SHR-1000-02-0

STATUS REPORT ON ADEQUACY OF CATEGORY I STRUCTURAL BACKFILL SOUTH TEXAS PROJECT ELECTRIC GENERATING STATION

TO

BROWN & ROOT, INC.

by

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ADEQUACY OF CATEGORY I STRUCTURAL BACKFILL
SOUTH TEXAS PROJECT ELECTRIC GENERATING STATION

1. INTRODUCTION

1.1 General

The Expert Committee presents this status report regarding our review of the adequacy of Categ ry I structural backfill. A comprehensive report is being prepared by the Committee, and it will be finalized upon completion of all activities by the Special Task Force established by Brown & Root, Inc. (B&R) and Houston Lighting & Power Company (HL&P) to perform an in-depth study of the structural backfill in response to Show Cause Order issued by the Nuclear Regulatory Commission (NRC).

1.2 Purpose and Scope

The purpose of this status report is as follows:

a) To reiterate the basic conclusions presented in the Interim Report issued by this committee on July 12, 1980.

b) To address each of the nine tasks originally assigned to the Committee, as identified in the response to Show Cause Order 2.

c) To present evaluations and conclusions for new tasks identified or completed after submittal of the Interim Report.

d) To identify work still in progress by the special B&R/HL&P Task Force, for which evaluations and conclusions will be made in the Committee's final report. Where appropriate, conclusions and/or recommendations are given in this status report.

The above items are addressed in the subsequent sections of this status report.

2. INTERIM REPORT

2.1 Basic Conclusions Reiterated

The Expert Committee's Interim Report (Reference 1) presented an assessment of the engineering adequacy and acceptability of the Category I structural backfill. That report was based upon the Committee's review of all pertinent aspects of the structural backfill design studies, specification criteria, construction procedures, inspection and testing procedures, results and documentation. It was also based upon review of the Standard Penetration Test (SPT) borings made at the site in two phases to evaluate the density of structural backfill.

As stated in the Interim Report, we conclude that with the type of compaction equipment used, the number of passes actually accomplished and the thickness of layers placed, a dense, homogeneous, compacted structural backfill resulted which is more than adequate for the intended use. Four small, isolated areas have been detected in which the Standard Penetration Resistance of the structural backfill, based on a commonly accepted correlation, would indicate a relative density less than construction quality control criteria. Our studies have demonstrated that, in spite of such isolated areas, the fill is sufficiently dense at all points tested to provide a high degree of safety against liquefaction during the postulated Safe Shutdown Earthquake (SSE). Since the test locations were selected in an unbiased manner, and their number is adequate to provide a representative sample of fill conditions, we conclude that the condition of the fill is adequate and satisfies the design requirements.

As described in the Interim Report, the June 1980 test fill demonstrated that the project construction procedures are capable of providing a uniformly dense structural backfill which meets the design and specification requirements.

The thin layer (4 to 10 inches) of backfill immediately below mat foundations probably has a relative density less than 80%. The detailed studies presented in the Interim Report show that even for an assumed density as low as 45%, the presence of such a layer will have no signifcant effect on the performance of the building as a result of the shaking produced by the postulated Safe Shutdown Earthquake (SSE).

2.2 Amplifying Committee's Review of Settlement Measurements

This section provides information additional to that presented in section 2.7 of the Interim Report, regarding the Committee's review of field settlement measurements. We have reviewed pertinent settlement measurements involving monitoring points surrounding and beneath the structures of both Units 1 and 2, to see if there has been any measurable compression of the structural backfill after it had been placed.

There are three basic types of monitoring points being measured at the site to determine settlement, as follows:

a) Borehole Heave Point (BHP)

b) Sonde Extensometer (Sondex)

c) Structural Bench Mark

A schematic drawing showing the installation concept of each type of monitoring point is presented as Figure 1.

In order to determine the amount of any compression of the structural backfill, a comparison is made between the settlement of a structural bench mark and the corresponding settlement of a nearby BHP or Sondex. If the structural bench mark moves downward more than the nearby BHP or Sondex during the same time inteval, compression of the fill is indicated.

The sets of monitoring points beneath and surrounding the Auxiliary Buildings at Units 1 and 2 provide particularly good data regarding compression of about 17 feet of structural backfill beneath the foundation mats. For the Unit 1 Auxiliary Building, a compression of about ¼-inch is indicated, which is approximately equal to the amount of compression calculated using the imposed building pressure and the soil modulus value E for very dense structural backfill material as determined from References 2 and 3. Thus, a very dense structural backfill is indicated.

The data from Unit 2 shows no measurable compression of the structural backfill beneath the Auxiliary Building, which is consistent with the lighter existing structural loading for that building. This lack of measurable settlement also reflects a high modulus value and a very dense structural backfill condition.

3. NINE COMMITTEE TASKS IDENTIFIED BY SHOW CAUSE ORDER RESPONSE

The principal purpose for this Expert Committee is to provide an independent review and assessment of the engineering adequacy and acceptabilty of the Category I structural backfill already in-place. As listed in the Show Cause Order response to item (2), nine tasks were originally assigned to the Committee. This section of the current report presents the nine tasks and addresses the status of each.

3.1 Task 1: Reviewing laboratory tests and design analyses performed on the backfill concerning liquefaction and settlement.

The Committee has reviewed the pertinent results of laboratory testing and design analysis performed on the backfill, concerning liquefaction and settlement. The data reviewed was in two 1975 soil reports by Woodward-Clyde Consultants (WCC): 1) Basic Soils Data report, Reference 2; and 2) report on Evaluation of Liquefaction Potential, Reference 3. The settlement behavior, as related to compression of structural backfill, is discussed in section 2.2 above. We are in agreement with the tests conducted and the results obtained, and confirm that the high quality structural backfill material, compacted as recommended, would provide safe support for Category I structures when analyzed for settlement and liquefaction potential under the SSE.

3.2 Task 2: Reviewing the recommendations and criteria for compaction of the backfill as documented in a series of geotechnical engineering reports issued by WCC.

References 2, 3 and 4, all issued by WCC, provide backup data, recommendations, and criteria for compaction of the backfill. Reference 4 is the report on Excavation and Backfill. The structural backfill recommendations given were based upon an assessment of strength, compressibility and liquefaction potential of available granular soils. We are in agreement with the recommendations and criteria given.

3.3 Task 3: Reviewing the construction specifications and procedures for compaction of the backfill, including the actual construction and quality control methods employed in the field.

The Committee has reviewed the construction specifications for structural backfill compaction (Reference 5) and has summarized the pertinent requirements in the Interim Report (section 2.2.b). The quality control procedure, as given by Reference 6, was also reviewed and found to prescribe adequate methods for achieving the Structural Backfill specification requirements; refer to section 2.2.c in the Interim Report.

The main requirements are for the structural backfill material to be placed in uniform layers with loose thicknesses not exceeding 18 inches for unrestricted areas, nor greater than 8 inches in restricted areas where hand-operated tampers are used. All Category I structural backfill is to be compacted to a minimum relative density of 80 percent with a running average of at least 84 percent (ASTM: D 2049-69). In both restricted and unrestricted areas, compaction efforts are to continue until passing test is obtained.

The 84 percent minimum criteria for average relative density was developed to assure that a series of marginally passing tests would not occur in one area, and should not be regarded as an indication of the overall average relative density which actually is about 95 percent.

We confirm that the structural backfill recommendations, which were given in WCC reports and judged to be adequate by this Committee, were successfully translated into project specifications and quality procedures. It should be noted that the relative density requirements as recommended by WCC (75% minimum, 80% average) were upgraded in the project specifications to 80% minimum, 84% average.

The Committee has reviewed the actual construction and quality control methods employed in the field. Based on our observations and findings, which are summarized in section 2.3 of the Interim Report, we conclude that the actual construction and quality control methods were adequate to achieve an acceptable, dense, homogeneous, compacted structural backfill.

3.4 Task 4: Reviewing the prescribed and implemented inspection and testing procedures for compaction control of the backfill placement, including the quality control methods and documentation implemented in the field.

We have reviewed the quality control inspection and testing procedures as given in References 5, 7 and 8, and judge them to be adequate for confirming that the structural backfill has been properly placed and compacted. Sections 2.4 and 2.5 of the Interim Report describe the testing procedures and results, which demonstrate that the specified backfill material was placed to achieve densities exceeding the design requirements.

Both inspection and testing were conducted by Pittsburgh Testing Laboratory (PTL). With respect to fill placements, the PTL representative checked for proper lift thickness before compaction was allowed in any area. Documentation consisted of a checklist and Earthwork Inspection Reports (EIR's). One or more inspectors were used each shift, depending upon the amount of work being accomplished, and each inspector prepared a checklist and an EIR for their shift. The EIR's describe the placement areas and other pertinent observations. The checklist includes noting the maximum lift thickness, i.e, that the 18-inch maximum loose lift thickness allowed by specification had not been exceeded; and checking conformance with pertinent specification requirements.

