSTRATE

~~To further evaluate if~~ the fill in the area is satisfactory, an earthen preload will be performed on the west borated water storage tank area ~~will be performed~~ prior to construction of the tank. The existing tank ring and valve pit will be monitored How long to predict future settlement, and to allow remedial action, if any, before the tank is constructed. For the east borated water storage tank, a preload (either using earthen materials or filling the tank after construction) will be performed. The selection of the method chosen will be based on the results from the preload of the first tank.

CAH

??

It is expected that the preloads, together with the majority of the boring results, will confirm the adequacy of the foundation materials in this area. The preloads will also allow prediction of the residual settlements expected for the life of the plant.

Response (to 6b)

The diesel fuel oil storage tanks have been filled and are being monitored for settlement to predict future settlement and assess the need for remedial work required to ensure limited residual settlement. These tanks are supported on medium to very stiff sandy clay and clean sand fill. These tanks are surrounded with backfill consisting of very loose to dense clean sands and very soft to stiff clays. These adjacent materials do not meet PSAR requirements. Locations of borings made in this area are shown in Figure 9-1. A cross section summarizing the results of these borings is shown in Figure 6-7. If results of the evaluation made on these tanks cannot enusre limited residual settlements, the tanks will be surcharged or removed and reconstructed. The loose snad fill will be grouted.

CAH
GENERAL — SHOULD STATE WHAT WILL BE DONE OR NOT. SHOULD NOT LEAVE OPEN. SHOULD KNOW WHAT ACCEPT/REJECT IS AND STATE POSITIVELY WHAT WE WILL DO — LEAVE OUT WEASLE WORDS

Response (to 6c)

As described in the response to Part a, one or both borated water storage tank areas will now be preloaded before the tanks are constructed, using an earthen surcharge load. No significant foundation problems are anticipated, and the preload on the west tank is expected to confirm this. If necessary, an earthen preload will also be performed on the east tank. Although removal of the tanks after construction would be both costly and require a schedule delay, the tanks are accessible and removal remains a viable alternate if unexpected future foundation problems in this area necessiate remedial actions.

Question 2

Discuss the consideration given to, and estimate the cost of, grouting any natural lacustrine deposits (sands) upon which safety-related structures are founded.

Response

DEH

IS THIS TRUE?

~~Consideration will be given to grouting any~~ natural lacustrine sand deposits, WHICH ARE LOOSE AND ~~that would be~~ susceptible to liquefaction. WILL BE GROU[illegible]

CAH

Borings made to date ONLY ~~indicate that these materials are isolated and have only been identified~~ in one boring at the service water pump structure. Borings will be made to identify the extent of this material. A grouting program would cost an estimated $250,000 for the cantilevered portion of the structure.

Question 5

To what extent will additional borings and measurements be taken after completion of preloading programs to ascertain that the material has been compacted to the original requirements set forth in the PSAR.

Response

(CAH)

EXPAND

It is not expected that material properties of the surcharged fills will reach those properties associated with compaction requirements set forth in the PSAR. Material properties will be evaluated based on settlement-rebound measurements made during and after removal of surcharge loads. For these reasons, it is not planned to make borings or associated measurements after surcharge removal. FOR THE PURPOSE OF ESTABLISHING PSAR CRITERIA.

CAH

AND AFTER REVIEW WITH THE NRC STAFF, THE FSAR WILL BE CHANGED — GSK

SHOULD EXPAND ON THEORY AS TO WHY SURCHARGING IS NOT THE SAME AS COMPACTION BY LAYERS

Question 4

Specify and justify the acceptance criteria which you will use to judge the acceptability of the fill, structures, and utilities upon conclusion of the preload program. Compare these criteria with that to which the material was to have been compacted by the original requirements set forth in the PSAR. The response should consider all areas where preloading is either planned or in progress (i.e., diesel generator building, borated water storage tanks, diesel fuel oil storage tanks, Unit 1 transformer, condensate storage tanks, and others still under evaluation). Describe how conformance to these criteria will result in assurance that unacceptable residual settlements cannot reasonably be expected to occur over the life of the plant. For each such area, state the extent of residual settlement which will be permitted and the basis for each limit.

FSAR NOW CONTROLLING DOCUMENT AS PSAR SHOULD B.U. DEAD DOCUMENT — IF FSAR INADEQUATE, FSAR CHNG NOTICE REQ'D

Response

FILL [illegible] CAH

Acceptance of each surcharge program will require that the structures and utilities withstand the dynamic design criteria established in the PSAR and ~~twice~~ the predicted long-term total and differential settlements. This may require redesign of foundations and/or other remedial work. The resulting long-term settlement and bearing capacity predictions will be compared to the requirements set forth in the PSAR after

CONSISTENT? WHAT BASIS / CAH

AMENDMENTS

completion of the surcharge programs. Surcharge programs are not expected to compress the fills to the densities associated with the compaction criteria set forth in the PSAR. HOWEVER SINCE THE FSAR IS THE CONTROLLING DOCUMENT THE RESULTS OF THE SURCHARGE PROGRAM INCLUDING CRITERIA WHICH MUST BE MET WILL BE REVIEWED WITH THE STAFF AND WILL BE MADE TO THE FSAR

~~Criteria to be used to determine the acceptability of the fills~~, structures, and utilities upon conclusion of preload programs will be based on their behavior during preloading. This behavior will be monitored by measuring movement of the structures and ~~/or~~ borros anchor settlement rods and settlement plates placed in the fill and ~~the~~ buildup and dissipation of excess pore water pressure ~~measured by piezometers placed throughout the fill~~. Movements of selected piping will be monitored before, during, and after preloading to ascertain the effects of loading. Duct banks will be evaluated based on verification that they are functional by field testing. Rate of settlement will be evaluated based on consolidation-rebound curves to predict additional settlement that will occur ~~after surcharge removal~~ under final loading conditions WITHOUT THE SURCHARGE (CAH). Expected dynamic soil-structure behavior will be evaluated based on stress-strain moduli at low strain levels measured during rebound and shear wave velocity measurements from cross hole tests to be conducted in the fill material.

IS THIS TEST SUITABLE FOR HIGHLY VARIABLE MATERIAL? (CAH)

FOR HOW LONG?

The extent of residual settlement that will be allowed for each structure to be surcharged will depend on the extent of settlement each structure experiences during surcharging,

(CAH)

DELETE (CAH)

and therefore cannot be established at this time. This information will be forwarded to the NRC by __________. The surcharge program for the diesel generator building is in progress. Sands susceptible to liquefaction will be grouted, densified by other means, removed, and replaced, or gravel drains will be installed to prevent pore pressure buildup after surcharge removal. The location of surcharge instrumentation is shown in Figure 4-1. Soil and building response data from the measurements performed to date are summarized in Figures 4-2 through ________. Results of monitoring selected utilities are shown in Figure ______.

LIMIT TO WHICH [illegible] [illegible] [illegible] WAS NOT ASKED ABOUT IN [illegible] (CAH)

GSK – (See QUESTION # ____)

A preload program is planned for one or both of the borated water storage tanks. The condensate storage tanks will be constructed, filled, and monitored for settlement. The Unit 1 transformer area will be surcharged prior to completion of construction. The diesel fuel oil tanks have been filled and are currently being monitored to determine any need for surcharging or other remedial action. Acceptance of these diesel fuel oil tanks will be based on a design to withstand those settlements experienced, plus double the future predicted settlement. If designs cannot allow for this settlement, the tanks will be surcharged prior to making piping connections or removed and replaced.

No Surcharge TCC

Describe, unclear DEH

General: Very important to [illegible] that the [illegible] [illegible] [illegible] [illegible] [illegible] and that if [illegible] demonstrate [illegible] as we had stated in the PSAR (CAH)

Discussed in AA. on 1-14-80 /

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1/10/80

Question 4

Specify and justify the acceptance criteria which you will use to judge the acceptability of the fill, structures, and utilities upon conclusion of the preload program. Compare these criteria with that to which the material was to have been compacted by the original requirements set forth in the PSAR. The response should consider all areas where preloading is either planned or in progress (i.e., diesel generator building, borated water storage tanks, diesel fuel oil storage tanks, Unit 1 transformer, condensate storage tanks, and others still under evaluation). Describe how conformance to these criteria will result in assurance that unacceptable residual settlements cannot reasonably be expected to occur over the life of the plant. For each such area, state the extent of residual settlement which will be permitted and the basis for each limit.

Response

Acceptance Criteria

a. Fill - The acceptance criteria for the fill are based on predicted residual settlements and differential settlements after final connections are made. These predicted values are listed in Table 4-1.

b. Structures - A structure is acceptable if it withstands specific load combinations without exceeding allowable code stresses: 5

   1. Load combinations specified in FSAR Section 3.8

   2. Special load combinations due to the variable stiffness of the support media (refer to Questions 14 and 15)

c. Utilities - Systems and components subject to the preload program will be ~~acceptable~~ if proven by test or analysis to perform their intended function with sufficient margins of safety for all loading conditions.

   1. Buried Piping - Buried piping must withstand specific load combinations compared to the following allowable code stresses:

      a) Applicable ASME criteria see Question 17 for wording

      b) Special ~~considerations~~ due to the variable settlement of the fill (refer to Question 17)

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2. Electrical Duct Banks - Electrical duct banks must meet the seismic design conditions of the response to Question 13.

