CRYSTAL RIVER UNIT 3

HURRICANE STUDY

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CRYSTAL RIVER UNIT NO. 3

HURRICANE STUDY

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1.0 SUMMARY

The purpose of this report is to present the method of protecting Crystal [illegible]r Unit No. 3 from the Probable Maximum Hurricane (PMH) whose parameters were obtained from ESSA Memorandum HUR 7-97 and described in detail in "Report - Verification Study of Dames and Moore's Hurricane Storm Surge Model with Application to Crystal River Unit No. 3 Nuclear Plant - Crystal River Florida - for Florida Power Corporation". Conjunctively, this report responds to questions asked by the Atomic Energy Commission in their letter to Florida Power Corporation (FPC) dated March 12, 1973.

2.0 CONCLUSIONS

It is concluded without reservation that with the protection described herein, Crystal River Unit No. 3 will be completely and conservatively protected against the PMH and that safe shutdown conditions can and will be effected and maintained during the PMH.

3.0 INTRODUCTION

3.1 Current Studies

As a result of studies completed in October 1972, Crystal River No. 3 was designed to safely withstand a PMH which would produce a surge level of 29.6 feet above the mean low water (MLW) elevation of 88 feet. A more current study has been completed recently by Dames and Moore and the results are included in their report titled, "Report - Verification Study of Dames and Moore's Hurricane Storm Surge Model with Application to Crystal River Unit No. 3 Nuclear Plant - Crystal River Florida - for Florida Power Corporation" dated July 13, 1973 hereinafter referred to as the Dames and Moore Report. The new report includes the PMH parameters along with substantiation and justification of the results if such a PMH should occur. Primary information includes a surge level of 29.4 feet above MLW elevation of 88 feet and a resulting "waveless" surge to elevation 119.5 feet over the top of the embankment whose design elevation is 118.5 feet. The hurricane would track from southwest to northeast.

3.2 AEC Questions

On March 12, 1973 the Atomic Energy Commission advised Florida Power Corporation that additional information was required regarding the Probable Maximum Hurricane (PMH) surge level and the associated wave runup. This report therefore also addresses itself to the following questions which were presented in Enclosures (1) and (2) of the AEC letter. Questions 1 through 5(c) of Enclosure

(1) are addressed in the aforementioned Dames and Moore report dated July 13, 1973.

Enclosure (1)

5(d) For each safety-related structure, system, and component identified as necessary for plant protection (see Request 2.16 in enclosure (2)), and based on both a stillwater level of 33.4 ft. MLW and your fully verified stillwater elevation estimate, provide tabulations of the height of the most significant (average of the highest one-third) and the maximum (1 percent) waves, or the breaking waves (whichever is the most severe) and the associated runup for each case.

5(e) Discuss the applicability of your hydraulic model studies for estimating runup on and over the soil-cement protected embankment and on interior facilities for both water levels and wave conditions discussed herein.

Enclosure (2)

2.15.3 Identify those safety-related structures, systems, and components necessary for safe operation (see Safety Guide 29). Compare the conditions identified in Request 2.15.1 above with the design bases and general adequacy of each facility to perform its required function, and indicate any action required to assure functionability for hurricane conditions up to those requiring shutdown.

2.16 For hurricane conditions more severe than those for which operation would be allowed, up to and including PMH conditions that both you and the staff have estimated, identify those safety-related structures, systems and components necessary to assure maintenance of shutdown conditions. Discuss the ability of each structure, system and component to withstand both the static and dynamic consequences of hurricanes up to and including those of PMH severity for both still-water level estimates.

2.17 Provide the minimum submergence levels for both circulating and service water pumps.

2.18 We understand that the soil-cement protected embankment is required to maintain the function-ability of safety-related facilities during hurricane conditions. Substantiate its ability to withstand the static and dynamic consequences of water level and frontal wave action for both PMH estimates. Documentation may consist of reference to other control facilities which have experienced conditions similar to those postulated for the Crystal River site, to full-scale hydraulic model studies, or its analytical studies of static and dynamic forces. Also, discuss the ability of the protection and the embankment to withstand wave overtopping. If the embankment is not required for hurricane protection, provide your assumption of its failures during such events and the consequences of failure on required safety-related facilities.

2.19 Prov de substantiated assurance of the ability of s..ety-related structures, systems, and components necessary for safe operation, and those required for cold shutdown and maintenance thereof, to withstand rainfall and spray; either associated with severe hurricanes, or independently thereof. For instance, discuss the ability of site drainage, including the roofs of safety-related structures and exterior penetrations, to safely store or pass runoff without a loss of function.

4.0 PMH CONDITIONS AT PLANT

4.1 Storm Surge Level

The maximum storm tide level provided by Dames and Moore is elevation 117.4 feet (MLW is at elevation 88.0 feet). This determination considered the topography of the site along the approach path and the critical section shown on Figure 1. The maximum storm tide level also considered a two-foot reduction due to the effects of backwater storage resulting from the extensive flooding of the surrounding countryside some five hours prior to the peak of the surge hydrograph and runoff into peripheral areas not directly affected by the hurricane surge. The site in this extreme circumstance becomes analogous to a small island in a mountain of water. The storm surge hydrograph is shown on Figure 4, and includes the combined effects of the hurricane surge and the astronomical tide.

4.2 Wave Action and Runup

4.2.1 Figure 6 summarizes the relationships between the stillwater level, wind-generated wave height, breaking wave height and wave runup. The maximum (highest 1 percent of the waves) and significant (average of the highest one-third) wave heights were determined by wind vectors normal to the coast along the traverse, and were calculated using the storm surge computer output. Figure 5 presents a summary of this analysis for wind-generated waves approaching the site. On Figure 6, the intersection of the breaking wave curve and the generated wave height curves show that with the highest 1 percent of the waves breaking, the maximum height of the waves that can travel across the fill approaching the plant is 15.0 feet. With the average of the highest 33 percent (shown as Hs) of the waves breaking, the maximum height of the waves that can reach the protective embankment without breaking is 13.9 feet.

4.2.2 Maximum tidal setup will be produced by winds blowing onshore along a traverse normal to the offshore bottom contours. Consequently, the critical approach path for a hurricane was from the southwest, tracking on a northeasterly course. However, the approach path of wave trains that will produce maximum runup at the site is along a north-south section with the waves approaching the plant from the south. This critical traverse of the wave train is across a reach of natural ground about one mile wide, then over 600 feet of compacted fill (elevation 98), and against an embankment slope (berm) rising to a top elevation of 118.5 feet, which protects the plant. This concept of maximum wave action occurring perpendicular to the hurricane winds was considered to be a conservative assumption for this already extremely severe condition.

4.2.3 The effects of breaking waves and wave runup on the embankment slope were evaluated from the model tests previously conducted at the University of Florida. Before performing the runup tests, experiments were conducted to determine the most adverse test conditions (i.e., the combination of wave period and height which caused the maximum runup over the tidal range of interest). From these pre-test experiments and periodic checks during the tests, it was found that the maximum runup occurred with a prototype wave period of 5.4 seconds, and prototype wave amplitudes of 10 to 15 feet, the specific height depending on the test case. The wave action testing was conducted at prototype tide levels from elevation 104 to 120 feet in two-foot increments. Unlike the spectrum of wind-generated waves, the model tests were conducted with waves essentially uniform in height.

4.2.4 Initial tests were conducted on a smooth slope embankment. Subsequent tests simulated the runup effects against the stepped embankment. The test results indicated no overtopping of the smooth slope below tide levels of 110 feet, with occasional overtopping starting at a tide level of 112 feet and becoming continuous above elevation 114. Tests on the stepped slope revealed no overtopping below tide elevation 112 feet, slight overtopping at 114, and more continuous overtopping above elevation 116 feet. Pertinent results of the model tests are shown on Figures 2 and 3 and on Tables 1, 2 and 3.

4.2.5 The applicability of the model tests for the 29.4-foot surge level is illustrated by Figure 3 and amplified by two letters from Dr. R. G. Dean dated April 3, 1972, and April 11, 1973, contained in the Attachments. As indicated by Figure 3, the median and maximum runup elevations occurring at a tide level of 117.4 feet are 122.5 and 123.5, respectively on Profile 5, the profile corresponding to the stepped slope that will be constructed at

Crystal River Nuclear Station. The actual elevation of th water overtopping embankment at elevation 118.5 will be somewhat less than the elevation of the runup on the test slope because with the limited slope height (the steps above 118.5 used in the model do not exist in the actual design) the surging water will reach elevation 118.5 and merely fall over on the embankment.