When viewed on a day to day basis, the checklists and EIR's provide a good record of backfill activities. Further information regarding EIR's is presented subsequently in this report.

3.5 Task 5: Reviewing and analyzing compaction quality control test results collected and documented covering the construction period from 1976 to 1980.

The Committee has reviewed and analyzed the compaction quality control test results covering 1976 to 1980. Appendix K to Reference 9 presents a summary of all relative density tests being studied. For ten generalized test areas, with five surrounding and beneath the buildings of each unit, the relative density results (number of tests, range and mean relative densities) are tabulated area by area at one-foot intervals of elevation. The relative density results were used in the statistical analyses presented later in this status report.

As described in section 2.5 of the Interim Report, the tabulated results demonstrate that the structural backfill was compacted to achieve consistently high density results at all elevations. The mean relative density values ranged from 93 to 102 percent, which exceed the specified requirements. The testing frequency was high and also exceeds specification requirements. The uniformity of structural backfill compaction was further demonstrated by the test fill results (see section 3.9 below).

3.6 Task 6: Reviewing the results of special investigations of the placed backfill to assess adherence to the design requirements, including the results of the test boring program performed during the Spring of 1980.

Special investigations have included the two-phase boring program conducted in 1980, and the measurement and mapping of compacted lift thicknesses in open excavations at the site. These special investigations were both addressed in the Interim Report.

The Committee has reviewed the two-phase boring program which was conducted at the site to evaluate the density of placed backfill and assess adherence to design requirements. Section 2.8 of the Interim Report describes the boring program and results.

The boring program results clearly demonstrate that very dense conditions were achieved by compaction of the structural backfill. Out of 238 Standard Penetration Tests (SPT) conducted in the twenty-one Phase I borings, only eight SPT results indicated densities less than the 80 percent minimum construction control criteria, based on the Gibbs and Holtz correlation. These eight tests identified four potential problem areas. Twenty-eight additional borings were then made as Phase II, to better define the horizontal and vertical extent of potential zones of density less than the construction quality control criteria.

Section 4 of the Interim Report presents an analysis of liquefaction potential based upon the SPT data taken from the four potential problem areas. The analysis determined that in all cases, except one, the SPT values showed the sand backfill to have a factor of safety greater than 1.5 against liquefaction. For the one SPT value which does not meet this criterion, the factor of safety is 1.4. The negligible pore pressures which might build up in isolated zones are not considered significant with respect to the adequacy and safety of the overall structural backfill.

The Committee has concluded that the fill is sufficiently dense at all points tested to provide a substantial degree of safety against liquefaction during the postulated SSE. Since the test locations were selected in an unbiased manner, and their number is judged by this Committee to be adequate to provide a representative sample of the fill conditions, it is concluded that the condition of all structural backfill, as placed, is adequate to meet the design requirements. We have concluded that the number of borings in the completed two-phase program is sufficient and that no further borings are required.

Measuring and mapping compacted lift thicknesses in open excavations at the site was discussed in section 2.3.b of the Interim Report. The minimum lift thickness identified by WCC geologists was about 4 inches and the maximum lift thickness identified was about 17 inches. Compacted lift thicknesses ranged generally from 10 to 16 inches, thus demonstrating compliance with the maximum 18-inch loose lift requirement.

3.7 Task 7: In the event low density pockets are delineated in the backfill, reviewing the efficacy of the methods proposed by B&R Engineering to treat those pockets to bring them into conformance with the design requirements.

Since the condition of the structural backfill, as placed, is judged to be adequate, there is no need for remedial action to be taken.

3.8 Task 8: Inspecting the on-site laboratory facilities and equipment used by the existing testing agency, and reviewing the laboratory test procedures.

Committee members have inspected the on-site laboratory facilities and equipment used by Pittsburgh Testing Laboratory, the existing testing agency, as noted in section 1.3 of the Interim Report. The procedures were also observed and they were judged to be satisfactory, and the workmanship was very good; refer to Interim Report section 2.4.b.

3.9 Task 9: Reviewing the procedures, observing field operations, and evaluating the results of the test fill program.

Committee members have observed the test fill during site visits, as noted in section 1.3 of the Interim Report. We have also reviewed the procedures and our evaluation of the results is presented in section 3 of the Interim Report.

From the results of the June 1980 test fill, the Committee has concluded that the project vibratory rollers are capable of compacting the specified lift thicknesses to a uniform density throughout the structural backfill.

4. NEW TASKS

4.1 Maximum Density by Wet Method

A new task assigned to the Committee by B&R, in response to questions by the NRC, is to evaluate the wet method of determining maximum density. The purpose of this evaluation is to determine whether or not the wet method should be used in determining maximum density, rather than the dry method which has been used for control testing up to this date. A secondary purpose is to describe the type of soils that can be expected to attain higher maximum density using the wet method in lieu of the dry method.

ASTM: D 2049-69, the reference testing standard, states as follows:

> "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 testing 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." (underlining added)

As further background information, the dry method test for maximum density is run by placing oven-dried backfill soil into a 0.1 cubic foot mold (6 inches I.D. by approximately 6 inches high), adding a surcharge weight of 57 pounds to the top of the sample, and then vibrating the sample on a vibratory table for 8 minutes. For the wet method, a similar procedure is repeated using saturated soil; however, two vibration periods are used: during and just after filling the mold, the saturated soil is vibrated for 6 minutes; then the 57-pound surcharge weight is added and the sample vibrated for an additional 8 minutes.

The NRC has requested clarification of the words "some soils" as stated in Note 2 of ASTM: D 2049-69; i.e., they want to know what type of soils reach a higher maximum density when using the wet method. This information is not documented in published literature, but the best information we can gather is that the wet method usually controls for well-graded sand and gravel samples that are slightly silty (fines content between 5 and 12 percent passing No. 200 sieve), while the dry method usually controls for clean sand such as used for backfill at the site. For clean soils, when the maximum density as determined by the dry method is 125 pounds per cubic foot (pcf) or less, the dry method nearly always controls. In the range of 125 pcf to 130 pcf the dry method controls part of the time and the wet method controls part of the time, with the difference between the two values generally falling within the range of normal testing variations; thus, the dry method maximum value would be valid. Soils with a maximum density above 130 pcf are usually controlled by the wet method.

Table 1 in this status report presents the results of "Comparative Tests of Dry vs. Wet Methods for Determining Maximum Density", as performed by the PTL project field laboratory and by Clarence Chan (see Interim Report). The Table 1 test results show that for some samples, the maximum density obtained by the wet method was about one or two percent higher than the densities obtained by the dry method. On the other hand, with reference to the comparison tests on samples taken from the same test fill location 8B2 and the comparison tests by Clarence Chan on sample 8A2, the maximum density obtained by the wet method was about one or two percent lower than obtained by the dry method.

In light of the above data, it is the judgment of this Committee that the maximum density values as determined by the dry method are valid. The following reasons are given to substantiate this conclusion:

a) With reference to Table 1, the maximum density as determined by the dry method falls within the range of plus or minus one or two percent from the maximum density obtained by the wet method. Thus, within this range of normal and expected testing variations, the dry method maximum is deemed valid.

b. Virtually all the structural backfill soils have had a maximum density less than 130 pcf, where the dry method value is judged to be valid as previously described.

c) The average values of maximum density used for quality compaction control have generally been less than 125 pcf, where the dry method normally controls.

d) With reference to section 2.6 of the Interim Report, we have concluded that the true minimum densities of backfill soils are about 3 or 4 pcf lower than determined by PTL; thus, the production tests are judged to be conservative because the reported relative density values are, in effect, too low. These lower minimum density values would offset those few instances where the wet method maximum value may actually be one or two percent higher than the dry method value.

e) The dry method test produces more consistent and repeatable results. The wet method, which is a more difficult and time-consuming test, often produces inconsistent results due to "muddy" sample conditions and related segregation.

We, therefore, conclude that the wet method is not as satisfactory as the dry method for the purposes of quality control testing of STP backfill materials and should not be used. Unless further questions are raised, this task is considered to be complete by the Committee.

4.2 Statistical Analysis

Dr. A. H-S. Ang, Professor of Civil Engineering, University of Illinois at Urbana-Champaign, has assisted the Committee in the review of the statistical analysis of structural backfill density tests. Dr. Ang is an expert in statistics and probability, with more tha 20 years experience in this field.