Justification and Comparison to PSAR

a. Fill - The compaction requirements set forth in the PSAR were based on the premise that significant engineering properties, strength, and compressibility are related to the degree of compaction. The relevant engineering properties have been established by more direct means during the preload program.

The surcharge and the completed portion of the diesel generator building produced stresses in the fill that exceeded those that will prevail when the structure is operational. The surcharge was maintained until the rate of residual settlement became sufficiently small to allow a conservative prediction of residual settlement by extrapolation. It can then be concluded with assurance that the rate of settlement will be considerably less than the prediction. Because of the initial variability of the degree of compaction of the fill, it is unlikely that the compaction requirements of the PSAR will be satisfied at all points; however, because of the ensured favorable settlement characteristics due to the surcharge, the design intent of the PSAR has been met. 5

Rebound measurements of the diesel generator building were made during surcharge removal to allow estimates of the dynamic stiffness of the supporting medium. Following removal of the surcharge, shear wave velocity measurements were also taken to provide further supporting information on dynamic stiffness of the fill. Shear wave velocity measurements were also made in the service water structure area, condensate tank area, and borated water storage tank area (BWST). These data show the shear wave velocity of the fill material exceeds the 500 fps used as the lower bound design basis.

The analysis of the sand fill indicated a potential for liquefaction in limited areas. A permanent dewatering system has been selected as a positive solution to eliminate the liquefaction potential.

b. Structures - The justification of techniques used to evaluate the diesel generator building has been described in the responses to Questions 14 and 15.

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c. Utilities - The justification of techniques used to evaluate the buried utilities is described in the responses to Questions 13 and 17.

Extent of Residual Settlement

a. Diesel Generator Building - The intent of the preload program for the diesel generator building has been achieved, and removal of the surcharge was started on August 15, 1979, and completed on August 30, 1979. During the July 18, 1979, meeting with the NRC, R.B. Peck summarized the adequacy of the surcharge program as follows:

> The results of the preload procedure have been convincing. The observed pore pressures were smaller than actually anticipated, and they dissipated rapidly. Hence, primary consolidation was accomplished quickly, and the curve of settlement as a function of the logarithm of time became linear shortly after the completion of placement of the fill. Therefore, it is possible to forecast the settlement that would occur at any future time by simple extrapolation, on the assumption that the surcharge will remain in place. Even this amount of settlement would be acceptable. However, the projected settlement determined on this basis is 5
> an upper bound because the surcharge will be removed, and the real settlements will certainly be smaller.

Settlements can also occur as a result of densification of sand fill during earthquake ground shaking. These settlements were evaluated using the approach described by Seed and Silver[1] and recommendations on multidirectional shaking given in ~~Ryke~~ Pyke, Seed, and Chan[2]. These were based on a safe shutdown earthquake (SSE) acceleration of 0.12 g and soil borings made through the fill in the diesel generator building area prior to the preload program.

The upper bound settlements and differential settlements which are the design basis for the diesel generator building area are tabulated below. These are based on an evaluation of the settlement magnitudes and patterns predicted on the basis of: a) the surcharge program for static loading and b) shakedown calculations for earthquake conditions.

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Contribution	Settlement (inches)	Differential Settlement (inches) N-S	E-W
Building			
Static, 40 years	1-1/2	3/4	1/2 (1)
Earthquake shakedown	1/2	1/2	1/2
Pedestals			
Static, 40 years	1-1/2	1/2	1/2
Earthquake shakedown	1/2	1/2	1/4
Diesel engine foundation vibrations	1/2	1/2	1/4

(1) May also occur along the northeast part of the building

Settlements due to dewatering from elevation 627' to approximately 600' will be small (approximately 1/2 inch), essentially elastic and uniform, and will take place before final connections are made. delete

The above values are acceptable upper bound values for the following reasons.

1. The 40-year contribution of 1.5 inches is based on stresses in the fill during the surcharge program which are greater than the magnitudes which will be experienced during operation.

2. The 40-year contribution of 1.5 inches is the highest value among 32 predicted values in which 30 values ranged between 0.4 and 1.1 inches and 2 values were approximately 1.3 and 1.4 inches, respectively. The larger values were predicted along the south wall where more clay was encountered in the borings.

3. The shakedown contribution of 0.5 inch is based on the assumption that the sand is dry, which ignors the benefit from capillary action due to moisture.

In summary, the future settlement of the diesel generator building and pedestals will be a combination of the above values.

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b. Borated Water Storage Tanks - Soil borings within and around the BWSTs show the conditions are satisfactory for support of the tanks. A comparison between standard penetration test results for the borings within and around the tanks and the borings taken at the diesel generator building before surcharge shows the conditions at the tanks are better than those at the diesel generator building before surcharge. Based on the size of the loaded areas occupied by the tanks and the more favorable conditions at the tanks, it is estimated that the residual settlement of the BWSTs will be less than the 40-year prediction for the diesel generator building. It is estimated that the residual settlements of the BWSTs will be on the order of 1 inch. The actual value will be determined based on the full-scale test to be performed by filling the tanks with water and monitoring them until the rate of movement becomes small, thus allowing prediction of residual settlement by extrapolation. The minimum duration of the test will be 4 months. No significant sand fill was encountered in the borings below and around the tank and therefore settlement due to earthquakes is not applicable in this case.

c. Emergency Diesel Fuel Oil Storage Tanks - The emergency diesel fuel oil storage tanks are buried structures that have already been subjected to a full-scale loading by filling with water for 8 months. The test was terminated because settlements under these test conditions were minimal. Furthermore, based on the
preload program at the diesel generator building, it 5
was observed that primary consolidation for plant backfill material was accomplished in 3 to 4 weeks after the surcharge load was applied. The test for the tanks lasted 8 months and has been judged sufficient to achieve the desired primary consolidation of the backfill under the full weight of the tanks and to obtain sufficient settlement data which can be extrapolated to the 40-year life of the tanks. Based on these measurements, the residual settlement of these tanks is expected to be less than 1 inch. To confirm this estimate, measurements will be continued. Based on the borings within and around the tanks, no significant sand fill was encountered below the tank foundation elevation and therefore settlement due to earthquakes is not applicable in this case.

d. Unit 1 Transformers and Condensate Tanks - The Unit 1 transformer is non-Seismic Category I, but has been preloaded with 5 feet of sand and monitored. The non-Seismic Category I condensate storage tanks will also

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be monitored. In addition, the design includes a flexible connection detail which will allow relative movement between the tanks and the attached piping. Estimated settlements for these structures are given in Table 4-1.

Assurance that Unexpected Residual Settlement will Not Occur

The preloading at any structure serves the following purposes.

a. A primary benefit of preloading a building is that most of the settlement and differential settlement occurs before the building is put into service. Connections to the building can then be made after most of the differential settlement has already taken place, which will ensure a reliable design for the connections affected by differential settlement.

b. The preload is also a full-scale load test of the foundation soils. Data obtained during preloading will provide a reliable relationship between settlement and load, which will be used to predict residual settlements of the structure.

c. The preload consolidates soft areas of clay fill, resulting in improved engineering properties of the fill. 5

As a result of the improved properties of the fill and based on the full-scale load test characteristic of the preloaded fill, a reliable prediction of upper limits of static residual settlement can be made. This will provide the assurance needed that unacceptable settlements will not occur during the life of the plant.

These settlements are conservative because they are based on stress levels in the fill beneath the building which are greater than the actual stresses imposed by the dead weight of the building alone.

The earthquake shakedown settlement estimates are conservative because the calculations assume that the sand is dry. Because the sand will never be dry, the presence of capillary forces in the partially saturated soil will reduce actual settlements below those predicted.

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Settlements due to permanent dewatering will be small ~~and will take place before final connections are made. This will allow for making permanent adjustments as required for the long-term operation of the plant.~~ approximately 1/2 inch

~~Provisions will be made~~

(1) H.B. Seed and M.L. Silver, "Settlements of Dry Sands During Earthquakes," Journal of the Soil Mechanics and Foundations Division, Proceedings of the A.S.C.E. (April 1972) pp 381-397

(2) R. Pyke, H.B. Seed, and C.K. Chan, "Settlement of Sands Under Multidirectional Shaking," Journal of the Geotechnical Engineering Division, Proceedings of the A.S.C.E., Vol 101, G.T. 4, (April 1975) pp 379-401

5

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TABLE 4-1

RESIDUAL SETTLEMENT (S) AND DIFFERENTIAL SETTLEMENT (ΔS) CRITERIA

Facility	Contribution to S and ΔS (inches)					
	Static 40-year Consolidation			Earthquake Shakedown		
		ΔS			ΔS	
	S	N-S	E-W	S	N-S	E-W
Diesel generator						
Building	1-1/2 (1)	3/4	1/2	1/2 (3)	1/2	1/2 (4)
Pedestals (5)	1-1/2 (1)	1/2	1/2	1/2 (3)	1/2	1/4
Borated water storage tanks	1 (2)	1/2	1/2	N/A	N/A	N/A
Diesel fuel tanks	1 (1)	1/2	1/2	N/A	N/A	N/A
Condensate tanks	1-1/2	3/4	3/4	N/A	N/A	N/A
Transformer pads	1	1/2	1/2	1/2	1/2	1/2

5

(1) Based on full scale test measurements. Was monitored by ~~To be ensured by scheduled measurements~~ (FSAR statement)

(2) Based on evaluation of settlement measurements at the diesel generator building. To be verified by direct measurements on the tanks.