4.2.6 For a slope of unlimited height, the maximum and median runup corresponding to the indicated stillwater hydrograph were developed using Figure 3 and are shown on Figure 6. When the height of the wind-generated waves reaching the protective embankment becomes 10 to 15 feet (the range of wave heights found from the model tests to cause the greatest runup), the results of the model tests become applicable. Until that time, the wind-generated waves would produce less runup than indicated; the runup from the test results is therefore shown as a dotted line. Employing the conservative assumption that the wave height in the model tests was the "maximum" generated wave height (i.e., assuming that all of the waves attacking the embankment are "maximum" waves), Figure 6 shows that the model results become applicable at 23.2 hours after the center of the hurricane crosses the continental shelf. This is also the time of maximum stillwater level.

It is estimated that overtopping of the embankment by the maximum runup begins about hour 22.7 and continues until hour 24.2. For the reasons described, the elevation of the water overtopping the edge of the actual embankment will be less than the runup elevations shown in Figure 6. In this 1.5 hour period, the maximum depth of stillwater at the safety class structures nearest the edge of the embankment (a distance of about 100 feet) due to overtopping, is estimated to be one foot. At locations along the plant embankment that are not exposed to direct wave attack, overtopping should not occur. Water that does overtop the embankment on the windward side of the plant will drain off the embankment on the lee side.

5.0 VITAL EQUIPMENT AND ITS PROTECTION

5.1 Continuous Power Generation

5.1.1 Under any conditions, the equipment necessary for power generation is as follows:

1. On-site diesel power generators, and their support equipment (fuel systems, cooling systems, switchgear).
2. Reactor decay heat removal equipment
   a. Nuclear services closed cycle cooling system.
   b. Decay heat removal system.
   c. Decay heat closed cycle cooling system.
   d. Decay heat sea water system.
   e. Nuclear services sea water system.
3. Circulating water pumps.
4. Electric transmission facilities (switchyard).
5. Turbine-generator support equipment.

5.1.2 Power generation at sea levels above elevation 98 is not possible since the circulating water pumps, located at the intake canal, and the transmission facilities in the swi .ayard will begin to flood.

5.1.3 The circulating water pumps begin to cavitate when the water level drops below elevation 81 feet; however these pumps do not serve any equipment necessary to maintain the reactor in a safe condition.

5.2 Equipment Necessary for Safe Shutdown

5.2.1 Systems listed under items 1 and 2 Subsection 5.1.1 must remain functional during storms of any degree of severity up to and including the PMH. Ability of this equipment to remain functional is assured by the facility design discussed herein.

5.2.2 On-site emergency power generation equipment is located within the main structure, and is protected from flooding by concrete barriers. Fuel storage tanks are located underground, and are restrained against damage from their own bouyancy by hold down straps and concrete anchor slabs. Tank vents are above postulated wave tops to prevent sea water from entering the tanks via the vent lines. Diesel engine cooling is provided by a self-contained air radiator system within the structure.

5.2.3 The nuclear services closed cycle cooling system, decay heat removal system, and decay heat closed cycle cooling system are located within the auxiliary building, which is protected from flooding by water-tight doors where necessary.

5.2.4 Pumps and heat exchangers serving the decay heat sea water and nuclear services sea water systems are also located within the protected auxiliary building. Sea water is admitted to the pump sump chambers via two conduits connected to the intake structure at the intake canal. Although this intake structure will be inundated at the postulated hurricane sea levels, no active equipment necessary to maintain the reactor in a safe condition is located at the structure.

5.2.5 The sea water pumps located within the auxiliary building, take suction from a chamber designed for a static sea level of 140 feet, which is well in excess of the postulated hurricane sea levels. These pumps require a minimum pump submergence level of elevation 70 feet, 10-1/2 inches (water surface elevation) for satisfactory operation. The increase in static pressure on pumps, piping, heat exchangers, and other components in the sea water systems. due to the increased pump suction pressure, are less than the design conditions for these components.

5.2.6 The above equipment is powered by the emergency diesel generators. In addition, if power should be lost, sump pumps can be operated from the emergency diesel generators, to dispose of any leakage, through water-tight door seals.

6.0 DETAILED FLOOD PROTECTION

6.1 Protective Assumptions

6.1.1 Figure 8 indicates the locations of seals, water-tight doors, concrete barriers and walls which have been included in the design to protect access openings in the structure which may be subjected to flooding during the peak of the PMH (about 1.5 hours).

6.1.2 It should be noted that although the anticipated water level during the peak of the PMH is only to elevation 119.5 on the south decreasing to elevation 117.4 on the north, protection has been conservatively provided as though the maximum wave runup to elevation 123.5 exists over the entire south embankment.

6.2 Component Protection

Component protective facilities required for local protection are shown on Figure 8 and are as follows:

1. Turbine building (already protected to elevation 121'-10" except at door openings whose thresholds are at elevation 119 feet).
   a. Five water-tight doors at door openings will be provided to elevation 122 feet.
   b. Water-tight doors will also be provided for the two air shafts located on the west and east sides of the turbine building to elevation 122 feet.

2. Auxiliary building (thresholds of door openings are at elevation 119 feet).
   a. Three water-tight doors at door openings will be provided to elevation 122 feet on the east and to elevation 124 feet on the south.
3. Diesel generator building (thresholds of openings are at elevation 119 feet).
   a. Two concrete barriers will be constructed at the outer side of the air intake enclosure walls to elevation 124 feet.
4. Reactor building
   a. A concrete water barrier will be constructed up to elevation 124 feet, approximately three feet outside the present reactor building wall between the equipment access hatch and the intermediate building, and between the equipment hatch and the auxiliary building. Each wall will have a water-tight seal at its extremeties.
   b. Equipment access hatch will be provided with a water-tight door at the entrance to elevation 124 feet.

5. Water-tight seals will be installed on:
   a. Tendon gallery access hatch
   b. Heat exchanger room hatch
   c. At reactor building barrier walls
   d. At interfaces between the diesel generator and auxiliary buildings.

e. Underground electrical conduit entering the building below grade

6. The vent pipes on diesel fuel tanks will be raised to prevent the entrance of excessive amounts of water.

7. Water-tight door will be provided at the entrance to the borated water storage tank area to elevation 124 feet. (Threshold elevation is at 119 feet).

6.3 Structural Integrity

All safety-related structures have been checked for both new high water criteria (surge levels of 117.4 feet and 121.4 feet) and found to be structurally adequate to sustain the forces, both static and dynamic, caused by the PMH. The intake structure was also found to be structurally adequate to sustain the forces caused by the PMH.

6.4 Rainfall Protection

6.4.1 With the exception of the diesel generator, all other components are protected from rainfall and water spray created from high winds.

6.4.2 In the diesel generator building it is possible for spray to be driven into the air intake opening; however, this contingency has been prepared for by providing suitable floor drainage to intercept the water and conduct it away from the area.

6.4.3 Outside the structure, the site drainage system has been designed to preclude ponding, even during the Probable Maximum Precipitation (PMP). The design utilized the Rational Method with the following parameters:

Rainfall Intensity - 10 inches per hour

Runoff Coefficients: 0.95 for roof surfaces
0.80 for paved areas
0.40 for soil

6.4.4 The greatest overland distance that runoff must travel to reach a catch basin is only 200 feet, at a minimum ground slope of 0.5%.

6.4.5 Roof drains discharge directly into the storm drainage system, and were designed to accommodate a rainfall intensity of 6 inches per hour. For this design capacity, no roof ponding will occur up to a 1000-year rainfall. In the event of the PMP, ponding could occur up to a maximum of three inches around the eaves of the structures, and buildup beyond this would overflow the eaves. The roof structures have been adequately designed to support the ponded water.