Dr. Ang has reviewed the statistical/probability studies of the relative density results as conducted by WCC; their results are presented in Reference 10. He is in agreement with the methods used by WCC, which have resulted in the statistical results included as Table 2 in this status report. As given in WCC's table, the probability of fill with a relative density less than the 80 percent minimum construction control criteria, ranged from 0.3% to 6.2% for the ten Category I structural backfill areas. An overall weighted average of 3.9% probability was obtained, and the 90% confidence limits of this probability were also determined.

Even of greater importance than the postulated amount of backfill with relative densities less than 80 percent are the very low statistical probabilities that portions of backfill may have relative densities less than 70 or 60 percent. WCC has calculated that an average of less than 0.1 percent of the backfill could have relative densities less than 70 percent, and the probability of relative densities less than 60 percent is virtually zero (10^{-6} or 10^{-7}).

In addition to reviewing WCC's statistical results, Dr. Ang has also conducted independent studies to examine the plausible range for the percentage of backfill with a relative density less than the 80 percent minimum construction control criteria. Using a method called the "AOQ method" (Average Outgoing Quality), the probable percentage of backfill with a relative density less than 80 percent is 2.4 percent. Using the SPT data from WCC's Phase I borings, a value of 2.8 percent is obtained for this same probability. These values have the same order of magnitude as those obtained from the more detailed studies by WCC, and are, therefore, in general agreement with those studies.

4.3 Significance of Statistical Percentage of Backfill with Relative Density Below 80%

The following table presents a summary of the statistical analysis results, as obtained by WCC and Dr. Ang:

Statistical Analysis Summary

	Probability* of Structural Backfill with Relative Density Less Than:		
	80%	70%	60%
AOQ Method (Ang)	2.4%	--	--
SPT Values (Ang)	2.8%	--	--
WCC Extensive Analysis	3.9%**	< 0.1%**	Virtually Zero

* Probability also represents fraction of fill postulated to have relative densities less than values shown.

** Average Values.

If the structural backfill has relative densities as indicated by the statistical analysis results shown above, we conclude that the risk of liquefaction is negligible. We have thus reached the same conclusion as in the studies presented in the Interim Report; that is, the fill is sufficiently dense to provide a high degree of safety against liquefaction during the postulated Safe Shutdown Earthquake (SSE).

It is further the judgment of the Committee that a minimum relative density of 70 percent would be sufficient to provide an ample margin of safety against liquefaction of the project backfill soil under the postulated SSE. Thus, if all the structural backfill had been compacted to actual relative densities between, say 70 and 80 percent, we conclude that the risk of liquefaction would still be virtually negligible. This example, where 100 percent of the backfill could have relative densities between 70 and 80 percent and would still be adequate, should be contrasted with the much better existing backfill conditions where statistically there is only 3.9% (average) below 80% and less than 0.1% average with relative densities below 70%. This comparison demonstrates the high degree of adequacy for the existing structural backfill.

Case studies given in the WCC Statistical Report (Reference 10) may serve to put into perspective the amount of tests sometimes allowed below specification requirements. Based on published data from several highway projects, the case studies show that up to 20 percent or more tests fell below the minimum compaction standards; this compaction was judged to be satisfactory for the particular projects. At the South Texas Project, recompaction and retests were accomplished for an entire placement area whenever the relative density test fell below 80 percent. Thus, zero percent of the tests were allowed to be less than the minimum requirement, with the statistical distribution curves indicating the possibility that an average of only 3.9% could be less than the 80 percent minimum criteria.

5. WORK IN PROGRESS

5.1 Comparison of SPT Results with Backfill Placements and Density Test Results

WCC is currently conducting, as a part of the Task Force effort, an analysis to compare the Standard Penetration Test (SPT) results (N-values) with backfill placements and density test results. The main objectives of this work are as follows:

a) To prepare comparative logs for the original Phase I borings, permitting a detailed evaluation to be made of the relationship between the SPT sample locations, N-values, lift locations, and field relative density test results from each lift placement penetrated by the borings. Conclusions are to be made from these comparative logs as to the representativeness of the SPT results.

b) To examine the four backfill areas in Unit 2 with N-values indicating relative density less than 80% (based on Gibbs and Holtz method). The purpose of this is to determine, if possible, why such zones were indicated.

NOTE: As stated in section 4 of the Interim Report, the identified zones of relative density less than 80% were still found to exhibit a high degree of safety against liquefaction during the postulated SSE.

We have reviewed the comparative logs being prepared, and note that the graphical presentations of N-values and relative density from density tests both confirm that backfill compaction achieved very dense conditions. The field density test results at the boring locations appear to be no different in distribution than that for the overall backfill. This will be further studied by additional statistical analyses.

We have reviewed additional data being gathered about the four areas with SPT values less than the construction quality control criteria. It should be noted that one of the four areas contained two small zones. Our examination of this data indicates that two of the five zones were very small, about 6 by 10 feet in size, and one lift thick. The other three zones are primarily confined to local spots adjacent to cut slopes or to the lower layers adjacent to subgrade; therefore, it is unlikely that similar areas would be found within the interior backfill mass. It is the judgment of this Committee that any such local zones, either within the backfill mass or adjacent to the subgrade or cut slopes, would not affect the overall quality and behavior of the densely compacted structural backfill.

5.2 Locations and Sequence of Category I Backfill Placements

In accordance with the basic Technical Reference Document (Reference 9), a comprehensive Engineering evaluation is being made of the construction conditions, placement sequence and density testing of the Category I structural backfill placed at the STP. Thus far the B&R/HL&P Task Force has completed a tabulation of density tests for all Category I buildings, which data have formed the basis for the statistical analysis described in this report. Field verification of lift thicknesses in accessible areas has also been completed (refer to section 2.3.b of the Interim Report).

Work in progress includes data retrieval and processing regarding the structural backfill, to develop cross sections and plan views to provide a visual representation of the spatial relationships of lift placements, sequence of placements and in-place density testing. We view this work as providing a basis for the following:

a) Evaluation for the retrievability of data,

b) further verification of lift thicknesses and evaluation of construction conditions,

c) verification that the specifications for testing were met, and

d) further assessment of the engineering adequacy of the structural backfill.

The plans and cross sections showing structural backfill placements are near completion for the Mechanical-Electrical Auxiliary Building for Unit 2 (MEAB #2). We have reviewed preliminary copies of these drawings. Similar drawings for MEAB #1 are scheduled for completion in a few weeks. Also underway is the drafting of all density test locations on plans and cross sections for all Category I structures at Units 1 and 2. We have reviewed preliminary copies of these drawings.

From the Earthwork Inspection Reports and Density Test Reports on file at the site, all necessary data to assess the adequacy of the structural backfill is retrievable. The detailed plans and cross sections showing backfill placement areas demonstrate that the sequence of backfill placements can be reconstructed from the quality control records.

The records show that virtually all placements conformed to the specification requirements. With respect to lift thicknesses, it should be noted that lifts had been numbered in sequence from the bottom up, and that elevations for the lifts were duly recorded. The drawings and records confirm that the lifts were placed and tested in accordance with the specifications. Possible exceptions to the specified requirements are addressed in section 5.3. The preliminary drawings showing the locations of density tests on plans and in cross sections, show a good distribution of tests throughout the backfill areas. The test locations are judged to provide a representative sampling of the backfill and the frequency of testing is adequate.

It is the judgment of the Committee, based upon engineering considerations, that it is not necessary to plot the backfill placements beyond those nearly completed for MEAB #1 and MEAB #2. We believe that the original purposes for doing this work have already been fulfilled, and further efforts in this regard would not add significantly to technical evaluations of backfill adequacy. The plotting of all density test locations on plans and cross sections, however, should be completed.

5.3 Review of Earthwork Inspection Reports (EIR's)

The Earthwork Inspection Reports have been reviewed and checked by the B&R/HL&P Task Force, to determine whether or not the reported activities are in accordance with applicable specifications and construction procedures.

For the EIR's reviewed to date, members of the Task Force team have recorded comments regarding construction and documentation. These comments, generally noting minor inconsistencies, can be generalized into fourteen groups: 1) compaction - not completed, not documented; 2) material type - wrong type in structural area - 1 case only; 3) documentation problems regarding coordinates and elevations (data incomplete, doesn't match, etc.); 4) identification of test numbers (test numbers repeated); 5) lifts completed, but no mention made of when placed; 6) potential thick lift; 7) too low apparent test frequency; 8) placements without tests; 9) failing tests without documented retests; 10) incomplete document descriptions, non-technical (dates wrong, missing dates, etc.); 11) test locations outside lift area coordinates; 12) mixed backfill sources; 13) subgrade pumping; and 14) static rolling.