(3) Calculated

(4) Could also occur along the northwest part of the building

(5) These pedestals will settle an estimated 1/2 inch because of foundation vibrations during operation of the diesels.

TELECOPY

BEBC- 2835

Bechtel Associates Professional Corporation

Inter-office Memorandum

To: J.F. Newgen

Date: April 4, 1979

Subject: Midland Plant Units 1 & 2
Job 7220
Moisture Requirements
for Plant Area Backfill

From: R.L. Castleberry

Of: Engineering

At: Ann Arbor

Copies to: File: 0274, C-210-PR, C-2645

W. Barclay
L. Basinski
S. Blue
L. Dreisbach
D. Himmelberger
L. Stornetta
K. Wiedner
Com Log

RECEIVED
APR 5 1979
BECHTEL POWER CORP.
JOB 7220
PER ____________

Reference: BEBC-2694 dated 2/5/79

This memo clarifies the instructions found in the referenced memo and calls your attention to Specification Change Notices 7220-C-211-9001, 7220-C-210-9001, and 7220-C-208-9003. This will also resolve CPCo's commitment made to the NRC regarding moisture content and proctor testing.

The following is a brief description of the requirements for controlling backfill and moisture content in the plant area as identified in the SCNs.

1) The moisture content of ±2% of optimum is the controlling value to be implemented only at the time of density testing.

2) Information moisture tests are to be taken prior to and during compaction at sufficient intervals to ensure that the moisture content will be within the specified range when density tests are taken.

3) Density tests are to be taken immediately after an area has been compacted unless otherwise directed by the onsite soil engineer.

4) An area is to be reworked/rejected at the time of density testing if the moisture requirement is outside the ±2% of optimum range, even if the fill has obtained acceptable density.

5) All cohesive soils are to be compacted to not less than 95% of maximum dry density as determined only by ASTM D 1557, Method D.

Bechtel Associates Professional Corporation

TELECOPY

6) The actual uncompacted lift thickness of the backfill material shall be determined by field personnel after evaluation of the proposed compaction equipment. However, in no case shall the uncompacted lift thickness exceed 8 inches for heavy self-propelled equipment, and 4 inches for hand-operated equipment.

7) Cohesionless material shall be compacted to not less than 85% relative density as determined by ASTM D 2049.

for [signature] R.L. Castleberry

JGH/pd
3/27/16

Tom

Need comments back ASAP.

Don Horn

— 81

From A.A.O.
~~A.Q.O.~~

Don Hain
Consumers Trailer

CONSUMERS POWER CO.
RECEIVED
AUG 2 1979
FIELD QUALITY ASSURANCE
MIDLAND, MICHIGAN

PERSONNEL QUALIFICATIONS

As part of the cause analysis the education and experience of personnel involved in the soils operations at the Midland Job site were reviewed.

This review indicated that during the course of the Midland Project soils operation (7/73 to date of review) 51% of the personnel assigned to soils had at least an M.S. in civil or soils, or a B.S. plus one or more years of soils experience, or an equivalent combination of education and experience. This includes Bechtel QC Inspectors, Bechtel QC personnel doing reviews only, Canonie QC, U.S. Testing technicians, Bechtel Field Engineers, and Bechtel supervisors.

whats does this mean? - what level? what did he do?

This indicates that the personnel involved in the soils operations had sufficient education and experience to carry out the tasks assigned to them.

In addition, the review indicated that except for the initial period (7/73 - 1/75) when all personnel were 'new employees', an average of 39% of the senior soils people (described in the previous paragraphs) continued on from one period to the next. For the lower level soils personnel, 38% continued from the initial period over into the 1/75-10/76 period, but only 8% continued on into the 10/76-present period.

Many senior soils personnel were retained during the 1975 slowdown but there was a need to restaff with mostly new lower level personnel in 1976 to support the reactivation of soils activities. This resulted in some decrease in the average experience level of personnel, but sufficient qualified, experienced personnel were available at all times, especially when recognizing that the major portion of the soils work had been already completed.

Based on the foregoing, we have concluded that the qualification/experience level of personnel assigned to the Midland Project soils operations was not a probable (contributing) cause of the settlement problem at the Midland Jobsite.

Comment - How involved were the "Qualified" people in the soils work on a daily basis? It may be that the qualified people were not actively involved. Also are labrers included?

* GRANTED THE MAJOR VOLUME OF SOIL WAS PLACED BUT IN HINDSITE WE NO LONGER HAD ONE ~~ALONE~~ BACK SITE, ... MORE ... A IF YOU CONSIDER THE FACT THAT THE MULTITUDE OF LOCATIONS WITH SOILS WORK IN PROGRESS INCREASED TREMENDOUSLY, THEREBY INCREASING THE NEED FOR NO. OF EXPERIENCED QUALIFIED PERSONNEL. [illegible] 8-21-79

x

Question 2

Discuss the consideration given to, and estimate the cost of, grouting any natural lacustrine deposits (sands) upon which safety-related structures are founded.

Response

7 Consideration will be given to grouting any natural lacustrine sand deposits that would be susceptible to liquefaction. Borings made to date indicate that these materials are isolated and have only been identified in one boring at the service water pump structure. Additional Borings will be made to identify the extent of this material. A grouting program would cost an estimated $250,000 for the cantilevered portion of the structure.

Question 3

During the meeting on March 5, 1979, you stated that on August 21, 1978, construction survey data indicated a settlement approaching the maximum value given in FSAR Figure 2.5-48. However, your response to staff request 362.12 by FSAR Revision 18 states, "In July 1978, the settlement of the diesel generator building exceeded the anticipated values shown in FSAR Figure 2.5-48." Clarify this apparent inconsistency.

Response Check against letter to Keppler

An error has been noted in the response to Question 362.12 in FSAR Revision 18 dated February 1979. This response derived from the MCAR 24 interim report dated September 29, 1978, states that "the diesel generator building settlements were noticed to exceed anticipated values in July 1978." The "anticipated values" referred to in this report were not the "estimated ultimate settlement" values given in FSAR Figure 2.5-48. Instead, these "anticipated values" were merely values of settlement that were greater than the amount of settlement which would have been expected under usual conditions for the elapsed time. The preparer of the FSAR response to Question 362.12 revision 18 erroneously combined these two unrelated values.

The actual course of events of the diesel generator building settlement are as follows.

On July 7, 1978, construction survey personnel noted difficulty in closing a level circuit when laying out survey control markers for continued construction of the diesel generator building. A survey check was made against existing survey control marks in the building on July 10, 1978, with a settlement of 1.7 inches being the largest noted. On July 22, 1978, the first formal 60-day settlement reading required by Specification 7220-C-76 for the diesel generator pedestal was taken. This survey indicates that the diesel generator Number 4 marker has settled 0.135 foot as the worst case. In processing this data, Bechtel surveyors noticed a larger settlement than anticipated. The processed survey data was transmitted to project engineering on July 26, 1978. The combined results of the July 10, 1978, and July 22, 1978, readings prompted construction survey personnel to monitor the building settlement in excess of Specification 7220-C-76 frequency requirements. On August 21, 1978, a construction survey check of the elevation of the northeast anchor bolt top on the eastern diesel generator pedestal showed a settlement of 3.25 inches, which is in the range of the estimated ultimate value in FSAR Figure 2.5-48.

24

FSAR Figure 2.5-48 shows estimated ultimate settlement in the interior of the diesel generator to be 3.2 inches. The north corners of the diesel generator building have an estimated ultimate settlement of 3.0 inches, while the south corners have an estimated ultimate settlement of 2.8 inches.

Based on the survey results of August 21, 1978, Bechtel nonconformance report NCR 1482 was issued on the same day. The NRC resident engineer was immediately advised of this settlement condition on an informal basis. An exploratory boring program was initiated on August 25, 1978. An evaluation of the preliminary boring data was made by Bechtel engineering on September 6, 1978. This evaluation indicated that the settlement condition was reportable under the requirements of 10 CFR 50.55(e). On September 7, 1978, CPCo made an oral 10 CFR 50.55(e) report to the NRC, followed by five written interim reports submitted to date.

Settlement of the diesel generator building and pedestals are being monitored by using preset markers and not using anchor bolts whose elevation may have been dislodged during the placing of concrete. Therefore, we do not consider the settlement readings based on the anchor bolts a true indication of the settlement. This was the data used on August 21, 1978, and identified on NCR 1482.

Question 4

Specify and justify the acceptance criteria which you will use to judge the acceptability of the fill, structures, and utilities upon conclusion of the preload program. Compare these criteria with that to which the material was to have been compacted by the original requirements set forth in the PSAR. The response should consider all areas where preloading is either planned or in progress (i.e., diesel generator building, borated water storage tanks, diesel fuel oil storage tanks, Unit 1 transformer, condensate storage tanks, and others still under evaluation). Describe how conformance to these criteria will result in assurance that unacceptable residual settlements cannot reasonably be expected to occur over the life of the plant. For each such area, state the extent of residual settlement which will be permitted and the basis for each limit.