6.5 Sump Pumps

Internal sump pumps are located as follows:

Pump Locations	Quantity	Capacity	Discharge to:
Turbine Room	2	500 gpm	Industrial Waste Treatment Pond
Nuclear Service Cooler Area	2	250 gpm	Sea Water Discharge Canal
Condensate Pump Pit	2	50 gpm	Turbine Room Sump

Pump Locations	Quantity	Capacity	Discharge to:
Tendon Access Gallery	2	50 gpm	Sea Water Discharge Canal
Auxiliary Building	2	125 gpm	Misc. Waste Storage Tank
Reactor Building	2	100 gpm	Misc. Waste Storage Tank
Decay Heat Pit	2	30 gpm	Misc. Waste Storage Tank
Laundry and Shower	1	30 gpm	Neutralizer or Misc. Waste Storage Tank
Turbine Room	2	150 gpm	Sewage System

7.0 WATER-TIGHT DOORS

With the exception of the large turbine room door where the track enters and the two large openings on the south in the fuel handling area, permanent doors will be installed on the structure ready to be closed in the event of an emergency. These doors will feature fail-proof, leak-proof, compression-type seals that are activated when the door is closed and latched. The remaining three large openings mentioned above will require mounting of flood panels when the need arises. These panels will feature expandable seals that compress against the door casement. In the unlikely event of a seal failure, these panels will have a compression-type seal as a backup measure. If the main seal should fail, hydrostatic pressure will force the panel against the back face of its guide slot compressing the backup seal.

8.0 PROTECTIVE EMBANKMENT

8.1 General Description

8.1.1 The plant structure is protected by a surrounding embankment constructed to elevation 118.5, which will be placed upon in-situ material at elevation 98. (See Figure 7). Because it is vital to the protection of the plant, the embankment along the south and west sides of the plant has been designed to withstand the dynamic forces of the maximum [illegible] and ensuing runup for the Probable Maximum Hurricane. The usual type of protection for this condition would be an adequate thickness of large size dumped riprap. Because of the absence of suitable riprap material in the area, soil-cement was originally considered as an effective means of preventing erosion of the slopes of the protective embankment. The original selection of soil-cement as the means for providing protection for the embankment was predicated on four basic considerations:

1. The absence of rock or other suitable similar material in the area.
2. The availability of native limerock with a very high calcium carbonate content, which will permit the soil base to react chemically in a soil-cement mixture, developing an extremely coherent internal structure. (The results of tests on this limerock material are contained in the Attachments.)
3. The results of soil-cement design mixes and tests conducted by Pittsburgh Testing Laboratory utilizing local native limerock and cement. (The tests results are contained in three reports included in the Attachments.)

4. Documentation of its successful use in Bonny and Merritt Dams by the Bureau of Reclamation.

8.1.2 Due to revised hurricane criteria and analyses, the surge level has been changed from an original elevation 112.6 to elevation 117.4. Wave heights have correspondingly increased from an initial height of 11.4 feet to 15.0 feet. To provide greater resistance against the increased wave forces, an armor covering of 3000 psi reinforced concrete over 1500 psi soil-cement is planned. The reinforced concrete will provide resistance to erosion and dynamic impact and the 1500 psi soil-cement will provide a stiff backing and act as a rigid material that the concrete covering will be keyed into. The criterion of 1500 psi for the soil-cement is based on a 90-day compressive strength. Thus, referring to Figure 7, it can be seen that the current design employes a layered system of in-situ lime-rock, compacted Zone III fill, soil-cement and reinforced concrete.

8.1.3 Stability analyses were undertaken for the embankment in order to establish the degree of stability the embankment possesses against a hypothetical failure along a circular arc passing through both the foundation and the embankment. Using the following parameters, the minimum factor of safety against failure is 4.3.

	Friction Angle (ϕ)	Cohesion (c)
Embankement	45.5°	0 tsf
Overburden	37.0°	0 tsf
Upper Foundation	47.0°	5 tsf
Lower Foundation	49.0°	5 tsf

8.2 Foundation Material

8.2.1 The foundation material upon which the embankment will be constructed was placed in 1964 from construction excavations on-site. This material has had nine years to consolidate, with considerable construction activity surcharge. No significant settlement is anticipated from this material or from the Zone III material placed in the embankment under 95 percent of Maximum Modified Proctor density compaction criteria required by GAI Specification SP-5901. Any potential settlement will occur prior to the placement of the concrete cover since the full weight of the embankment will be imposed on the foundation first during construction.

8.2.2 The characteristics of the limerock material existing as foundation material, and proposed for the embankment, are documented in the FSAR and in the Attachments of this report. The material is a friable limerock with a high magnesium carbonate and calcium carbonate content and the following general characteristics:

Specific Gravity	2.57-2.72
Liquid Limit	26
Plasticity Index	NP
Absorption	29.5%
Compacted Void Ratio	0.38
Compacted Coeff. Permeability	10^{-5} cm/sec
Maximum Dry Density	112.8-121.6 pcf
Optimum Moisture	11.1-14.1%
Triaxial Shear Strength	$\emptyset = 45.5^{\circ}$ c = 0
Average Compaction during Placement on Site	98.4% Maximum Modified Proctor

8.3 Zone III Fill

8.3.1 The general characteristics of Zone III fill were mentioned in Subsection 8.2.2. When compacted to 95% of Modified Proctor density, as required by Specification SP-5901, Zone III becomes a very dense and stable material.

8.3.2 The gradation of the material is shown on Figure 9 as a range covering samples tested during placement in the embankment and in soil-cement design mix tests. It should be noted that in 40 tests of field density, the compaction averaged over 98 percent of maximum Modified Proctor density.

8.3.3 The compacted Zone III material has a low permeability of 1 x 10^{-5} cm/sec and it was noted in conducting triaxial shear tests that difficulty was experienced in attempting to saturate the samples. This characteristic will be beneficial in preventing potential uplift forces from developing beneath the surface materials. ne embankment mass will not respond quickly to saturation from increasing tide levels that have a duration of only 10 hours since several weeks would be required to totally saturate the mass.

8.4 Soil-Cement

8.4.1 The design of the soil-cement section was based upon the results of its use and documentation in Bonny and Merritt Dams by the Bureau of Reclamation, and also on the results of three design-mix programs conducted by Pittsburgh Testing Laboratory, which are included in the Attachments to this report. The use of soil-cement in these cases was for the actual armor protection, whereas now it is being used only as the backing for the concrete armor. However, soil-cement has been successfully used on the following dams:

Dam	Location	Date	Agency
Bonny Dam	Colorado	1951	Bureau of Reclamation
Merritt Dam	Nebraska	1961	Bureau of Reclamation
Cheney Dam	Kansas	1962	Bureau of Reclamation
Ute Dam	New Mexico	1963	N.M. Interstate Stream Commission
Holiday Dam	Pennsylvania	1963	Holiday-Pocono Land Inc.
Okay Levee	Arkansas	1964	Corps of Engineers
Glen Elder Dam	Kansas		Bureau of Reclamation

8.4.2 The results of the three test programs conducted using Crystal River materials are contained in the Attachments. Mixes A and B were tested in October of 1969 utilizing Zone III material and Type II cement in proportions of 9 and 10 percent by weight. Average 28-day compressive strengths for Mixes A and B were 1140 and 1420 psi, respectively. The second program in November 1969 used also Zone III Material and Type II cement in three mixes containing 3, 4 and 4.5 bags/cu. yd. These mixes developed average 90-day compressive strengths of 1850, 2150, and 2960 psi, respectively. The third program in July 1970 used Type I cement in proportions of 7, 8 and 9 percent by weight with Zone III material. Average 90-day compressive strengths for Mixes 1, 2 and 3 were 930, 1100 and 1240 psi, respectively. The test results for all three programs are summarized in Tables 4, 5, 6 and 7 and on Figures 10, 11 and 12.

8.4.3 Placement of soil-cement shall be in accordance with GAI Specification SP-5901 and will yield a 90-day compressive strength of 1500 psi.

8.5 Reinforced Concrete Armor

8.5.1 The use of concrete as protection is quite common. Concrete has been used to pave dams and levee slopes, used as shore protections along coast lines, used as dams to contain lakes and reservoirs, and used as spillways which are subjected to a degree of abuse which could never be equaled in its use as the embankment protection at Crystal River 3.

8.5.2 During a PMH, the concrete armor would be subjected to a maximum dynamic load of 32 psi. Although the probability of a washout ever occurring beneath the armor is nearly zero, for design purposed it was assumed that the individual concrete panels, whose maximum size will be 20 feet square, had a washout occur beneath them. It was also assumed that the washout left a 15-foot square area unsupported, thus introducing bending stresses. Based on working stress design, the concrete reinforcement was designed to resist the bending stresses in this unsupported condition. Both the concrete and reinforcing stresses are within the allowable tolerences provided by ACI-318-63, based on working stress design.

8.5.3 Since the chances of a washout are unlikely, the armor will always be continuously supported. The only induced stresses, therefore, will be purely compressional. This, then, indicates that the armor is very conservatively designed.