The first eleven of these comments are judged to be minor documentation inconsistencies reflected in the EIR's. It should be noted that the Density Test Reports (DTR's) were also reviewed, and that tests which were taken in backfill placements were not always so noted on the EIR's. With respect to comment 2), further review of construction data confirmed that the wrong type of soil could not have been placed in the structural area; the coordinate was incorrectly given on the EIR. Referring to comment 6), later EIR's were found where such thick lifts were cut down to specification requirements.

We conclude, from a technical standpoint, that the first eleven comments need not be addressed further. Pursuing them further would not add significantly to engineering evaluations of backfill adequacy.

Comment 12) describes a construction condition of insignificant consequence, since the backfill soils from all the sources were essentially identical in soil type and density properties. No further work needs to be done for this comment, from a technical standpoint.

Comments 13) and 14) should be further pursued. Each case should be studied and an engineering evaluation made. Current indications are that the static rolled areas involved the initial lift of backfill at certain locations in order to prevent or minimize subgrade pumping. This procedure is a common industry practice. Compaction of the next lift continues until tests in the initial lift meet density requirements. Passing test results in some of the static rolled lifts have already been identified, but all such areas should be evaluated.

In summary, minor inconsistencies have been noted in the documentation, particularly when making comparisons between different reporting periods; e.g., EIR's prepared on different days may give slightly different coordinates for the same placement area. (It is also recognized that coordinates are generally estimated, which is considered to be an acceptable industry practice.) These minor inconsistencies are not of any technical significance, and the data compilation (refer to previous section 5.2) shows that the backfilling operations were, in fact, systematic when interpreted with engineering judgment.

Our overall conclusion is that the records are basically good and confirm that backfilling was accomplished in a workmanlike manner as per specification requirements and applicable procedures.

6. SUMMARY OF CONCLUSIONS

6.1 Backfill Adequacy

Based upon work to date, we see no change to the conclusions given in the Interim Report. In fact, the detailed evaluations by the B&R/HL&P Task Force have strengthened our conclusion that the condition of the Category I structural backfill, as placed, is better than design requirements. In spite of isolated areas of potential density below 80 percent, as was identified by the two-phase SPT boring program, and hypothesized by statistical analyses, we conclude that the backfill is sufficiently dense to provide a high degree of safety against liquefaction during the postulated Safe Shutdown Earthquake (SSE).

6.2 Recommended Tasks for Completion

We recommend that the following tasks be completed:

a) Preparation of drawings showing backfill placements on plans and cross sections for MEAB #1 and MEAB #2.

b) Preparation of drawings showing all density test locations on plans and cross sections for Units 1 and 2.

c) The graphs depicting the comparison of SPT results with backfill placements and density test results.

d) Engineering evaluation of EIR comments 13) and 14) regarding subgrade pumping and static rolling, respectively.

The final comprehensive report by this Committee, which will further address the above tasks, will be submitted upon completion of these activities by the B&R/HL&P Task Force.

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H. Bolton Seed
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Stanley D. Wilson
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Chairman

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10. Woodward-Clyde Consultants, "Statistical Analysis of Relative Density Data for South Texas Project, Units 1 and 2," Preliminary Report WCR-9027-01-0, transmitted by WCC letter ST-WC-BR-5768, October 17, 1980.

Table 1

COMPARATIVE TESTS OF DRY VS. WET METHODS FOR DETERMINING MAXIMUM DENSITY SOUTH TEXAS PROJECT

		Vibration Method - Modified ASTM: D 2049-69			
		Wet Method		Dry Method	
Sample Designation	Rheostat Setting	Max. Dens. pcf	% of Dry Max. Dens.	Max. Dens. pcf	% of Dry Max. Dens.
A. COMPARISON FOR SAMPLES S-3101, S-3102 & S-3103					
S-3101	40	125.8	100.9	124.7	100.0
S-3101	60	125.8	100.9	--	--
S-3102	40	126.7	102.0	124.2	100.0
S-3102	60	127.1	102.3	--	--
S-3102	60	126.6	101.9	--	--
S-3103	40	127.3	100.8	126.3	100.0
S-3103	60	125.9	99.7	--	--
B. COMPARISON USING SAMPLES TAKEN FROM SAME TEST FILL LOCATION, 8B2 (8-PASS LANE, ROW B, LIFT 2)					
S-3103	40	127.3	98.9	126.3	98.1
S-3103	60	125.9	97.8	--	--
8B2 (TF)	40	--	--	128.7	100.0
8B2 (CHAN)	75/100	--	--	128.6/128.7	99.9/100.0
C. COMPARISON USING SAMPLE FROM TEST FILL LOCATION 8A2 (8-PASS LANE, ROW A, LIFT 2)					
8A2 (CHAN)	75/100	126.2	98.5	127.4/128.1	99.5/100.0

D. SAMPLE LOCATIONS

S-3101	Stockpile 6+50/A+990
S-3102	FHB #2 10+00/D+80 (Elev. 27)
S-3103	Test Fill Location 8B2 (8-pass lane, row B, lift #2)
8B2 (TF)	Test Fill sample taken during density testing at location 8B2
8B2 (CHAN)	Test Fill sample sent to Clarence K. Chan from location 8B2
8A2 (CHAN)	Test Fill sample sent to Clarence K. Chan from location 8A2

NOTES:

1. All tests by Pittsburgh Testing Laboratory (PTL) except tests by Chan.
2. Considering both wet and dry methods, highest value is underlined.

Table 1. STATISTICAL SUMMARY OF FINAL RELATIVE DENSITY DATA

Data Set	Unit	Bldg	Sample Size	Sample Mean	Sample St.d.	Sample Skew	Shift that fits skew	Mean of log of (sample-shift)	St.d. of log of (sample-shift)	Goodness-of-fit Lognormal model K-S	Goodness-of-fit Lognormal model Chi^2	P[R<80]	90% Confidence Limits Lower	90% Confidence Limits Upper
1	2	MEAB	328	92.5	10.19	1.218	66.17	3.201	.3734	.2195	.0011	.062	.047	.079
2	2	FH	497	94.0	9.82	.822	57.28	3.570	.2625	.4093	.1514	.045	.035	.056
3	2	DG	93	95.1	10.22	.416	20.80	4.299	.1369	.4677	.6322	.056	.029	.087
4	1	MEAB	436	94.9	10.17	.557	41.33	3.964	.1881	.29[illegible]	.3713	[illegible]	[illegible]	.064
5	1	PA	14	102.8	17.26	1.236	60.34	3.670	.3929	.9946	.9952	.038	.003	[illegible]
6	2	CA	503	95.6	10.13	.519	36.41	4.066	.1699	.0718	.0091	.043	.033	.055
7	1	DG	93	96.2	10.34	.679	49.79	3.813	.2202	.8055	.6063	.033	.014	.057
8	1	FH	521	95.2	8.86	.680	55.41	3.659	.2201	.7779	.7287	.019	.013	.025
9	1	CA	355	95.9	8.96	.705	57.13	3.631	.2281	.9527	.6221	.014	.010	.021
10	2	PA	13	99.8	9.40	.666	56.82	3.738	.2160	.9489	.8603	.003	.000	.021
Total*			2853									.039	.035	.043

*Each data set weighted by number of data points

NOTE:
Table reproduced from WCC preliminary report "Statistical Analysis of Relative Density Data for South Texas Project Units 1 and 2" WCR-9027-01-0

LEGEND:
MEAB - Mechanical-Electrical Auxiliary Building.
FH - Fuel Handling Building
DG - Diesel Generator Building
PA - Containment Building "Pedestal"
CA - Containment Building