Response

Make clearer post-preload calc., etc.

Acceptance of each surcharge program will require that the structures and utilities withstand the dynamic design criteria established in the PSAR and twice the predicted long-term total and differential settlements. This may require redesign of foundations and/or other remedial work. The resulting long-term settlement and bearing capacity predictions will be compared to the requirements set forth in the PSAR after completion of the surcharge programs. Surcharge programs are not expected to compress the fills to the densities associated with the compaction criteria set forth in the PSAR. Review with NRC Staff & amend FSAR

Criteria to be used to determine the acceptability of the fills, structures, and utilities upon conclusion of preload programs will be based on their behavior during preloading. This behavior will be monitored by measuring movement of the structures and/or borros anchor settlement rods and settlement plates placed in the fill and the buildup and dissipation of excess pore water pressure measured by piezometers placed throughout the fill. Movements of selected piping will be monitored before, during, and after preloading to ascertain the effects of loading. Duct banks will be evaluated based on verification that they are functional by field testing. ? Rate of settlement will be evaluated based on consolidation-rebound curves to predict additional settlement that will occur after surcharge removal under final loading conditions. Rep. #5 MCAR 24 Expected dynamic soil-structure behavior will be evaluated based on stress-strain moduli at low strain levels measured during rebound and shear wave velocity measurements from cross hole tests to be conducted in the fill material.

The extent of residual settlement that will be allowed for each structure to be surcharged will depend on the extent of settlement each structure experiences during surcharging, and therefore cannot be established at this time. This information will be forwarded to the NRC by ____________. The surcharge program for the diesel generator building is in progress. Sands susceptible to more limiting liquefaction will be grouted, densified by other means, removed, and replaced, or gravel drains will be installed to prevent pore pressure buildup after surcharge removal. The location of surcharge instrumentation is shown in Figure 4-1. Soil and building response data from the measurements performed to date are summarized in Figures 4-2 through ________. Results of monitoring selected utilities are shown in Figure _______.

A preload program is planned for one or both of the borated water storage tanks. The condensate storage tanks will be constructed, filled, and monitored for settlement. The Unit 1 transformer area will be surcharged prior to completion of construction. The diesel fuel oil tanks have been filled and are currently being monitored to determine any need for surcharging or other remedial action. Acceptance of these diesel fuel oil tanks will be based on a design to withstand those settlements experienced, plus double the future predicted settlement. If designs cannot allow for this settlement, the tanks will be surcharged prior to making piping connections or removed and replaced.

X

Question 5

To what extent will additional borings and measurements be taken after completion of preloading programs to ascertain that the material has been compacted to the original requirements set forth in the PSAR.

Response

It is not expected that material properties of the surcharged fills will reach those properties associated with compaction requirements set forth in the PSAR. Material properties will be evaluated based on settlement-rebound measurements made during and after removal of surcharge loads. and after review ~~of~~ with board & state in FSAR For these reasons, it is not planned to make borings or associated measurements after surcharge removal.

Keeby

Question 6

You propose to fill the borated water storage tanks and measure the resulting structure settlements.

(a) On what basis do you conclude a surcharge no greater than the tank loading will achieve compaction to the extent intended by the criteria stated in the PSAR? What assurance is provided by the technique that residual settlement for the life of the plant will not be excessive?

(b) A similar procedure is proposed for other tanks, including the diesel fuel oil storage tanks, and should also be addressed.

(c) The borated water storage tanks have not yet been constructed and are to be located upon questionable plant fill of varying quality. Provide justification why these safety-related tanks should be constructed prior to assuring the foundation material is suitable for supporting these tanks for the life of the plant. For example, can the tanks be removed with reasonable effort without significant impact?

How can you defend this.
DEH

Response (to 6a)

The results of field explorations in the borated water storage tanks area generally indicate satisfactory fill. To date, 18 borings have been taken in this area. Three of these borings indicate some soft materials. However, based on three borings per tank, there has been no identified unsatisfactory material directly beneath the borated water tanks.

To further evaluate if the fill in the area is satisfactory, an earthen preload on the west borated water storage tank area will be performed prior to construction of the tank. The existing tank ring and valve pit will be monitored to predict future settlement, and to allow remedial action, if any, before the tank is constructed. For the east borated water storage tank, a preload (either using earthen materials or filling the tank after construction) will be performed. The selection of the method chosen will be based on the results from the preload of the first tank.

It is expected that the preloads, together with the majority of the boring results, will confirm the adequacy of the foundation materials in this area. The preloads will also allow prediction of the residual settlements expected for the life of the plant.

Response (to 6b)

The diesel fuel oil storage tanks have been filled and are being monitored for settlement to predict future settlement and assess the need for remedial work required to ensure limited residual settlement. These tanks are supported on medium to very stiff sandy clay and clean sand fill. These tanks are surrounded with backfill consisting of very loose to dense clean sands and very soft to stiff clays. These adjacent materials do not meet PSAR requirements. Locations of borings made in this area are shown in Figure 9-1. A cross section summarizing the results of these borings is shown in Figure 6-7. If results of the evaluation made on these tanks cannot enusre limited residual settlements, the tanks will be surcharged or removed and reconstructed. The loose snad fill will be grouted.

Response (to 6c)

As described in the response to Part a, one or both borated water storage tank areas will now be preloaded before the tanks are constructed, using an earthen surcharge load. No significant foundation problems are anticipated, and the preload on the west tank is expected to confirm this. If necessary, an earthen preload will also be performed on the east tank. Although removal of the tanks after construction would be both costly and require a schedule delay, the tank are accessible and removal remains a viable alternate if unexpected future foundation problems in this area necessiate remedial actions.

Question 8

What tolerance is placed upon the alignment of the diesel generators and upon what is this limit based? How will the present differential settlement of the diesel generator pedestals be corrected? Discuss the extent and rate of residual settlement of the diesel generator pedestals predicted over the life of the plant. In view of the variability of the foundation material indicated by Bechtel's Interim Report 4 to MCAR 24 which was forwarded by your letter of February 23, 1979, how can long-term differential settlement be predicted with sufficient confidence to assure reliable start-up and operation of the diesel generators when needed? What surveillance program (and inspection frequency) for the pedestals do you intend to conduct to assure detection of misalignment before these limits can be reached? What corrective action, and the basis therefore, do you propose if these limits should be approached?

Response

The tolerances of the shaft alignment of the diesel generators are based on the manufacturer's recommendations. According to Delaval Turbine, Inc. of Oakland, California (the manufacturer of the four identical diesel generators), a 5-degree combined tilt and roll will have no effect on the performance of the engine and generators (confirmation awaiting). The present tilt and roll is less than 0.2 degrees. The diesel generators at Midland are similar in design to marine engines designed and manufactured by Delaval Turbine, Inc. which are subjected to tilt and roll larger than 5 degrees at more frequent cycles.

The effects of the differential settlement of the pedestal on the fuel oil drip return line could cause oil to leak around the fuel oil injectors. This is a housekeeping problem and not a safety problem. 36

The established nozzle allowables (force and moments or displacement) for the piping system at the interface of the diesel generator are within acceptable limits and are not expected to exceed these allowables based on a maximum tilt and roll of 5 degrees. Instrument tubing and electrical wiring have sufficient flexibility to not be a problem for the specified tilt and roll.

Figure 8-1 is a graphical representation of the time settlement rate of the diesel generator pedestal corners. Weekly settlement values are indicated on the chart. As of March 16, 1979, pedestal 2 had the greatest tilt at 0.089 and 0.087 feet and the greatest combination of tilt and roll at 0.078 and 0.089 feet. Pedestal 4 had the greatest roll at

0.058 and 0.034 feet and the greatest settlement of 0.449 feet. Figure 8-2 identifies settlement values at their respective corners along with tilt and roll.

The engine and generator are located on one continuous independent foundation. The dimensions of the four identical foundations are shown in Figure 3. The foundation for the diesel generator is a reinforced concrete structure having a minimum compressive strength of 4,000 psi. The dimensions and composition of the pedestal are such that it has enormous bending and torsional stiffness. Therefore, the pedestal will act as a rigid body, with the top of the pedestal within one plane and not a warped surface. As evident from Figure 8-3, all four corners of the pedestal lie on one plane within the survey accuracy of .01 foot.

Following is a list of options available to correct the differential settlement of the diesel generator pedestals.

1) Use as is. The shaft alignment between the engine and generator can be maintained with no adverse effect on safety because the engine and generator are in the same plane.

2) Add a layer of grout to provide a horizontal drive shaft position. This option is limited by the maximum grout thickness.

3) Remove the first few inches of concrete from the pedestal block and replace it with a top layer of concrete to provide a horizontal surface. This option may be used when the grout limit in Item 2 is exceeded.

4) Pressure grouting under the pedestal to bring the pedestal up to a horizontal position.

The actual method of modification will be determined when the settlement data are evaluated after the preload is removed.

The weight of the pedestal and the surcharge load now being applied on top of the pedestal area is at least two times the total weight of the operating diesel generator and pedestal. The purpose of the surcharge operation is to consolidate the fill material in and around the diesel generator building and reduce the residual settlement during the plant life. Based on the settlement data recorded during preload, the maximum differential settlement is expected to be within the original design requirements.