8.5.4 The mixing and placement of concrete shall be in accordance with GAI Specifications SP-5569 and SP-5618.

9.0 PROTECTION AGAINST A SURGE LEVEL OF 33.4 FEET

9.1 Design Conditions

If it is required to use the 33.4 foot surge level, the stillwater level hydrograph, generated wave height, breaking wave height and wave runup would be approximately as shown on Figure 15. As can be seen from Figure 15, the intersection of the breaking wave curve and the generated wave height curves shows that with the highest 1 percent of the waves breaking, the maximum height of the waves that can travel across the fill approaching the plant is 18.0 feet. With the average of the highest 33 percent (shown as Hs) of the waves breaking, the maximum height of the waves that can reach the protective embankment without breaking is 15.5 feet.

9.2 Alterations to Plant Protection

If one must consider protection against a surge level of 121.4 feet, the following changes would need to be made to the existing protection scheme:

1. The embankment on the south and west sides would be increased in height to elevation 127 feet (Refer to Figure 14). This would be necessary in order to intercept waves and floating debris such as an oil barge before the structure is struck.
2. Barriers and water-tight doors with a top elevation of 124 feet would increase in height to elevation 127 feet.
3. Water-tight doors with a top elevation of 122 feet would increase in height to elevation 127 feet except openings on the east side of structure which would increase to elevation 124 feet.

4. A concrete barrier with a top elevation of 127 feet must be built at a distance of about 2 feet outside the present turbine room wall (See Figure 13). This barrier would extend from the main transformers toward the west and around to the air shaft located on the west side of the turbine room.
5. Prior to high water, the neutralizer tank, condensate storage tank and fire water storage tanks must be filled for stability.

9.3 Likelihood of Surge to Elevation 121.4

9.3.1 The unlikelyhood of the surge reaching a level of 121.4 feet is is verified in the Dames and Moore Report dated July 13, 1973. Because of this confidence, Section 9.0 of this report has been included only to answer question 5(d) of Enclosure 1 of the AEC letter to Florida Power Corporation dated March 12, 1973.

9.3.2 It is concluded that the protection described above for a surge level of 121.4 feet represents the same assurance of safe protection as described for the 117.4 foot surge level. However, it is further concluded that a lesser but safe level of protection could be accomplished, based on AEC requirements, by elimination of protection item 1 above and increasing the height of watertight doors and concrete barriers to elevation 129 feet, thus eliminating a significant cost penalty in protection schemes between surge levels of 117.4 and 121.4 feet.

REFERENCES

Report - Verification Study of Dames & Moore's Hurricane Storm Surge Model with Application to Crystal River Unit No. 3 Nuclear Plant, Crystal River, Florida, Dames and Moore, Inc. for Florida Power Corporation.

Report of Model Tests to Determine Extreme Runup at Florida Power Corporation, Crystal River Site, Department of Coastal and Oceanographic Engineering, Florida Engineering and Industrial Experiment Station, University of Florida, April, 1969.

Holtz, W. C. and Walker, F. C. Soil-Cement as Slope Protection for Earth Dams, Journal of the Soil Mechanics and Foundations Division, ASCE Proceeding, Vol. 88, SM-6, December, 1962.

Wilder, C. R. and Koller, E. R. Soil-Cement Protection for Earth Dams, World Dams Today, Japan Dam Association, p. 260-264, November, 1967.

Soil-Cement Slope Protection for Earth Dams, Portland Cement Association, 1965.

TABLES

TABLES

WAVE RUNUP AND OVERTOPPING FOR PROFILE 3 TESTS
(Stepped Slopes, Maximum Elevation 118.5 Feet)

Run No.	Tide Level (Ft.)	Elevation of Max Runup (Ft.)	Elevation of Median Runup (Ft.)	Depth of Max. Over-topping (In.)	Depth of Median Over-topping (In.)	Median Wave Height (Ft.)
29	118	118.5*	118.5	66	45	13.4
30	118	118.5	118.5	56	45	13.4
31	116	118.5	118.5	25	20	12.9
32	116	118.5	118.5	31	21	12.9
33	114	118.5	118.5	8	1	12.6
34	114	118.5	118.4	6	0	12.6
35	112	117.8	116.4	0	0	13.1
36	112	117.3	116.3	0	0	13.0
37	110	115.4	114.3	0	0	12.4
38	110	115.6	114.7	0	0	12.2
39	108	114.0	112.1	0	0	12.4
40	108	113.3	112.1	0	0	12.3
41	106	110.0	109.2	0	0	12.5
42	106	110.0	209.6	0	0	12.6
43	104	106.2	105.4	0	0	12.1
44	104	106.2	105.4	0	0	12.1

* Runup in excess of 118.5', shown as overtopping in Columns 5 and 6

TABLE 1
CRYSTAL RIVER UNIT 3
WAVE RUNUP MODEL TESTS
PROFILE 3
TEST RESULTS

RUNUP AND OVERTOPPING FOR PROFILE 4 TESTS
(Stepped Slopes, Maximum Elevation 124 Feet)

Run No.	Tide Level (Ft.)	Elevation of Max Runup (Ft.)	Elevation of Median Runup (Ft.)	Depth of Max. Over-topping (In.)	Depth of Median Over-topping (In.)	Median Wave Height (Ft.)
45	120	124.0*	124.0	40	31	16.2
46	120	124.0	124.0	40	30	16.2
47	118	124.0	124.0	11	3	15.4
48	118	124.0	124.0	7	0	14.5
49	116	122.8	121.8	0	0	14.8
50	116	122.9	121.8	0	0	14.8
51	114	120.1	118.6	0	0	14.8
52	114	119.5	118.6	0	0	15.0
53	112	117.9	116.6	0	0	14.5
54	112	117.6	116.6	0	0	14.8

* Runup in excess of 124.0' shown as overtopping, in Columns 5 and 6

TABLE 2
CRYSTAL RIVER UNIT 3
WAVE RUNUP MODEL TESTS
PROFILE 4
TEST RESULTS

RUNUP FOR PROFILES 5 TESTS
(Layout No. 3, Section B by GAI)

Run No.	Tide Level (Ft.)	Elevation of Max Runup (Ft.)	Elevation of Median Runup (Ft.)	Median Wave Height (Ft.)
55	118	124.2	123.1	12.2
56	118	123.8	123.1	12.2
57	116	122.1	121.0	11.6
58	116	122.2	121.0	11.6
59	114	119.8	119.1	12.9
60	114	119.5	118.9	12.9
61	112	117.1	116.1	11.9
62	112	116.1	115.1	11.2
63	110	114.4	113.3	10.5
64	110	113.9	113.0	11.0

TABLE 3
CRYSTAL RIVER UNIT 3
WAVE RUNUP MODEL TESTS
PROFILE 5
TEST RESULTS

Design Mix	Max. Dry Density, PCF	Optimum Moisture, %	Compressive Strength, psi
Mix "A"			
7-Day Cylinders	113.1	14.9	865
	112.6	14.5	955
	112.6	14.5	845
14-Day Cylinders	113.3	14.9	1070
	113.3	14.9	1030
	113.5	14.7	1040
28-Day Cylinders			1150
			1095
			1180
Mix "B"			
7-Day Cylinders	113.0	14.7	885
	113.1	14.7	1040
	112.8	14.8	960
14-Day Cylinders	111.1	14.8	1250
	110.9	15.2	1170
	111.4	15.6	1230
28-Day Cylinders			1420
			1360
			1475

Note: Design "A" 286 lb. Type II cement @ Max. Density with OW/C + 1-2%
Design "B" 312 lb. Type II cement @ Max. Density with OW/C + 1-2%

TABLE 4
CRYSTAL RIVER UNIT 3
SOIL-CEMENT DESIGN MIXES
OCTOBER, 1969

Specimen	Age (Days)	Size	Strength - psi
1500 psi Concrete	7	6" x 12"	155
1500 psi Concrete	7	6" x 12"	175
Soil-Cement Mix A	14	4" x 4.6"	185
Soil-Cement Mix B	14	4" x 4.6"	215
Soil-Cement Mix A	28	-	210
Soil-Cement Mix B	28	-	235

TABLE 5
COMPARISON OF SOIL-CEMENT
WITH 1500 PSI CONCRETE IN
SPLITTING TENSION
(ASTM C-496)