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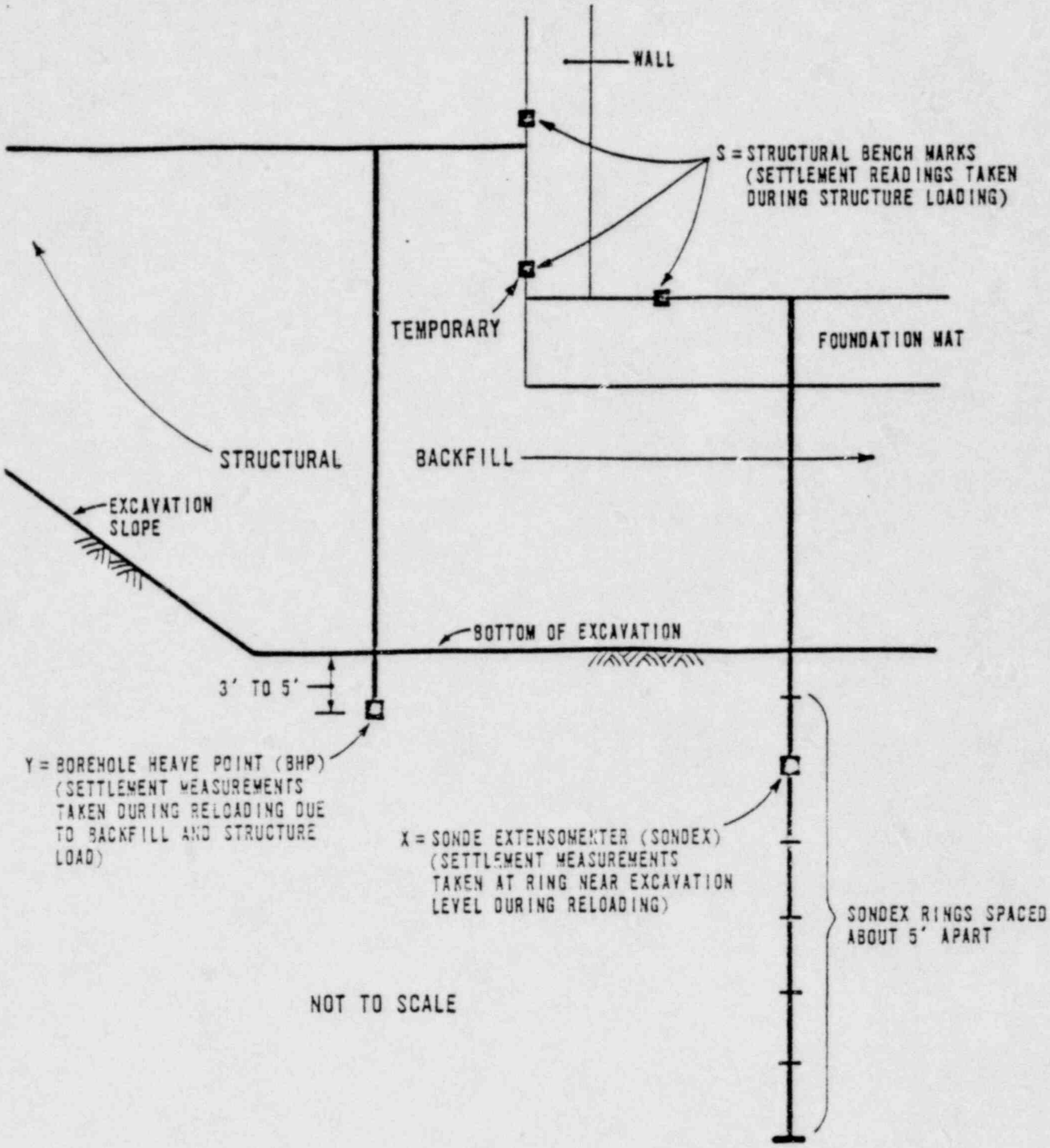
TABLE 2

PRELIMINARY DRAFT COPY



NOTE:

APPROXIMATE AMOUNT OF BACKFILL COMPRESSION DETERMINED FROM READINGS S MINUS Y AND S MINUS X; HOWEVER, THESE DIFFERENCES INCLUDE SOME COMPRESSION OR SETTLEMENT OF NATURAL SOIL DOWN TO POINT Y OR X.

SOUTH TEXAS PROJECT

SCHEMATIC DRAWING OF SETTLEMENT MONITORING POINTS

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FIG. 1

Woodward-Clyde Consultants

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504-525-1154

C. B. PETTERSSON

B&R

30 Se tember 1980
ST-W(BR-5729
SFN: D-0540/P-0087

Brown & Root, Inc.
P.O. Box 3
Houston, Texas 77001

Attention: Mr. J.L. Hawks
Engineering Project Manager

Subject: Standard Penetration Test Evaluation and Validity Report
South Texas Project Electric Generating Station
WCC Foreign Document No. WCR-9026-1-0

Gentlemen:

Woodward-Clyde Consultants (WCC) has performed two soil boring programs to study the structural backfill at the South Texas Project. The borings were made to determine the standard penetration resistance N-value and relative density of the structural backfill. The data from the two boring programs has been previously reported (Ref. 1, 2, and 3). This letter presents a validation of standard penetration tests made during the two boring programs and an evaluation of the N-value data collected. It is the final report on the structural backfill evaluation for Units 1 and 2. An amendment to the "Excavation and Backfill Report" (Ref. 25) will be made subsequently.

I. Initial Boring Program Validation

Woodward-Clyde Consultants made twenty-one (21) borings in the structural backfill during the period 28 January 1980 to 8 February 1980. These borings were made at the request of Brown & Root (B&R) to determine the standard penetration resistance N-value of the structural ba fill. The selection of the boring locations for the initial 15 borings was made jointly by WCC and B&R in an unbiased manner to obtain general coverage data of the structural backfill at the STP site. Pertinent information from the 15 borings is tabulated in Table 1 and the location of the borings is shown in Reference 1. Six additional borings were made as part of the initial program, to provide supplementary data from the vicinity of the initial 15 borings. Information for these 6 borings is also tabulated in Table 1 and their location is also shown in Reference 1.

During the standard penetration test boring operation for the first or initial program two nonconformances were noted, at time of drilling, and were thought to have an effect on the standard penetration N-value data. They were:

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(1) Drop hammer plus lifting chains for the Custom Auger Drilling rig weighed 148.9 lbs. rather than 140 lbs. as specified in ASTM D-1586.

(2) Drive shoe for the Custom Auger Drilling rig's split-barrel sampler measured 1.47 in. I.D. and 1.97 in. O.D. rather than 1.375 in. I.D. and 2.0 in. O.D. as specified in ASTM D-1586. (The measurements determined in the field. Subsequent office measurements are presented in Table 2).

These nonconformances were described and resolved (Ref. 4 and 5). The evaluation of the Custom Auger hammer assembly and the recorded weight showed that the lifting chains should not have been weighed as part of the drop hammer weight assembly. The hammer, which supplies the driving force, was weighed and was determined to weigh 139.8 lbs. (The initial weight report from the Pittsburgh Testing Laboratory STP field laboratory stated the weight as 138.9 lbs. That report contained a transposition of digits. A later corrected report stated the weight as 139.8 lbs.) The reported hammer weight minus lifting chains was 139.8 lbs. and was determined to be within acceptable limits.

The dimensions of the Custom Auger split-barrel drive shoe are shown in Table 2. At the time of drilling, and subsequently, it was decided that the diameter differences did not have a significant effect on the measured standard penetration resistance N-value data.

During the initial drilling operation three minor equipment differences were also noted. There were as follows:

(1) The weight of the hammer part used by the Younger Drilling Company (Younger) drill rig weighed 142 lbs.

(2) A Terzaghi drive shoe was used by Younger and it had a slightly rounded cutting edge which is typical of that drive shoe.

(3) The bore diameter of the split barrels used by Younger and Custom Auger was slightly larger than the throat diameter of the drive shoe. It was approximately 1-1/2 in. I.D.

These differences were evaluated at the time of the initial borings. The results are as follows:

(a) It was decided that the 2 lb. difference (1.4% of the total weight) did not have any significant effect on the standard penetration test N-value data;

(b) The Terzaghi drive shoe might have produced a minor influence on the N-value data but it was judged to be small and insignificant; and

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(c) The slightly larger inside bore diameter of the split-barrel might actually decrease the N-value (a conservative measure) but it was judged to be of little or no significance on the N-value data.

During and after the preparation of the initial boring program report (Ref. 1) further reviews and evaluations were made regarding the two nonconformances and the three equipment differences. The two nonconformances and three equipment differences fall into two categories; hammer weight and split-spoon sampler dimensions. These two categories are discussed subsequently.

The drop hammer provides the force used to move or drive the sampler into the soil. Geotechnical engineers believe that considerable variation in the drilling equipment and drilling techniques can be tolerated without affecting the usefulness or accuracy of the standard penetration N-value data. The variation from the ASTM requirement hammer weight for the initial STP boring program are minus 0.2 lbs. (0.15%) for the Custom Auger rig and plus 2 lbs. (1.4%) for the Younger rig. Thus, the Custom Auger N-value data may be slightly conservative (N-values slightly less than a "true" value), and the Younger N-value data may be slightly nonconservative (N-values slightly more than a "true" value). Various investigations and literature references (Refs. 8, 9, 10 and 11) show that there are several variables, not limited to the force used to drive the sampler, that influence the determination of standard penetration N-values. After reviewing these references, it is concluded that the force on the drill rod and sampler can vary as much as ten (10) to twenty (20) percent and still be well within the normal limits used and accepted for standard penetration test. If the relationship between force and N-value is assumed to be a linear relationship, then a variation of plus or minus 1 blow per foot for an N value of 10 is quite acceptable and well within the normal acceptable test limits. It is concluded that the weight differences of the Custom Auger and Younger hammers is not a significant factor to the N-value data obtained in the initial boring program.