The points presently being monitored for settlment on the pedestal corners are the same points to be used for the foundation settlement data survey. It is required that these points be monitored on a 60-day cycle throughout

Operation Tech Spec.

the construction phase and for the first year of operation. ?
After 1 year of operation, the frequency will be reviewed
and possibly modifed. If the actual settlement exceeds the
estimated settlement, realignment of the diesel generator
may be necessary.

Z

Question 9

Based on the information provided in your Interim Report Number 4, it appears that the tests performed on the exploratory borings indicate soil properties that do not meet the original compaction criteria set forth in the PSAR and specification for soils work. Provide assurance that the soil under other Class I structures not accessible to exploratory borings meets the control compaction requirements.

Response

Soil properties of fill beneath Class 1 structures not addressed in Interim Report 4 have been evaluated by making additional borings in selected areas. Results of these borings indicate that backfill beneath a portion of the service water building and portions of the auxiliary building do not meet compaction requirements set forth in the PSAR. In the auxiliary

feedwater iso valve pit

building area, borings beneath the electrical penetration rooms E&W?

and railway bay indicate that remedial work as discussed in the response to Question 12 will be required. Other portions of the auxiliary building are currently being studied.

Keeley - Make more positive.

Show lean conc. where compacted fill was refer to conc. records.

Question 10

You have stated that the fill is settling under its own weight. What assurance is provided that the fill has not and will not settle locally under structures with rigid mat foundations, such as portions of the auxiliary buildinj or service water pump structure.

Response

If the potential for settlement of the fill under its own weight exists, remedial measures will be taken to provide adequate support. The service water pump structure and Unit 1 electrical penetration room will be underpinned. Other portions of the auxiliary building on fill are still under investigation.

When?

×

Question 11

In view of the variations indicated by present borings, what assurance exists that vertical borings taken adjacent to structures are sufficiently representative of fill conditions under the structure?

Response

The initial borings were intended for an early evaluation of the overall plant fill. These borings were generally in more accessible locations (i.e., immediately adjacent to, rather than within, the structures). During the last 6 weeks, additional borings were made through the structural slabs, which allows an evaluation of foundation materials directly beneath the structure (e.g., borings taken were within the service water pump structure, electrical penetration areas,
tower feedwater iso
control area, and railroad bay of the auxiliary building). ?
These additional borings, correlated with the previous borings taken from the structure periphery, will be used to define the fill conditions.

Rewrite!

Question 12

Document the condition of soils under all safety-related structures and utilities founded on plant area fill or natural lacustrine deposits. Based on the results of investigations, compare the properties and performance of existing foundation materials under all expected loading conditions with those which would have been attained using the criteria stated in the PSAR. If the foundation materials are found to be deficient, discuss measures that will be taken to upgrade them to cirteria stated in the PSAR.

Response

?

Soil conditions beneath safety-related structures and utilities are summarized on Table 12-1. This table refers to evaluations and/or remedial work to be done in each area. Remedial measures may not necessarily cause PSAR compaction criteria to be achieved, but will provide adequate support for the structures and utilities.

Table 12-1 references which borings were made in each area and cross sections summarizing these borings that are attached in Figures _____ through _____.

TABLE 12-1

	Supporting Material	Remedial Measures Planned	Other Remedial Work Under Consideration
Auxiliary Building			
Control Tower	Clay and/or sand fill and concrete	Being studied	Underpinning and/or grouting
Unit 1 Penetration Room	Clay and sand fill	Underpinning	Grouting
Unit 2 Penetration Room	Clay and/or sand fill	None	Underpinning and/or grouting
Unit 1 Access Shaft	Clay and sand fill	Underpinning	None
Unit 2 Access Shaft	Clay and sand fill	Underpinning	None
North End (Railway Bay)	Sand fill	Grouting	None
Service Water Building			
Portion Adjacent to Pond	Natural soil	None	None
Cantilever Portion	Clay and sand fill	Underpinning	Grouting
Diesel Fuel Oil Storage Tanks	Clay and sand fill	Surcharging	Removal of tanks
Service Water Pipes	Clay fill	None	Removal
Retaining Wall	Clay fill	None	None
Diesel Generator Building and Associated Utilities	Clay and sand fill and concrete	Surcharge fill grout loose sands	Connecting building and pedestals into a mat foundation
Tank Farm (Borated Water Tanks)	Clay and sand fill	Being studied	Surcharging

Question 13

How ahs the lack of compaction and the increase in soil compressibility affected soil-structure interaction during seismic loading and, therefore, the seismic response spectra in design?

Response

Seismic Category I structures, which where founded fully or partially on compacted fill were reexamined to determine the impact of lack of compaction and increase in soil compressibility on the soil-structure interaction and the seismic responses. The results of this evualation for each building and the underground utilities ~~utilities~~ follows:

1) Diesel Generator Building

The diesel generator building foundation rests entirely on compacted fill. A seismic reanalysis was conducted to account for the effect on soil-structure interaction due to both the degree of compaction and increase in soil compressibility.

The technique of analysis, as well as the computer programs utilized, are the same as those specified in the FSAR. The structural and soil properties are also the same, with the exception of shear wave velocity (V_s) and soil density (ρ).

The analysis considered fill ranging from soil with V_s = 400 ft/s and ρ = 120 pcf to soil with V_s = 1,359 ft/s and ρ = 135 pcf (natural soil).

Floor response spectra were generated and response spectra envelopes were developed for soil with a shear wave velocity in the range of 500 to 1,359 ft/s. Typical response spectra envelopes are attached in Figures _____ to _____.

Review of equipment qualification and diesel generator building design will be undertaken to the enveloped seismic responses.

2) Service Water Pump Structure

The service water pump structure foundation consists of two portions. At the lower elevation, a foundation mat (73'-11" by 90'-0") is founded on natural soil. At the higher elevation, a foundation mat (36'-1" by 86'-0") is founded on sturctural backfill.

A seismic reanalysis was conducted, taking only the foundation founded on the natural soil for soil-structure interaction computation. For the purposes of this analysis, the soil structure interaction effect from the higher elevation foundation media has been ignored.

The portion of structure founded on the structural backfill was assumed to be unsupported and as an extension of the major structured system founded on natural soil. A nominal soil dynamic modulus of elasticity of 22,000 ksf and a Poisson's ration of 0.42 were used as uniform foundation media properties to compute the soil impedance functions for this foundation. The seismic analysis technique, criteria, and programs used follow those specified in the Midland FSAR. Torsional response due to the eccentricity presented was estimated to be small in comparison to the response contributed by rocking and translational motions. Torsional loading will be considered in the design of the structure by the application of the design horizontal seismic loadings obtained form the decoupled seismic system at its eccentricity. A 15% increase in both magnitude and spectrum widening at the calculated torsional frequency was used to generate the floor response spectra.

Comparison of the seismic loading and typical floor response spectra between the modified foundation seismic analysis and those used in the original equipment qualification and structural design are shown in Figures _____ to _____.

3) Auxiliary Building

The structural backfill is only situated under a portion of the auxiliary building foundation, under the control tower and its adjacent wings. The rest of the auxiliary building foundation is founded on a natural foundation media, with a nominal shear wave velocity of 1,359 fps used in the analysis. A composite foundation lumped parameters, taking account of both compact and natural soil, was used for the soil-structure interaction analysis. An evaluation of the compacted soil properties which varied from V_s = 850 fps used in the original analysis to 500 fps, indicated that the impact to the overall lumped soil parameter is insignificant because its effect would be enveloped by the spectra widening.

4) Underground Utilities

(Later)

Question 14

For all seismic Category 1 structures (including, but not limited to, the diesel generator building) which are located on fill provide the results of an evaluation showing which structure you predict may experience settlements in excess of that originally intended, and provide an evaluation of the ability of these structures to withstand the increased differential settlement. For the diesel generator building and/or any seismic Category 1 structure which exhibits cracking, evaluate the effects of the existing and/or anticipated cracks on the performance of the intended function of these buildings. The calculated stresses for seismic Category 1 structures at critical locations should be tabulated and compared to that of allowable stresses as stated in the appropriate ACI Codes.

Response

The Seismic Category I structures located completely or partially on fill are identified in Figure 14-1. Table 14-1 provides the data regarding the present measured settlement and the maximum predicted ultimate settlement for these structures. Note that measured values do not represent the total settlement of the building from the start of their construction. They only represent settlement since the markers were installed during construction. Attempts at being made to obtain estimates of total settlement from the initial building construction stage based on construction records of scribes and/or anchor bolts.

As evident from the data presented in Table 14-1, except for the diesel generator building, the recorded settlements of other structures do not approach the ultimate settlement values.

The ability of these structures to withstand differential settlement is discussed in response to Question 15. No formal evaluation has been performed for differential settlement within a structure. All Class 1 structures except the diesel generator building are considered to be rigid and will therefore undergo rigid body motion without evidencing critical stresses. The differential settlement within a building as shown on FSAR Figure 2.5-48 does not include the effect of building structure stiffness, and therefore is not relied upon for building stress evaluation.

The diesel generator building, service water building, and parts of the auxiliary building (railroad bay, wings, and control room area) have been examined for cracks in the main structural elements. The identified cracks have been mapped. They are presented in Figures 14-2, 14-3, and 14-4. Also shown on these figures are the possible location of the anticipated structural cracks and their cause.