Property	Mix No. 1 3.0 Bags/cu yd	Mix No. 2 4.0 Bags/cu yd	Mix No. 3 4.5 Bags/cu yd
Compressive Strength			
7-Day cylinders (Av)	1025 psi	1800 psi	-
14-Day cylinders (Av)	1060 psi	1600 psi	-
28-Day cylinders (Av)	1460 psi	1935 psi	2800 psi
90-Day cylinders (Av)	1850 psi	2122 psi	2960 psi
Moisture Content			
7-Day cylinders (Av)	11.2%	11.5%	-
14-Day cylinders (Av)	10.9%	11.8%	-
28-Day cylinders (Av)	11.1%	11.1%	10.5%
90-Day cylinders (Av)	10.8%	11.4%	11.2%
Molded Weight			
7-Day cylinders (Av)	117.5 pcf	120.6 pcf	-
14-Day cylinders (Av)	116.3 pcf	120.0 pcf	-
28-Day cylinders (Av)	117.8 pcf	120.3 pcf	120.0 pcf
90-Day cylinders (Av)	118.6 pcf	120.3 pcf	120.1 pcf

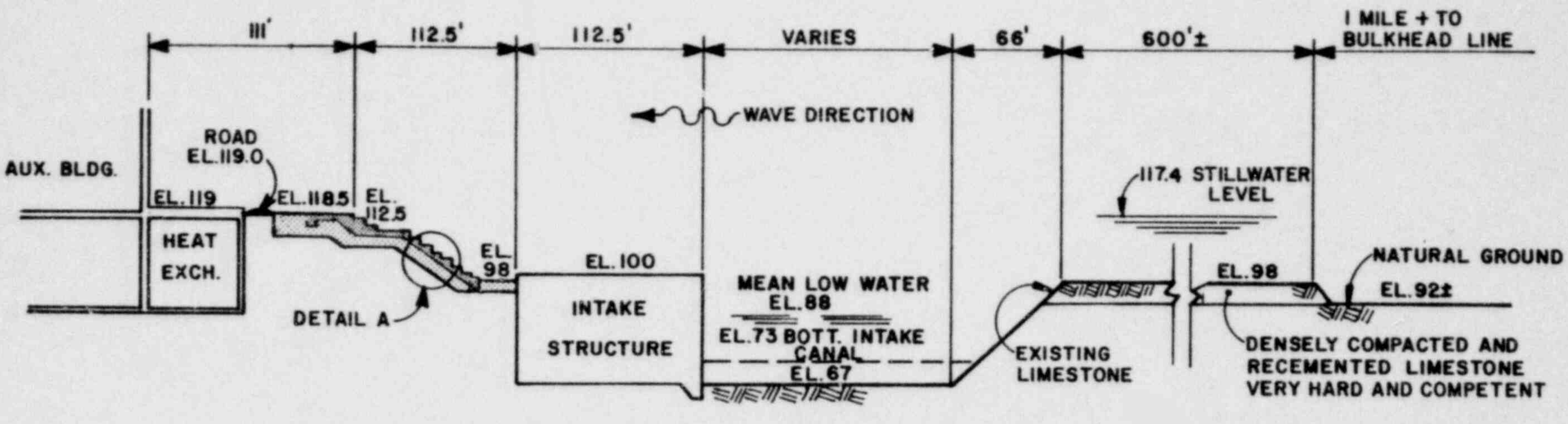
TABLE 6
CRYSTAL RIVER UNIT 3
SOIL-CEMENT DESIGN MIXES
NOVEMBER, 1969

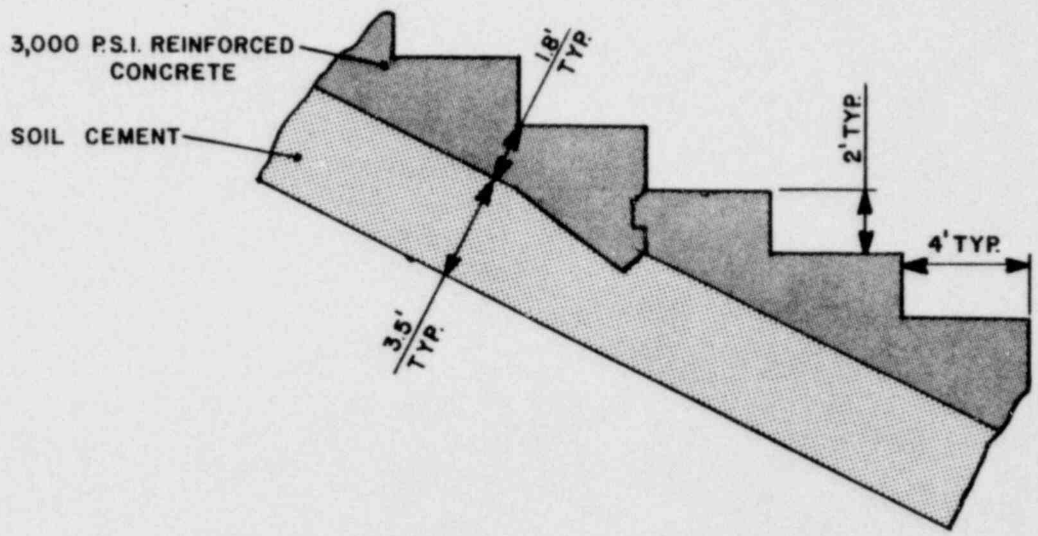
Property	Unit	Mix No. 1	Mix No. 2	Mix No. 3
Cement Content	lbs/cu yd	219	251	282
Cement Content	% by Wt.	7	8	9
Density (ASTM D558)	PCF	117.3	118.5	117.9
Optimum Moisture	%	13.5	13.4	13.7
Vol. Change (ASTM D559)	%	1.2	1.1	1.1
Wt. Loss (ASTM D559)	%	1.5	1.8	0.6
Compressive Strengths				
7-Days		490	560	650
28-Days		720	960	1080
90-Days		930	1100	1240

TABLE 7
CRYSTAL RIVER UNIT 3
SOIL-CEMENT DESIGN MIXES
JULY 29, 1970

FIGURES

FIGURES

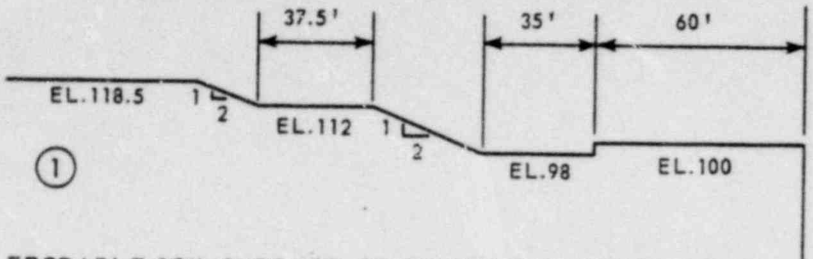




DETAIL A

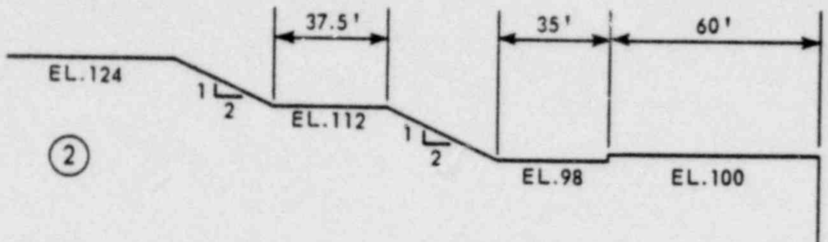
CRYSTAL RIVER UNIT 3
HURRICANE STUDY
SECTION OF MAXIMUM WAVE ATTACK
FIGURE 1

PROFILE TESTED	RESULTS OF INTEREST



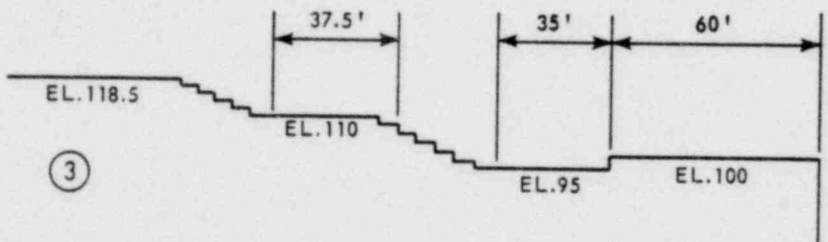
ERODABLE SOIL SURFACE, PRIOR TO HEAVY WAVE ATTACK