The split-barrel sampler dimensional concerns noted during the initial STP boring program are briefly resummarized below:

(1) Split-barrel I.D. is slightly larger than throat diameter of drive shoe (Custom Auger and Younger).

(2) The Younger drive shoe (a Terzaghi shoe) had a slightly rounded edge.

(3) Custom Auger drive shoe measured 1.45 in. I.D. at leading edge, and 1.43 in. I.D. at throat.

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The standard penetration test sampler regularly manufactured in the United States today has a split-barrel with an enlarged inside diameter to hold sampler liners and most drillers use these samplers without liners (Ref. 11). Also, samplers with the ASTM required diameter (1-3/8 in.) are only made on a special order basis. The use of a sampler with an enlarged split barrel inside diameter produces a reduction in N-value, and the percentage or amount of reduction increases with decreasing (lower) N-value in any type of soil (Ref. 11). This reduction is due to a relation of the sampler side friction as it enters the split-barrel. The reduction in "real" N-value will vary from 10% to 50% depending on the density of the soil mass being sampled (Ref. 11). It is therefore concluded that the "real" N-value of the soil mass will be 1 to 5 blows per foot more than the N-value observed and recorded for even the most loose soil at the STP site.

The bore diameter dimension differences were examined and reported by WCC (Ref. 14, 15, 16). The total force, summation of end-cone resistance, outside sampler barrel resistance, and shoe cylinder bore and inside spoon barrel resistance, necessary to move the split-barrel sampler in sand with different driving shoe configurations at different depths is shown in Table 3A and 3B. The calculation analysis for these comparisons was made to determine the force to drive the spoon when the spoon is embedded 12 inches in the soil medium at the various depths evaluated. This embedment depth was selected because it is the mid point of the N-value test.

It can be seen from Table 3A and 3B that there is a small driving force difference between the shoes used at the STP site. The soil medium parameters for the analysis shown in Table 3A were selected for a dense soil medium with a relative density of approximately 85% to 95%. The parameters include an angle of internal friction of 43 degrees. The soil mediums used for the analysis shown in Table 3B were selected for a loose soil medium with a relative density of approximately 55% to 65%. The analyses show that it takes the most force to drive the ASTM shoe and spoon, and less force to drive the worn shoe and spoon and Terzaghi shoe and spoon. The variation is between about 3% and 6%, and this is considered well within the normal or typical tolerance allowed for the variation in hammer force used and accepted in standard penetration testing. As an example there is only 4.1% difference in the required driving force between the ASTM shoe and the worn shoe at a 60 ft depth. To restate this comparison, a sand soil with 10 blows per ft N-value at sixty ft, as determined by the ASTM shoe, would have about 9.5 blows per ft N-value if a worn shoe were used to perform the blow count N-value test. Differences of 1 blow per foot in the low N-value range (5 to 15 blows per ft) are considered within a normal range when performing repeatability tests, and differences of ± 2 to ± 5 blows per foot are considered acceptable when performing repeatability tests in more dense or stiff soils.

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During the structural backfill evaluation several interesting items were learned regarding the use of different drive shoe configurations. One item was that there are four types of drive shoes used in making the so called standard penetration test in the United States. The most popular shoe, according to sales personnel of Acker Drill Company and Mobile Drill Company, is the Terzaghi shoe. It accounts for about 90% of the shoes made and sold by those two companies. The next most popular shoe is the ASTM shoe followed by the Raymond short end taper shoe, and the Raymond long end taper shoe. The Raymond short taper shoe is dimensionally similar to the Terzaghi shoe except it has two recessed flat wrench slots near the cutting end. The Raymond long taper shoe is dimensionally similar to the ASTM shoe except it also has two recessed flat slots to accomodate a wrench. The flat wrench slots on the Raymond shoe probably cause a slight but insignificant energy increase in N-values when compared with ASTM shoe N-values. Also, it is interesting to note that Messers. Gibbs and Holtz used a Raymond drive shoe when performing the research for their classic paper (Ref. 17). The relationships in this research paper were used by WCC to determine the relationship between N-value and relative density (Ref. 18).

It is interesting to note that much research and most of the standard penetration tests made in the United States, currently and apparently for some time, are not made with shoes or split-barrels of the ASTM D-1586 dimension or configuration. (It is thought by many geotechnical engineers that ASTM D-1586 has not been revised to the present state of practice).

As is known in the geotechnical engineering profession and as has been discussed herein there are several variables inherent with performing standard penetration testing that permits variations in the drilling technique and drilling and sampling equipment without significantly effecting the measured N-value. The standard penetration test is not purely an empirical in-situ test. It includes many theoretical and practical insights into the mechanics of testing and sampling with quantitative and qualifible evaluations regarding forces, energy and time involved in standard penetration testing and therefore N-values (Ref. 10). The apparent fact that each blow loads the sampled soil in cyclic undrained shear levels practical support to the use of N-values as an index text for liquefaction potential.

Based on our evaluation of the drilling and sampling performed during the initial boring program, and as reported herein, we believe that all of the N-value data collected is a valid and conservative representation of the structural backfill condition.

II. Second Boring Program Validation

Woodward-Clyde Consultants made twenty-eight borings in the structural backfill to provide additional N-value data in areas and zones where low N-value data were obtained during the initial test boring program. These borings were made during the period 24 March 1980 to 11 April 1980. The boring locations and sampling intervals were specified by WCC's Project Manager with concurrent approval from B&R Engineering. Drilling was performed in accordance with WCC procedure WCC-6000-K (Ref. 7). Procedure WCC-6000-K was developed utilizing sections and input derived from ASTM Standard D-1586 (Ref. 6). Logging of borings was performed by WCC procedure WCC-6000-K-1 (Ref. 19). The drilling was performed by Ables Drilling Company under the direction of the WCC Site Engineer working in conjunction with B&R's Field Engineer.

The work was performed in accordance with the WCC procedure WCC-6000-K-1 (Ref. 17) and WCC procedure WCC-6000-K (Ref. 7). New ASTM driving shoes were used in this drilling program. There were several split-barrel sampler manufacturer's deviations reported which were judged to have no effect on the N-value data collection. However, nonconformance reports were prepared as a means of control and documentation in accordance with project procedures. The nonconformances were:

(1) The diameter of check valve hole in split-barrel sampler was smaller than the 7/8" as specified by ASTM D-1586. The measured diameter was approximately 1/2".

(2) The inside diameter of the split-barrel sampler measured approximately 1 1/2" instead of the 1 3/8" as specified by ASTM D-1586.

(3) The split-barrel sampler head attachment tapers outward to allow fit of the split-barrel sampler to N-type drill rod.

The nonconformances were evaluated and resolved (Ref. 13). The evaluation of the nonconformances is summarized below.

(a) The size of the check valve hole in the split-barrel sampler did not restrict the outward flow of water as the soil moved into the split-barrel. Therefore, it did not effect or cause erroneous or high N-value data.

(b) The larger inside diameter of the split-barrel sampler does not cause increased N-values. In fact, a sampler with an enlarged diameter produces significant reductions in N-value data (Ref. 11). Thus lower N-values are measured than are actual (real) for the structural backfill tested. This is also discussed in the preceeding section and considered in the penetration resistance analysis.

(c) The outward taper at the split-barrel sampler head does not effect the N-value data. It does not penetrate the sand medium because it is in bore hole above the bottom of the hole at which point the sampling started.

The validity of the N-value data obtained during the second boring program is established and the data is correct and acceptable for use in subsequent engineering evaluations.

III. Evaluation of N-value Data

The N-value data collected during the two boring programs have been reported (Refs. 1, 2 and 3). As part of the N-value data reporting relative density values were calculated utilizing the Gibbs and Holtz relationship (Ref. 15 and 16). The results of the test boring N-value data and subsequent studies identified 4 areas of small extent in the structural backfill at Unit 2 with a density less than the construction quality control criteria, i.e., 80% relative density. The Unit 1 area N-value data indicates that the structural backfill equals or exceeds the construction quality control criteria of 80% relative density. The extent of the areas and zones with a density less than the construction quality control criteria in Unit 2 are briefly described below.

The first field investigation indicated four areas within the Unit 2 structural backfill as potentially having a density less than the construction quality control criteria. The second field investigation confirmed that two areas potentially had densities less than the construction quality control criteria. However, the other two areas were found to be in conformance with the construction quality control criteria. Area 1 (WCBV-204, west of Unit 2 tendon gallery access shaft) there were several, small, isolated zones which have a density less than criteria. In Areas 2 and 3 there is only one zone in each area with a low density. Area 4 has two zones with a density less than the criteria. Area 2 is west of the Unit 2 reactor containment building and boring WCBV-205 is within the area. Area 3 is east of the Unit 2 ME Aux. building and borings WCBV-208 and WCBV-209 are within the area. Area 4 is west of Unit 2 Fuel Handling Building and boring WCBV-203 is within the area. More detail regarding the locations is contained in reference 2. The areal extent in the four areas is described below.