The structural cracks in the diesel generator building are in the areas around the vertical electrical duct banks. They were caused by the estimated 1,000 kips of load transmitted to the duct bank. Since then, the concentrated load has been eliminated by cutting the duct bank and providing a 12-inch slip joint. For details, refer to the response to Question 7.

In the service water structure, the cracks are probably caused by the cantilever action of the northern part of the structure as shown in Figure 14-5. It is theorized that the cracks on the roof slab are due to the bending tension and on the walls are due to principal tension caused by shear.

No significant cracking has been noticed in the auxiliary building yet.

A crack in concrete indicates that the tensile strength capacity of concrete has been exceeded. Because no reliance is placed on concrete tensile strength in design for bending and axial tensile and calculations, the strength of the structure is not affected by the crack to resist these forces. The compressive forces can be transmitted through the crack by bearing and shear force by aggregate interlock or shear friction. Moreover, the stresses in these walls are small and only a fraction of the permissible stress is summarized in Table 14-2. Therefore, the cracks do not adversely affect the safety of the structure.

A large crack, especially when exposed to weather, can cause corrosion to rebar and consequent damage to the structure. To prevent damage, cracks larger than __ in exterior walls exposed to weather and ____ inside the building will be repaired using approved material and procedures per project specifications. The limiting widths of the cracks chosen for repair are based on ACI __ recommendation and industry practice.

A preliminary analysis has been performed based on the present deflected shape of the diesel building taking into account the different stages of construction when settlement was recorded. The stresses are summarized in Figure 14-6. A detailed analysis will be performed upon completion of the preload program to determine the stresses due to the differential settlement.

For the service water pumphouse and the auxiliary building, Aux wing (part.) no significant differential settlement has been noticed. It is therefore assumed that the structues on inadequately compacted fill are cantilevering from the part located on original soil or properly compacted fill. The loads and structural capacities are summarized in Figures 14-7 and 14-8. However, the foundations of these structures will need repair to provide adequate supports. The details are discussed in response to Question __. ?

Question 15

For all seismic Category 1 structures which are partially located on fill and partially located on glacial fill or original soils, provide a detailed evaluation of the ability of these structures to withstand the differential settlement. The possibility of not having a contact surface between the structures and the fill due to settlement ocurring prior to or during a seismic event should be considered over the life of the plant.

~~Response~~ Response

An investigation is presently underway to verify the foundation condition of all Seismic Category I structures which are partially or fully supported by fill material. This investigation, which is summarized in Answers _____, has shown that some areas (other than the diesel generator building) do not have sufficient bearing strength.

These areas will be modified to meet the required bearing strength by one of the following techniques.

1) Grouting of the foundation material to cause compaction of the material and increased bearing capacity

2). Removal of existing material and replacement by a lean concrete mix.

3) Piling is being considered, but only for use as a vertical support member (it will not be relied upon to supply horizontal resistance)

For structures which have sufficient bearing capacity, but a difference in foundation stiffness under various portions, analysis will be performed to determine the strains which may exist. Additional load combinations will be used which include settlement effects. These load combinations will be used with higher allowables because settlement is a self limited secondary effect. It is common to use higher allowables when self-limited effects are combined with real (mechanical) loads. This is illustrated in the ASME Code, Section III, Division 2 when self-limited thermal loads are combined with real loads.

For normal operating loads, which includes dead and live load, the structures will be checked to verify that the calculated strains do not exceed ng 90% of yield when real loads are combined with settlement effects. For this condition, all load factors will be 1.0. This requirement will ensure serviceability throughout the life of the structure.

For extreme factored conditions such as earthquake and tornado, the steel reinforcement strains will be limited to twice

that associated with

yield when settlement effects are combined with factored real loads. This criterion is applied to gross structural behavior and is not applicable to local areas subjected to tornado impact and pipe rupture effects.

?

For fill material that has been verified as acceptable by the boring investigation program, there is no expectation that it should settle during a seismic event for the following reasons.

acceptable criteria?

1) The fill was properly placed as verified by the blowcount measurement.

2) It was loaded by the construction of the structure with no unexpected settlement.

DEH

3) It was saturated with water after filling of the pond. For clays with poor compaction this would have created compaction and settlement.

4) The ground motion is small (0.06g OBE and 0.12g SSE) and the resulting strains from an earthquake will be small.

5) The increase in bearing pressure due to structural rocking and vertical seismic response will be small

compared to the compaction pressure used during construction.

X

Question 16

Since the plant area fill is apparently settling under its own weight, what assurance exists that the fill has not and will not settle locally under piping in the fill, resulting in lack of continuous support and causing additional stress not accounted for in design?

Response

Not all pipes

The effect of fill settlement will be accounted for by evaluating the deflected shape of the pipes being profiled. Stresses will be evaluated as $\boxed{\text{desbribed}}$ in the response to Question 17. The local settlement due to lack of support from the plant fill will become apparent in the pipe profile and the profile will actually define the pipe responses to these local settlements, if any. Thus, the stresses developed from the deflected shape will represent the acutal stresses caused by load from local settlement and/or from lack of support.

Can this be done with water in pipe? How much flow

The deflected shapes are being measured before the pipes are placed into service. Therefore, to account for the extra dead load stresses induced by filling the lines with liquid, the deflected shape stresses will be amplified ? to account for this liquid or the deflected shape will be verified when the pipe is filled.

Borings & pipe flex. should be added

Question 17 Where is current condition?

Identify and document the current condition of all seismic Category I piping founded in the plant area fill. Include all piping founded in the plant area fill whose failure could adversely impact safety-related structures, foundations, and/or equipment. Also, discuss how code-allowable conditions will be assured throughout plant life. If any essential piping has now or should later approach code-allowable stress criteria or cannot be determined, what measures will you take to alleviate these conditions?

Response

Figure ______ identifies the Seismic Category I piping founded in fill, as well as non-Seismic Category I piping founded in fill, if the failure of such piping could adversely impact safety-related structures, foundations, or equipment. Table _____ lists the current construction status and the current status of the profiling program for these lines.

Several possible modes of failure were considered for Seismic Category I structures, electrical duct banks, and pipes to fail because of a failure in non-Seismic Category I piping. Jet impingement and pipe whip are not considered to be credible failure mechanisms because of generally low piping pressures, separation criteria, and the restraining effect of the soil. Hydrostatic forces due to flooding are effectively considered by designing Seismic Category I structures for the probable maximum flood. The mechanism which remains and was considered in our analyses is that of erosion or "washout" of the founding soil for Seismic Category I structures, pipes, or duct banks.

To determine which buried non-Seismic Category I piping could have a potential adverse impact upon safety-related structures, foundations, or equipment, the following procedure was used.

1) Zones were established to encompass each Seismic Category I structure, buried pipe, and duct bank. These zones are shown in Figure _____.

2) The non-Seismic Category I pipes founded in fill within each zone were identified and tabulated.

3) A "zone of influence" was arbitrarily established for each non-Seismic Category I pipe so identified. This zone of influence was determined by a subtended angle of 90 degrees (45 degrees on each side of the pipe) extending from the pipe centerline upward to the surface and downward for three pipe diameters, with a minimum

distance of 1 foot below the pipe. Where there are buried elbows in the non-Seismic Category I line, the extent of the zone of influence was increased to subtend an angle of 60 degrees on the outside of the bend.

4) Each zone of influence thus established was then examined to determine the extent of its containment of Seismic Category I structures, foundations, pipes, and electrical duct banks.

5) Each Seismic Category I structure, foundation, pipe, and duct bank was evaluated assuming that the portion which is in the zone of influence is unsupported (i.e., the supporting soil has been eroded sufficiently so that it no longer provides any support).

Make the above "was" compatable

When? 6) If this evaluation shows that no Seismic Category I structure, foundation, or equipment is adversely impacted by the failure of a non-Seismic Category I pipe or portion of pipe (as evidenced by not exceeding those stresses allowed by the governing code when the coincidence of the Seismic Category I structure, foundation, or equipment with the zone of influence of the non-Seismic Category I pipe is considered to be unsupported), the failure of non-Seismic Category I pipe or a portion of the pipe is adjudged to have no adverse impact on safety-related structures, foundations, or equipment.

Done

7) If a finding of "no impact on safety" cannot be made, then the affected non-Seismic Category I pipe is included in the pipe settlement evaluation and monitoring program and is shown in Figure _____.

The pipe settlement evaluation and monitoring program... (Detailed description to be provided by civil, including pipes to be profiled, pipes not profiled and justification therefore, application of the preload program to pipe monitoring, and basis for prediction of ultimate settlement.)

When the extent of final settlement is predicted (following the preload program) for each Seismic Category I pipe and other pipes whose failure could adversely impact safety-related structures, foundations, or equipment, a stress analysis evaluation will be performed for that pipe.

The stress analysis evaluation for each pipe will be performed in the following manner.

1) For pipes which have profiles available, an analysis will be performed using the observed displacements.

2) For pipes which are subjected to the preloading program and which will be reprofiled following the removal of the preload, a second analysis will be performed using

the observed displacements from the second profiling of the pipes.