TIDE ** LEVEL (FT.)	MAXIMUM OVERTOPPING DEPTH (IN.)
118	88
*112	8



ERODABLE SOIL SURFACE, PRIOR TO HEAVY WAVE ATTACK

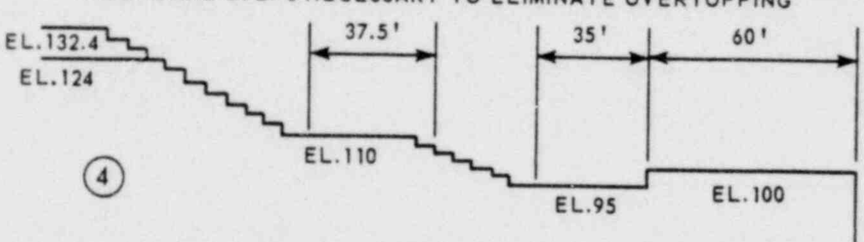
TIDE LEVEL (FT.)	MAXIMUM OVERTOPPING DEPTH (IN.)
120	40
*114	7



STEPPED SLOPES, AFTER SOIL SURFACE ERODED

TIDE LEVEL (FT.)	MAXIMUM OVERTOPPING DEPTH (IN.)
118	66
*114	6

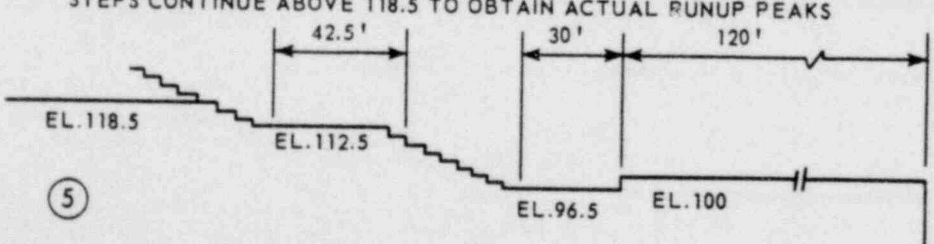
ADDITIONAL STEPS NECESSARY TO ELIMINATE OVERTOPPING



STEPPED SLOPES, AFTER SOIL SURFACE ERODED

TIDE LEVEL (FT.)	MAXIMUM OVERTOPPING DEPTH (IN.)
120	40
*118	7

STEPS CONTINUE ABOVE 118.5 TO OBTAIN ACTUAL RUNUP PEAKS



LAYOUT NO. 3, SECTION B, WITH STEPPED SLOPES

TIDE LEVEL (FT.)	ELEV. OF MAX. RUNUP (FT.)
116	122.2
114	119.8
112	117.1

** PROTOTYPE DIMENSIONS

* TIDE LEVEL AT WHICH OVERTOPPING STARTED

CRYSTAL RIVER UNIT 3

MODEL PROFILES TESTED, RESULTS OF INTEREST

FIGURE 2

SHEET 1

PROFILE 1 – SMOOTH SLOPES WITH MAXIMUM BERM ELEVATION 118.5 FT.:

NO OVERTOPPING OCCURRED FOR TIDE LEVELS RANGING FROM 104 TO 110 FT. OCCASIONAL OVERTOPPING STARTED WHEN THE TIDE LEVEL WAS 112 FT., BECOMING CONTINUOUS FOR TIDE LEVELS FROM 114 TO 118 FT. MAXIMUM OVERTOPPING DEPTH DEPTH NOTED WAS 88 INCHES.

PROFILE 2 – SMOOTH SLOPES WITH MAXIMUM BERM ELEVATION 124.0 FT.:

NO OVERTOPPING OCCURRED FOR TIDE LEVELS UP TO 112 FT. SLIGHT OVERTOPPING WAS RECORDED WITH A 114 FT. TIDE LEVEL. CONTINUOUS OVERTOPPING OCCURRED FOR TIDES OF 116 TO 120 FT., WITH A MAXIMUM DEPTH OF 40 INCHES.

PROFILE 3 – STEPPED SLOPES WITH MAXIMUM BERM ELEVATION 118.5 FT.:

NO OVERTOPPING OCCURRED FOR TIDE LEVELS FROM 104 TO 112 FT. SLIGHT OVER-TOPPING HAPPENED WITH A TIDE LEVEL OF 114 FT., BECOMING CONTINUOUS AT TIDES OF 116 AND 118 FT. MAXIMUM OVERTOPPING DEPTH RECORDED WAS 66 INCHES.

PROFILE 4 – STEPPED SLOPES WITH MAXIMUM BERM ELEVATION 124.0 FT.:

SLIGHT OVERTOPPING BEGAN WHEN THE TIDE LEVEL WAS 118 FT. AT A 120 FT. TIDE, THE MAXIMUM OVERTOPPING DEPTH WAS 40 INCHES. IN ORDER TO ELIMINATE ALL OVERTOPPING (NEGLECTING SPLASH-UP), IT WAS NECESSARY TO INCREASE THE 124 FT. ELEVATION WITH STEPS UP TO AN ELEVATION OF 132.4 FT.

PROFILE 5 – STEPPED SLOPES, SLIGHTLY DIFFERENT DESIGN THAN PREVIOUSLY TESTED (LABELED LAYOUT NO. 3, SECTION B BY GAI):

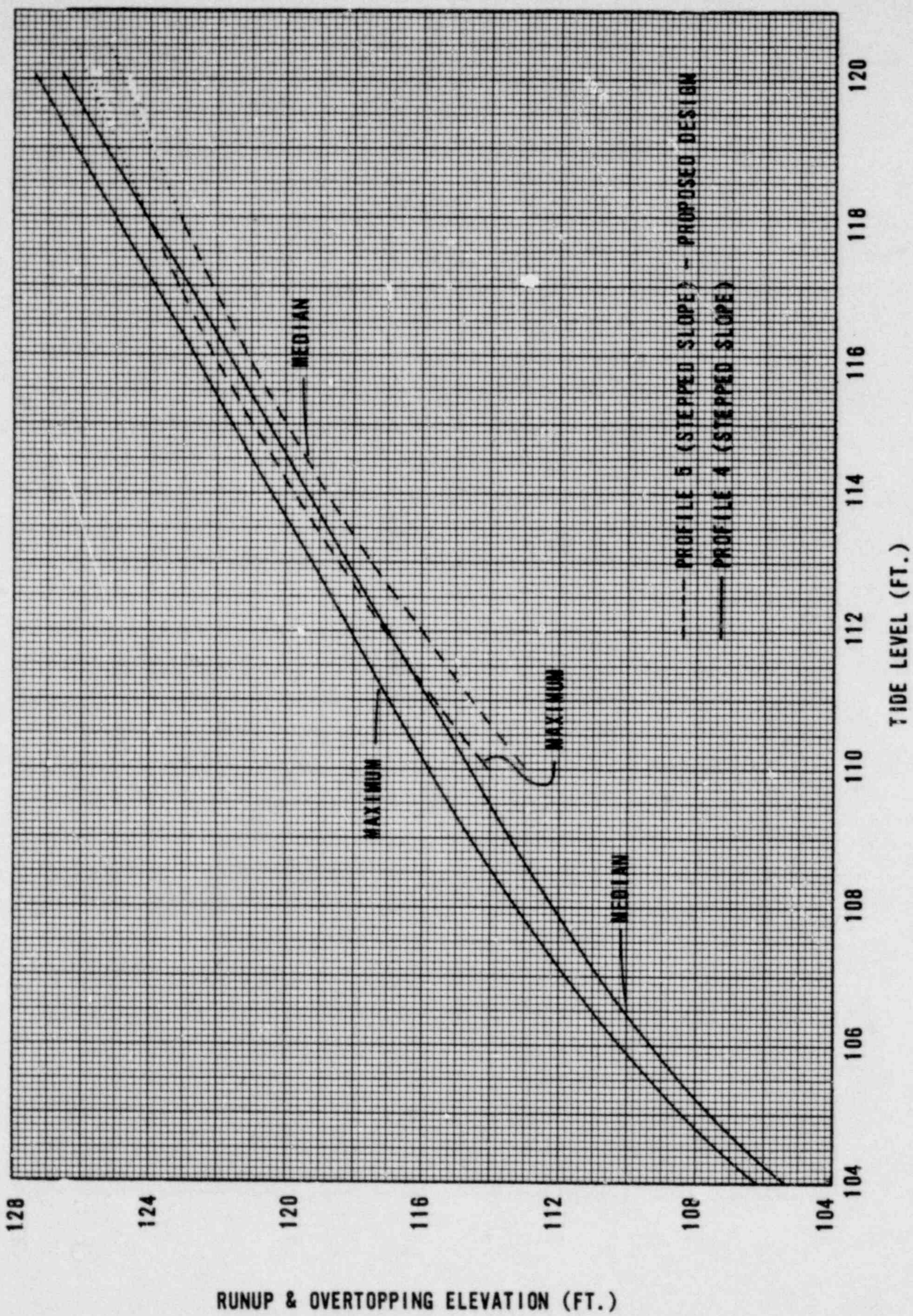
STEPS WERE CONTINUED ABOVE THE BASE ELEVATION OF 118.5 FT. IN ORDER TO PREVENT OVERTOPPING FOR THIS SERIES OF TESTS. MAXIMUM RUNUP PEAKS WERE SLIGHTLY LESS FOR THIS PROFILE THAN FOR OTHERS TESTED.