Dr. H. Bolton Seed, University of California, Berkeley, California was retained to provide consulting services regarding liquefaction potential and N-value data during the evaluations made for the structural backfill. The presentation of N-value data and its relationships to liquefaction potential for the STP site are presented in Dr. Seed's letter of June 16, 1980 (Ref. 21), his technical paper (Ref. 20), his input to the Expert Committee's Interim Report dated July 12, 1980 and transmitted July 23, 1980 (Ref. 21) and a clarification of evaluations (Ref. 22). The evaluations by Dr. Seed (Refs. 21, 22, 23 and 24) are summarized as follows and presented on Table 5.

Dr. Seed has stated that the liquefaction potential of a sand deposit may be evaluated based on a correlation of the stresses developed in a deposit with the Standard Penetration Resistance of the sand (N-value). The basis for this procedure is a series of detailed studies of sites which are known to have liquefied or not liquefied in earthquakes in Japan, Guatemala, Argentina, China, Venezuela, etc. as well as the United States. The correlations established and the procedure for making evaluations have been described by Seed (Ref. 22), but it should be noted that the correlations presented at that time have since been confirmed by an abundance of data from more recent earthquakes.

Following this approach, analyses have been made to determine the N-values which would have to be developed in the sand backfill at the South Texas Project site to provide a factor of safety of 1.5 against liquefaction if the water table was at the ground surface and the site were subjected to 0.1g from a Magnitude 6 earthquake. Details of the computations are presented in Reference 21.

For the above conditions, and with the condition that borings were made when the water table depth was about 30 ft below the ground surface and the final plant operation condition will have a water level at or near the ground surface, the results shown on Table 5 are obtained.

Area 1 (WCBV-204) - In area 1 the elevations, locations and relative densities of four small zones with standard penetration tests showing a density less than criteria are tabulated on Table 4. The distribution with depth and location of standard penetration tests with a relative density less than 80% is shown in reference 2.

Within Area 1 (WCBV-204) the zones with a density less than criteria fall into four groups based on depth and boring location. The low densities in the first of the groups are judged to result from low compactive effort adjacent to the near vertical excavation slopes. These zones as encountered in borings WCBV-204-V15, 204A, and 204-V5 are considered to be quite limited in areal extent. In borings 204A and 204-V5 a low density was measured at only one elevation in each boring therefore the vertical extent of the zone with a density less than criteria is assumed to be one lift thick. At boring 204-V15 a low density was measured at two elevations in two successive standard penetration tests. The vertical extent at boring 204-V15 is therefore probably limited to two lift thicknesses.

The second of the group of borings with a density less than criteria in Area 1 is judged to result from the placement of a 2 1/2 ft thick lift above El -44 in an area south of the Tendon Gallery access shaft. Densities within the 2 1/2 ft thick lift less than construction quality control criteria were measured at borings 204B and 204-V4. Placement and compaction of structural backfill in this zone is described in detail in B&R's meeting minutes of 12 March 1980 (Ref. 18).

The third group of borings in Area 1, borings 204-V9, 204-V2, 204-V10, 204-V11 and 204-V13, with a density less than construction quality control criteria are grouped between essentially El-38.8 and -41.8. We believe the zone of low density is limited to parts of two lift thicknesses.

The fourth of the four groups of borings in Area 1 with a density less than the construction quality control criteria was encountered in boring 204-V6. At boring 204-V6, in addition to one low density betwen El -38.8 and -41.8, two additional low densities were measured at elevations about 5 ft apart; obviously in two different lifts.

The results presented in Table 5 were compared with all the penetration resistance values, in Table 4, of the sand which are believed to represent relative densities less than 80%, based on the Gibbs and Holtz correlation. (Ref. 17 and 18). It may be seen that in all cases, except one, the sand is found to have a factor of safety greater than 1.5 against liquefaction and, for the one test which does not meet this criterion, the factor of safety is 1.4. This isolated zone where the factor of safety is found to be 1.4 would build up only negligible pore pressures during the SSE and is not considered significant in assessing the engineering adequacy of the overall fill.

Area 2 (WCBV-205) - Within Area 2 the vertical extent of the structural backfill zone with a density less than construction quality control criteria, in our opinion, confined to one compacted layer, approximately 16 in.thick, below about El -7.8 ft. Results of these standard penetration tests with a density less than construction quality control criteria of 80% relative density are listed on Table 4.

Area 3 (WCBV-208 & 209) - Within Area 3 there are two different zones with a density less than construction quality control criteria; one at the 208 series of borings to the north and one at the 209 series of borings to the south. In each instance the area with a density less than construction quality control was defined in the subsequent field investigation. As in Area 2 the width of each area is believed to be 6 ft; one roller drum width. At each location the vertical extent of the zone is believed to be one commpacted layer thick, approximately 16 in., below about El -0.5 at boring 208 and below about El -1.6 ft at boring 209. The results of the standard penetration testing are listed on Table 4.

Area 4 (WCBV-203) - Within Area 4 there are 2 zones with a density less than construction quality control criteria; one zone at about El -17 and the other at about El -34. At about El -17 the width of the zone with a density less than construction quality control criteria is believed to be 6 ft., 3 ft. on either side of boring 203. We believe the length extends from midway between borings 203-V1 and 203 to midway between boring 203 and 203-V2. The vertical extent of the zone is believed to be one compacted lift thick (approximately 16 in.). The zone is probably 6 ft. wide. Its northern extent is midway between boring 203-V1 and 203. The southern limit of the zone is as defined by a construction slope. Table 4 also shows the N-value related to that zone which have a relative density less than 80% the construction quality control criteria.

On the basis of the Dr. Seed's analyses (Ref. 21, 22, 23 and 24) it is concluded that the isolated locations where the relative density may be less than the value of 80% construction quality criteria originally specified, the fill is sufficiently dense at all points tested to provide a substantial degree of safety against liquefaction during the postulated SSE. Since the test locations were selected in an unbiased manner and their number is adequate to provide a representative sample of the fill condition, it can therefore be concluded that the condition of the fill as placed is adequate and that no further study or evaluation of the N-value data from the structural backfill is necessary. It is also concluded, from a design engineering point, that no remedial action is necessary.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

Charles S. Hedges

Charles S. Hedges
Project Manager

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Woodward-Clyde Consultants

Attachment to ST-WC-BR-5729

LIST OF REFERENCES

1. WCC letter May 28, 1980, ST-WC-BR-5672, Relative Density of Structural Backfill, WCC Foreign Document No. WCR-6000-09-1.

2. WCC letter May 28, 1980, ST-WC-BR-5671, Relative Density of Structural Backfill, WCC Foreign Document No. WCR-6000-11-1.

3. WCC letter March 5, 1980, ST-WC-BR-5595, relating to relative density of structural backfill.

4. WCC letter February 26, 1980, ST-WC-BR-5586, regarding nonconformance and corrective action report for drive shoe of split-barrel sampler not meeting ASTM D-1586.

5. WCC letter February 27, 1980, ST-WC-BR-5587, regarding impact of nonconformances on structural backfill study field investigation results.

6. ASTM Standard: D-1586-67(74), "Penetration Test and Split Tube Sampling of Soils."

7. WCC Procedure: Procedure WCC-6000-K, "Drilling Procedure for Study of Relative Density of Structural Backfill, South Texas Project, Revision 1."

8. Fletcher, G.A., "Standard penetration test: Its uses and abuses, "Journal of Soil Mechanics and Foundation Division", American Society of Civil Engineers, SM 4, Paper 4395, pages 67-75, July 1965.

9. de Mello, V.F.B., "The Standard Penetration Test", Proceedings, Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, San Juan, Puerto Rico, vol. 1, pages 1-86, 1971.

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11. Schmertmann, J.H., "Statics of SPT", Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, GT 5, Paper 14573, pages 655-670, May 1979.

12. WCC letter dated April 1, 1980, ST-WC-BR-5617, regarding nonconformance reports.

13. WCC letter dated April 18, 1980, ST-WC-BR-5643, regarding disposition of nonconformance reports.

14. WCC letter August 6, 1980, ST-WC-BR-5723, regarding analysis of soil test boring split-barrel shoe configurations.

15. WCC calculation package, calculation no. 9026-6, WCC letter September 30, 1980, ST-WC-BR-5755.

16. WCC calculation package, calculation no. 9026-6A, WCC letter September 30, 1980, ST-WC-BR-5756.

17. Gibbs, H.J. and Holtz, W.G., "Research on Determining the Density of Sand by Spoon Penetration Testing", Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

18. U.S. Atomic Energy Commission (NRC) Directorate of Regulatory Standards, Washington, D.C., "Classification, Engineering Properties and Field Exploration of Soils, Intact Rock and Insitu Rock Masses", May 1974.