3) Settlement data from the pipes which have been profiled twice will be used to predict the ultimate settlement of all of the piping founded in fill which is Seismic Category I or for which failure could adversely impact safety-related structures, foundations, or components. The method for predicting the ultimate settlement has not been chosen and will depend on the results of the preload program.

4) We propose ~~to~~ will use the allowable stress criteria in the 1977 version of the ASME Boiler and Pressure Vessel Code, Section III, Articles NC-3611.2(f) and NC-3652.3(b) to determine the acceptability of piping analyzed for ultimate settlement.

Based on our preliminary examination of the most severely deflected pipe identified to date, we do not believe that any piping has been overstressed when compared with the proposed allowable stress. If the results of our detailed stress evaluation show that portions of the piping have been overstressed, then those portions (probably elbows) will have to be removed and replaced.

If it is determined that the predicted ultimate settlement will lead to an overstressed condition, other corrective measures may also be considered (e.g., pressure grouting to return the line to a less deformed state).

Question 18

For all seismic Category 1 piping and all piping whose failure could adversely impact safety-related structures and/or systems, whether buried or not, describe what evaluations you plan to conduct to assure that such piping can withstand the increased differential settlement between buildings, within the same building, or within the piping system itself without exceeding code-allowable stress criteria. The potential influence due to differential seismic anchor movement should also be considered. Discuss what plans you have to assure compliance with code-allowable stress criteria throughout the life of the plant.

Response

Treatment of buried non-Seismic Category I piping, whose failure could adversely impact safety-related structures or systems, is presented in the response to Question 17 of this request. Failure of other non-Sesimic Category I lines which could adversely impact safety-related structures or systems is addressed in Chapter 3 of the FSAR, and includes high energey line break analysis, jet impingement and flooding studies, and design criteria for pipe whip and separation.

Therefore, only Seismic Category I, nonburied piping is addressed in this response. However, the evaluations described may also be applied to certain non-Seismic Category I piping as a matter of good engineering practice and in the interests of operational reliability.

Differential settlement between buildings has not been considered in the normal stress analysis performed for piping which traverses between the containments and the auxiliary building. However, it should be noted that most of these lines have not been connected at both ends yet, and are not normally connected until late in the construction sequence of the plant. Thus, most of the anticipated differential settlement takes place prior to at the time of connection. Provisions are incorporated in the piping installation specifications which require engineering resolution of any excessive misalignments so that these conditions do not go unnoticed.

A differential seismic allowance of 1/4 inch has been considered in the piping stress analysis. A reevaluation of the expected differential seismic movement is under consideration to determine whether the variance in soil properties will affect the seismic response of the structures.

A reexamination of the stresses in all of the Seismic Category I connecting piping between the auxiliary building and the containments is planned. This analysis will consider stresses induced in the piping by differential settlements between the buildings after connection of the piping, and will also consider the additional induced stresses due to the maximum expected differential settlement. For this evaluation, we propose to use the stress criteria discussed in the response to Question 17 to determine acceptability. Any piping shown by this evaluation to have already been overstressed will be replaced. Any piping which appears likely to be overstressed by the predicted maximum differential settlement will be modified by redesigning the pipe supports and/or the pipe itself. Pipes will be rerouted for increased flexibility if necessary to meet the stress criteria.

Surveyors ?

Differential settlement between the feedwater isolation valve structures and the containments is currently being monitored. The feedwater piping in these structures has a flexibility loop, so that exceeding the 3.0 S_c criteria because of differential settlement is extremely unlikely. However, a verification analysis similar to that performed for piping connecting the containments with the auxiliary building will be performed.

Except for the piping discussed above, Seismic Category I piping between structures is buried. Most of this piping has not been installed yet, and much of it enters the structures through sleeves which have clearances around the pipe. After connection, these gaps will be monitored to ensure that no excessive stresses are introduced into the piping systems. To relieve loads which are developed by differential settlement between buried piping and structures, pipe supports will be adjusted to relieve and distribute the loads. Any analysis of piping within the structures will be limited to the portion of piping between the first anchor inside the building and the buried pipe, and will be a part of the analysis discussed in the response to Question 17. What about Tanks & Piping?

Within Seismic Category I buildings, only the emergency diesel generators are founded independently from the building structure. Because this structure is currently in the midst of the surcharge program, no piping connections will be made between the diesel generator pedestals and the building structures in the near future. Most of this piping will be relatively small and will incorporate enough flexibility to accommodate more than the expected differential settlement. The air intake and exhaust ducts have expansion joints which serve to isolate the ducts from the diesel generator pedestals.

(CIVIL TO ADDRESS THE FLEXIBILITY OF STRUCTURES)

Structure deflections due to settlement variations under the structure are not expected to be of significance to piping systems within the structure. No reanalysis of the stresses in piping systems within a structure is anticipated due to these deflections.

The programs discussed are being initiated with the objective of ensuring that if settlements remain within the predicted range no further analysis, modifications, or monitoring will be required to maintain the settlement induced stresses within the limits imposed by the ASME Code. Only normal surveillance of piping and pipe supports is expected to be necessary.

No additional piping stress analysis has been performed yet. CPCo will give the NRC details of the plans when they have been developed, and will also provide summaries of the results of the analyses.

Question 19

The piping in fill under and in the vicinity of the diesel generator building could have deformations induced either prior to or during the preload program. What is the present status of any deformation in the piping, and what ultimate deformations are predicted. If any deformations are or will be excessive, what actions are being or will be taken to correct the condition?

Response

The all or most pipes which are located in the fill subjected to the influence of preloading the diesel generator building are listed in Table 19-1. Methods used to assess the condition of these pipes and the effects of the preload are profiling pipes with pressure devices, gap measurements, elevation survey, and analysis.

Following are discussions of each of these four categories.

1) Profiling Pipes with Pressure Devices

The pipes shown in Figure 19-1, SK-C-650, were profiled using a pressure registering device to determine the invert elevation of the deflected pipe.

A detailed discussion of the profiling technique can be found in the response to Question 17. The profile data from these pipes will also be used to evaluate other pipes in close physical proximity. The profiles taken to date were analyzed, and the stresses were low. The maximum bending stress was _ ksi. These pipes will be profiled again after the preload is removed.

The second profile will provide information to allow a correlation between additional overall settlement and additional deflection in the pipe. Any additional stress due to change in curvature will be calculated. This information will provide a relationship between additional stress and additional settlement. This relationship will allow for the prediction of stresses for future predicted settlements.

2) Gap Measurements

The gaps between penetrations and pipe entering the diesel generator building were measured at the top, bottom, and each side. The measurements were taken before the preload was applied and during the isolation of the electrical duct banks. These measurements did not change significantly, indicating that the pipes moved with the building during the building settlement subsequent to isolating the ducts. At present, none of the gap measurements indicate that the pipes are being deformed by the settlement of the diesel generator building because there are no cases where the gap between the top of the penetration and the pipe is zero. Additional measurements will be taken when the preload is removed. This information will be presented after the preload program is completed.

3) Elevation Survey

Bsof alp

By standard survey methods (i.e., level and transit), an elevation survey is being made of the condensate line, concrete encasement, and the line itself. Readings are being taken at the north and south end of the encasement. A time versus settlement curve and location are shown in Figure 19-2.

4) Analysis

Several lines which appeared geometrically sensitive to settlement and/or the preload were analyzed for an assumed settlement of 12 inches by the diesel generator building. The lines analyzed were the condensate lines entering into the turbine building, the circulating water lines, and the nonsafety-related service water line entering the turbine building.

After studying the results of this analysis, the following changes were made.

a) The condensate lines were disconnected at the turbine building to relieve the stress buildup cause by the differential settlement between the diesel generator building and the turbine building.

b) The roundness of one of the circulating water lines was measured to see if internal reinforcement is needed during the preload.

c) Profiling of the service water lines was extended to provide deflection information along this section of the line.

The roundness measurements taken to date on the circulating water line indicate that the pipe is generally taller than it is wide, giving no indication that reinforcing is needed.

Depending upon the performance of the backfill material during the preload program, the predicted settlement for these pipes appears to be small. The stress due to this settlement will be calculated as described in response to Question 17. The additional stress induced by the settlement which can be accepted before exceeding code allowables will be compared to the stress caused by the existing deflections, thereby predicting the total acceptable settlement.

Excessive deformations will not be acceptable for any safety-related pipe. Safety-related pipes must satisfy the procedure described in Question 17. For example, deformation ovalling in piping will be evaluated to determine if the ability of the pipe to perform its intended function or its structural integrity is imparied. Normally, ovalling of 2 to 5% is accepted for buried pipe. If a pipe cannot meet the applicable criteria, the pipe will be abandoned and relocated or reinforced to comply with the criteria.

A complete evaluation of all safety-related piping, including the completion of Table 19-1, will be presented after computing the preload program. It is estimated that this information will be presented to the NRC in June 1979.