CRYSTAL RIVER UNIT 3

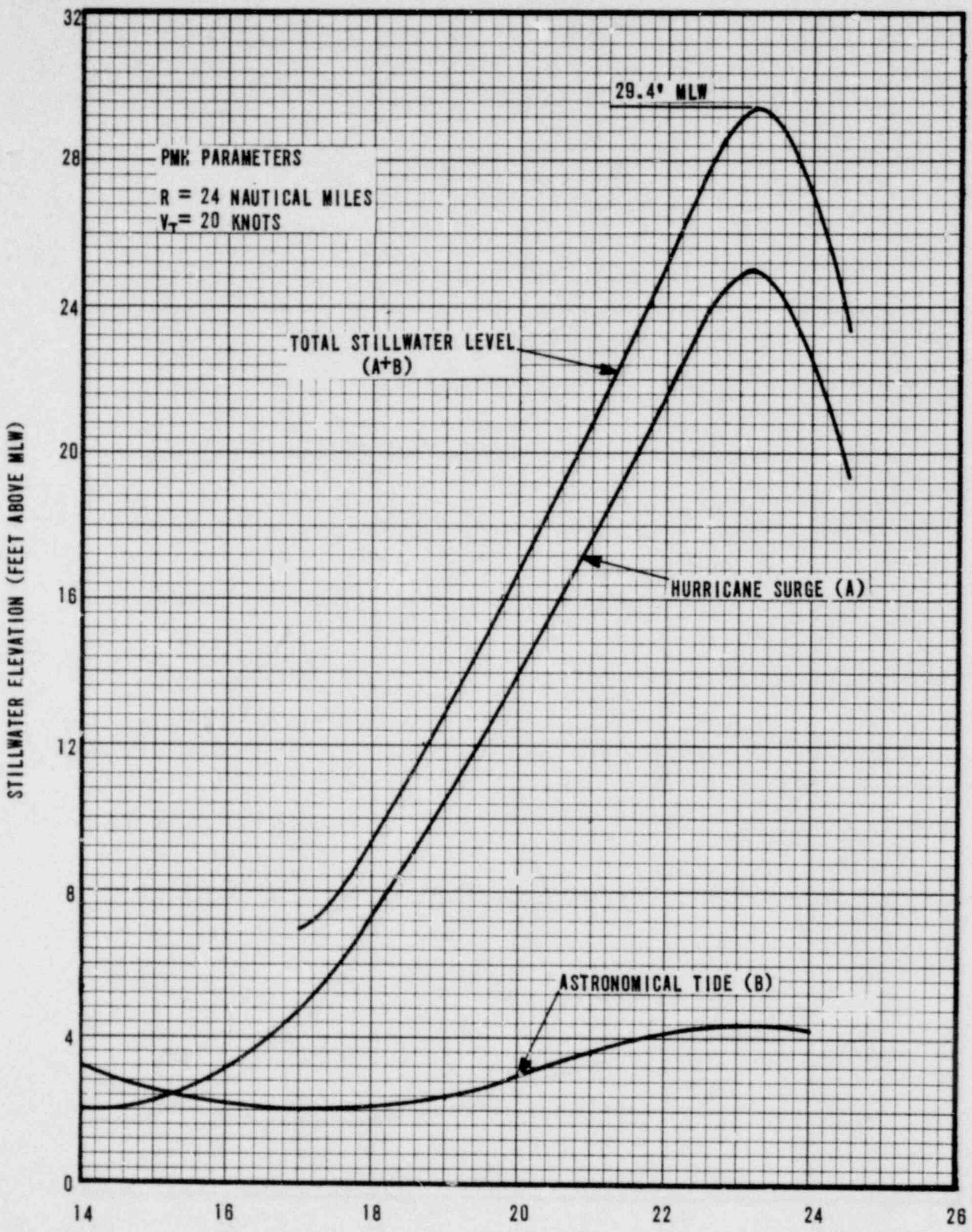
RESULTS OF MODEL TESTS

FIGURE 2

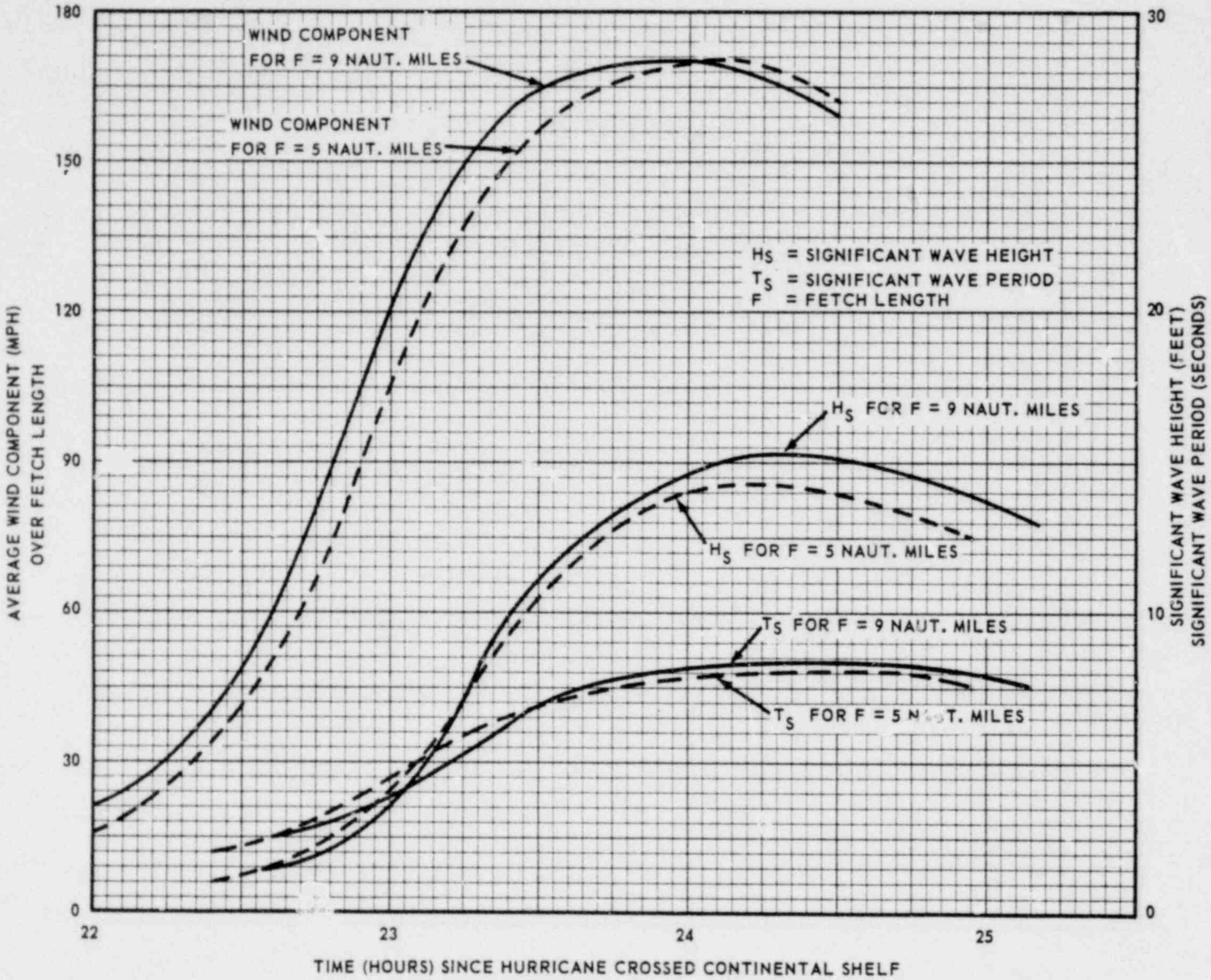
SHEET 2



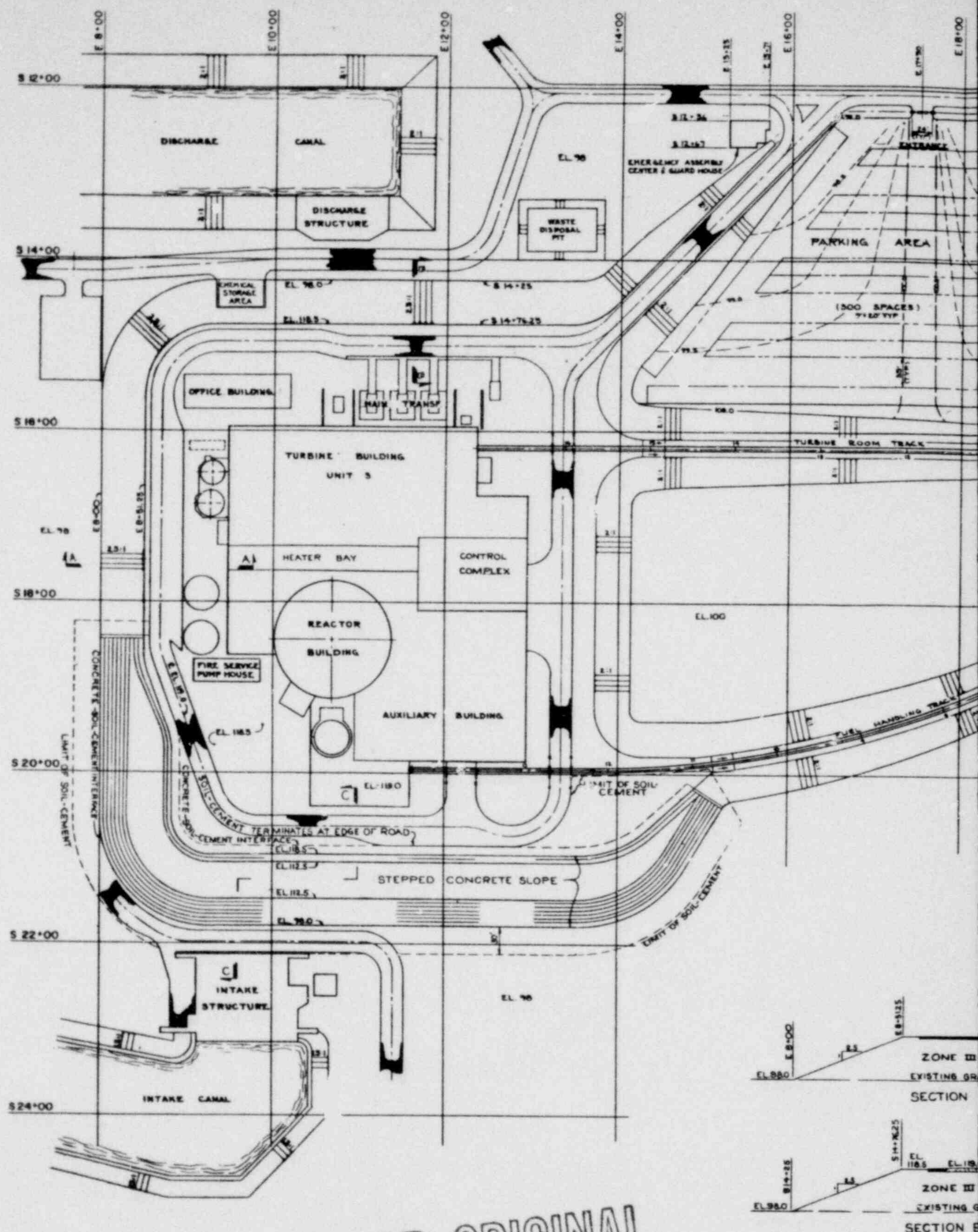