19. WCC Procedure: Procedure WCC-6000-K-1, "Appendix B-1, Instruction for Logging Borings, Construction Services Work Phase (formerly design work phase), Units 1 and 2", Revision No. 2, August 1, 1975.

20. Brown & Root letter, March 18, 1980, ST-BR-WC-1103, regarding structural backfill investigation.

21. WCC letter June 20, 1980, ST-WC-BR-5696, transmittal of Dr. Seed's letter dated June 16, 1980.

22. Seed, H. Bolton, "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes", Journal of Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 105, GT 2, pages 201-255, February 1979.

23. Brown & Root letter July 25, 1980, ST-BR-WC-1147, transmitting Expert Committee's Interim Report on Adequacy of Category I Structural Backfill, STPEGS, Document No. Y310XR1376BSH.

24. WCC letter dated August 15, 1980, ST-WC-BR-5729, regarding clarification of Dr. Seed's evaluations presented in references 17 and 19.

25. WCC report, Excavation and Backfill, Power Plant Structures, South Texas Project, Units 1 and 2, ST-WC-BR-0370, August 8, 1975, Y050XR037-WC.

TABLE 1

BORING DESIGNATION, LOCATION AND DRILLER

Field Boring No.	FSAR Boring No.	Unit	Driller*
WCBV-101	312	1	C
WCBV-102	313	1	C
WCBV-103	314	1	C
WCBV-104	315	1	C
WCBV-105	316	1	C
WCBV-106	317	1	C
WCBV-201	318	2	Y
WCBV-202	319	2	Y
WCBV-203	320	2	C
WCBV-204	321	2	C
WCBV-205	322	2	Y
WCBV-206	323	2	Y
WCBV-207	324	2	C
WCBV-208	325	2	Y
WCBV-209	326	2	Y
WCBV-201A	318A	2	Y
WCBV-204A	321A	2	C
WCBV-204B	321B	2	C
WCBV-205A	322A	2	Y
WCBV-205B	322B	2	Y
WCBV-208X	325B	2	C

* Driller designation: (Data from field boring logs.)

C - Custom Auger Drilling Service, Inc.
Y - Younger Drilling Company

TABLE 2

STANDARD PENETRATION TEST
SPLIT-BARREL DRIVE SHOE DIMENSIONS

Measurement	ASTM D-1586 Requirements	Custom Auger Shoe	Terzaghi Shoe	Commercially Available ASTM Shoe	WCC Drilling Procedure (Ref.7)
Shoe O.D.	2"	1.98"	2.00"	2.00"	2" ± 1/16"
Shoe I.D.	1 3/8"	1.45/1.43" (1)	1.40"	1.37"	1-3/8" ± 1/16"
Shoe Taper Length	3/4"	0.50"	0.62"	0.75"	3/4" ± 1/16"

Note:

(1) The leading edge of the drive shoe measured 1.45" I.D. and the upper throat of the drive shoe measured 1.43" I.D.

TABLE 3A

TOTAL FORCE REQUIRED TO MOVE SPLIT-BARREL SAMPLER WITH DRIVE SHOES OF DIFFERENT CONFIGURATIONS IN A DENSE SAND

Sample Depth Below Ground Surface	ASTM D-1586 (2) Shoe	Custom Auger Shoe	Terzaghi Shoe
20 ft.	14.0 kips (1)	13.4 kips	13.1 kips
40 ft.	27.5 kips	26.3 kips	25.8 kips
60 ft.	41.0 kips	39.3 kips	38.5 kips
80 ft.	54.5 kips	52.2 kips	51.3 kips

Note: (1) Forces rounded to nearest tenth number.

(2) See Table 2 for drive shoe dimensions.

Analysis performed for spoons embedded in dense sand with a relative density of approximately 85% to 95%.

TABLE 3B

TOTAL FORCE REQUIRED TO MOVE SPLIT-BARREL SAMPLER WITH DRIVE SHOES OF DIFFERENT CONFIGURATIONS IN A LOOSE SAND

Sample Depth Below Ground Surface	ASTM D-1586 (2)	Custom Auger Shoe	Terzaghi Shoe
60 ft.	13.5 (1)	12.3	12.3

Note: (1) Forces rounded to nearest tenth number.

(2) See Table 2 for drive shoe dimensions.

Analysis performed for spoons embedded in loose sand with a relative density of approximately 55% to 65%.

TABLE 4
STANDARD PENETRATION TESTS WITH A DENSITY LESS THAN
CONSTRUCTION QUALITY CONTROL CRITERIA OF 80% RELATIVE DENSITY

AREA	BORING	SAMPLE NO.	SAMPLE DEPTH FT	SAMPLE ELEVATION FT	RELATIVE[(1)] DENSITY, %	STANDARD PENETRATION N-VALUE	N-VALUE REQUIRED FOR FS = 1.5 (REF. 21)	N-VALUE REQUIRED FOR FS = 1.5 IN ISOLATED LOOSE POCKET (REF. 21)
1	204A	1	63.5 to 65.0	-36.2 to -37.7	46.2	15	16.5	12
	204B	9	68.2 to 69.7	-40.9 to -42.4	78.1	52	17	12
	204V2	4	68.0 to 69.5	-38.4 to -39.9	70.6	41	17	12
	204V2	5	69.9 to 71.4	-40.3 to -41.8	60.0	29	17	12
	204V4	4	65.6 to 67.1	-38.3 to -39.8	66.0	34	16.5	12
	204V4	5	67.6 to 69.1	-40.3 to -41.8	65.4	34	16.5	12
	204V4	6	69.6 to 71.1	-42.3 to -43.8	54.0	22	17	12
	204V5	1	60.7 to 62.2	-31.3 to -32.8	73.4	43	16.5	12
	204V6	1	58.6 to 60.1	-31.3 to -32.8	75.0	44	16.5	12
	204V6	3	63.6 to 65.1	-36.3 to -37.8	70.4	39	16.5	12
	204V6	4	65.6 to 67.1	-38.3 to -39.8	77.0	50	17	12
	204V9	1	67.9 to 69.4	-38.3 to -39.8	62.4	31	17	12
	204V9	2	69.9 to 71.4	-40.3 to -41.8	40.0	11	17	12
	204V10	1	66.5 to 68.0	-38.3 to -39.8	65.6	34	17	12
	204V10	2	68.5 to 70.0	-40.3 to -41.8	57.2	25	17	12
	204V11	4	68.7 to 70.2	-40.4 to -41.9	42.7	13	17	12
	204V13	4	69.7 to 71.2	-40.3 to -41.8	56.7	25	17	12
	204V15	1	57.1 to 58.6	-27.3 to -28.8	51.1	18	16.5	12
	204V15	2	59.1 to 60.6	-29.3 to -30.8	62.3	29	16.5	12
2	205	14	33.5 to 35.0	- 7.8 to - 9.3	74.3	32	16.5	12
	205B	4	33.7 to 35.2	- 8.0 to - 9.5	72.0	30	16.5	12
3	208	12	28.5 to 30.0	- 0.5 to - 2.0	67.8	25	16.5	12
	209	12	28.5 to 30.0	- 0.1 to - 1.6	66.1	24	16.5	12
4	203	18	43.4 to 44.9	-16.7 to -18.2	66.6	28	16.5	12
	203	25	61.1 to 62.6	-34.4 to -35.9	78.8	50	16.5	12
	203V2	7	58.5 to 60.0	032.0 to -33.5	78.9	50	16.5	12

(1) Relative Density calculated using actual ground elevation at boring, densities corresponding to 80% relative density and average groundwater elevation beneath Units 1 or 2.

TABLE 5

N-VALUES REQUIRED TO PROVIDE LIQUEFACTION POTENTIAL
FACTOR OF SAFETY (FS) AT DIFFERENT DEPTHS
IN THE STP STRUCTURAL BACKFILL

Depth Below Ground Surface - ft	N-value required for FS = 1.5	N-value required for FS=1.5 in isolated loose pocket*
20	14.2	10
40	16.5	12
60	16.5	12
70	17	12.3

* Note: The N-value required to prevent liquefaction in an isolated loose pocket is less than that required in a uniform mass because the strains in the loose pocket are controlled by the stiffness of the surrounding denser soil but the stresses are determined by the shear modulus of the looser sand in the pocket. (Ref. 17).