TABLE 19-1

Serv should be included

Pipe Identification		Safety-Related
Pipes entering diesel generator building.		
1HBC81, 82	8"Ø	Yes
2HBC81, 82	8"Ø	Yes
1HBC-310, 311	8"Ø	Yes
2HBC-310, 311	8"Ø	Yes
1HBC-497	2"Ø	Yes
2HBC-	2"Ø	Yes
1HBC-	1-1/2"Ø	Yes
2HBC-	1-1/2"Ø	Yes
2GBF-341	4"	No
1JBD-437	8"	No
1JBD-537, 538	3"	No
2JBD-537, 538	3"	No
XHG	6"	No
0YBJ-13	12	No
2YBJ-8	12	No
Pipes in vicinity		
0HBC-27, 28	10	Yes
0HBC-53, 54, 55, 56	26	Yes
1HCD-169	20	No
2HCD-169	20	No
1HCD-513	6	No
2HCD-513	6	No
1JBD-1, 2	26	No
2JBD-1, 2	26	No
1JBD-437	8	No
0YBS-13	12	No
2YBS-8	12	No
CIP	6"	No
Circulating water	96"	No
Circulating water	72"	No
Oily waste		No
Sanitary sewer		No

Check with Mech. (Matrix)

Question 20

Provide assurance that the stress levels of all components (e.g., pumps, valves, vessels, supports) associated with seismic Category I piping systems that have been or will be exposed to increased settlement will be within their code-allowable stress limits. Also, provide assurance that deformations of active pumps and valves installed in such systems will be kept within limits for which component operability has been established.

Response

The analysis of Seismic Category I piping systems which have been or are expected to be affected by settlement will encompass the total extent of the settlement effect on the ~~whose differ. settlements? Clarify!~~ piping. Affected pump and nozzle loadings will be analytically checked to verify that they are within specified or vendor-accepted limits. Flanged joints may be disassembled if necessary, and the nature of the resulting separation may be used to evaluate the loads transmitted by the joint.

Equipment supports are normally designed to accept the allowable piping reaction loads, and therefore will be unaffected by settlement as long as the nozzle allowables are not exceeded.

For piping systems which have been exposed to additional loads induced by settlement, piping support loads will be verified to be in accordance with the design loads by analysis. The expected maximum differential settlement will be used to verify that pipe support loads will not become excessive, or alternately, to establish a requirement for future support recalibration. How done not understood.

For flanged pumps and valves which may have been exposed to settlement-induced effects, flanges will be disassembled to determine the magnitude of the reaction load. After verifying that this load is acceptable, the piping system will be recalibrated (if necessary) to minimize the loads, and the flanged joint will be reassembled. Using the expected maximum differential settlement, the system will be analytically examined to determine whether the potential induced loads are acceptable or whether to establish a requirement for future recalibration.

For the few systems with installed, welded-in valves which may have been subjected to high loadings induced by settlement, an analytical evaluation will be used to demonstrate that the valves have not been subjected to deforming loads. If this cannot be determined, the valve will be physically examined to determine if it has been unacceptably deformed. However, the valves are generally stronger than the piping to which they are welded, and deformation will occur first in the piping system at areas of stress concentration, such as elbows.

Question 21

Your letter of December 17, 1978, on the settlement of the diesel generator foundations and building advised us that the use of a preload to densify the existing fill material in place had been selected as the major corrective action plan. Bechtel's Interim Report 3 to MCAR 24 forwarded by your letter of January 5, 1979, identifies six alternative plans for corrective action, from which your soil consultants have advised that only two suitable options exist at that time (i.e., the preload option or the option to remove and replace the building and fill material). We require the following additional information regarding the basis for selection of these two options:

(c) Discuss for each option the probability of achieving the degree of compaction intended by the original requirements stated in the PSAR.

(d) What other significant factors influenced your selection?

Response (to Part 21c)

The preload option may not produce densities uniformily meeting the PSAR compaction criteria, but will produce foundation conditions suitable for supporting the diesel generator building as discussed in the response to Question 4.

Removal and replacement of the diesel generator building and/or fill would have allowed achievement of the PSAR compaction criteria.

Response (to 21d)

Listed below are other factors that influenced the choice of the preload over the replacement option.

1) Defining the Limits of Soil (e.g. influencing the diesel generator building foundation)

It might be necessary to remove soil from beneath adjacent structures (turbine building transformer pads, and steam water valve pits). Removal of this soil could pose safety problems for the structures and the personnel working the the area. Even if it were not necessary to remove this soil, the excavation would be close enough to these structures, requiring extensive protective measures.

2) Construction Ease

The large excavation required for the replacement option would interfere with other construction. In addition, it would be difficult to do the earthwork because of the high water table in the area.

3) Interface Problems

Problems would be encountered with compacting the new fill in the deep excavation. The high water table and constant dewatering would make it difficult to control the excavation slopes and fill moisture content required for compaction. The building and utilities would still experience settlement (originally predicted at 3.2 inches), whereas at the completion of the preload program, the building and utilities will attain most of the total lifetime settlement. Utilities running from the fill which is left in place to the new fill could experience more deformations than those already experienced because there would be no gradual transition between the two zones.

Question 22e

For those activities identified in response to Item d above, identify each which is significant in terms of weight addition to structures founded totally or partly on or in fill.

Response (to 22e)

The construction activities within the various safety-related structures scheduled to be completed during the next 24 months are identified in response to 22d above. The estimated weight in place and weights to be added during this construction period are compiled in Table 22e-1. The weights to be added to the borated water storage tanks are signficant. However, for the other structures, the weight to be added to complete the construction is found to be minimal.

TABLE 22(e)-1

	Structure/Component	Estimated Total Weight In Place (kips)	Estimated Total Weight To be Added (kips)	Percent Weight To be Added
1.	BWST 1T-60	860	4,340	500%
2.	BWST 2T-60	760	4,340	570%
3.	Auxiliary building wings			
	Unit 1	7,700	350	5%
	Unit 2	-do-	-do-	-do-
4.	Auxiliary building railroad bay between column A and AA			
	Between columns 4.55 and 5.1	3,800	80	2%
	Between columns 5.1 and 7.4	5,700	100	2%
5.	Main feedwater isolation valve chamber	650	6	1%
6.	Service water pump structure	4,770	200	4%
7.	Emergency diesel oil storage tanks*	3,770	486	13%

*The tanks are currently filled with water.

Question 22f

Identify all alternative solutions associated with the plant area fill settlement which would be foreclosed by continuation of any of the above activities.

Response

As noted in the above responses, nine ~~eight~~ Seismic Category I structural areas, as well as the yard piping/utilities, have been identified as safety-related structures or systems founded on plant area fill where additional construction work is necessary to complete the facility & where there is questionable fill. These structural areas include:

? ?

1,2) Electrical penetration areas (both Units 1 and 2) of the auxiliary building

3) Control tower of the auxiliary building

4) Railroad bay of the auxiliary building
limit

5) Service water pump structure

6) Diesel generator building

7) Borated water storage tanks

8) Emergency diesel fuel oil tank

9) Feedwater ISO Valve

With the exception of the borated water storage tanks, all structural work for the above items is complete. However, there is significant work remaining in the mechanical and electrical areas, including the installation of piping, electrical trays and cables, cabinets, other mechanical equipment, HVAC, etc. In the service water pump structure, the large service water pumps must still be installed.

A review of the alternative solutions which might be foreclosed by continued construction activities in these areas include the following.

1) Portions of the Auxiliary Building, Including the Electrical Penetration, Control Tower, and Railroad Bay Areas

Any required corrective measures for these several areas will likely be performed using underpinning or other repair methods installed from outside of the structure (i.e., sink an access shaft down from plant grade, and then tunnelling beneath the existing structural foundations slab). Because the added weight resulting from the remaining construction work to go is minimal (i.e., 5% or less, electrical and mechanical items, see Table 22e-1), there is no risk that continued construction activity would foreclose on this option.

An alternative solution to provide corrective repairs to these auxiliary building areas would be to initiate repairs from within the structure. Continued construction activity would add congestion in the repair areas, and make this alternative more difficult to implement. However, much of the congestion already exists. Also, if necessary, portions of the installed electrical and mechanical services could be later removed, albeit at a cost and schedule penalty.

In summary, continuing construction activities in these several areas of the auxiliary building does not foreclose any corrective actions.

2) Service Water Pump Structure

The north and east sides of the service water pump structure are accessible for underpinning from outside of the structure. The continued installation of electrical and mechanical items, including several large pumps, would add to the congestion inside the building and make repairs from within the structure less desirable. However, similar to the auxiliary building areas, it is possible to remove such items if necessary. Again, continued construction activity in this structure does not foreclose on any future repair methods.

3) Diesel Generator Building

This area is currently surcharged, and no construction activities are underway. No construction work in this area will be resumed unitl MCAR 24 is satisfactorily resolved.

4) Borated Water Storage Tanks

A preload program will be implemented in this area as described in Question 6 above. At least one and possibly both tank areas will be preloaded before the tanks are erected. Upon completion of the tanks, the water loads will represent a five-fold increase on soils loading.

If the taak areas require corrective measures, the installed tnaks will not preclude grouting or similar repair methods. If complete soils replacement is required, the tanks are accessible for removal, although at significant cost and schedule penalties.

5) Emergency Diesel Fuel Oil Tanks

Does not agree with Table 12-1

Similar to the above comments for the borated water storage tnaks, the emergency diesel fuel oil tnak foundation areas may be grouted, or, if soils replacement under the tanks is required, the tanks could be removed.

6) Yard Piping/Utilities (Later)

Based on the above considerations, there is no risk in allowing current construction activities in these areas to continue which might later foreclose on any anticipated alternative corrective measures.