CRYSTAL RIVER UNIT 3
RUNUP AND OVERTOPPING
VS
TIDE LEVEL
FIGURE 3



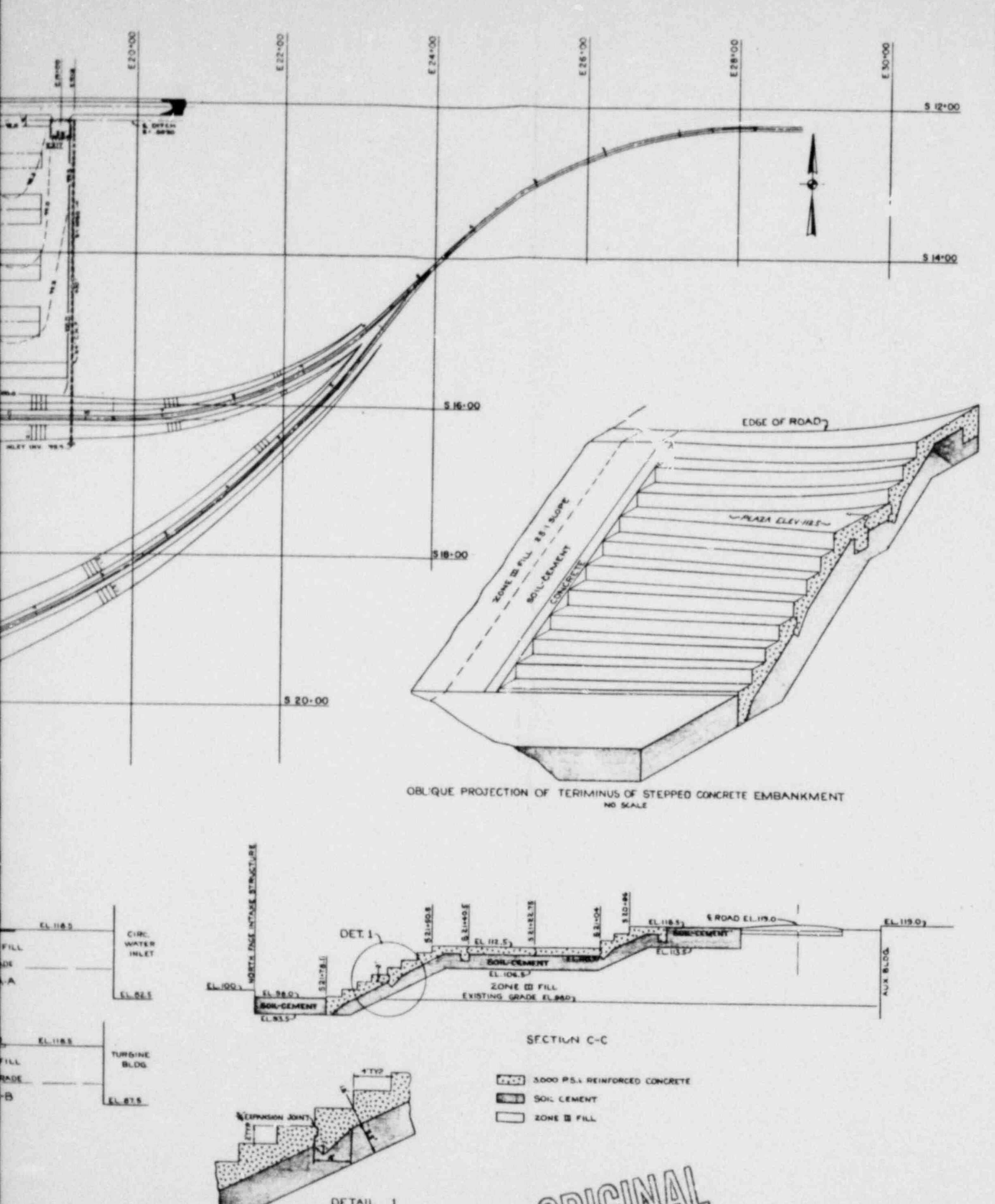
CRYSTAL RIVER UNIT 3
STORM SURGE HYDROGRAPH, PMH
FIGURE 4



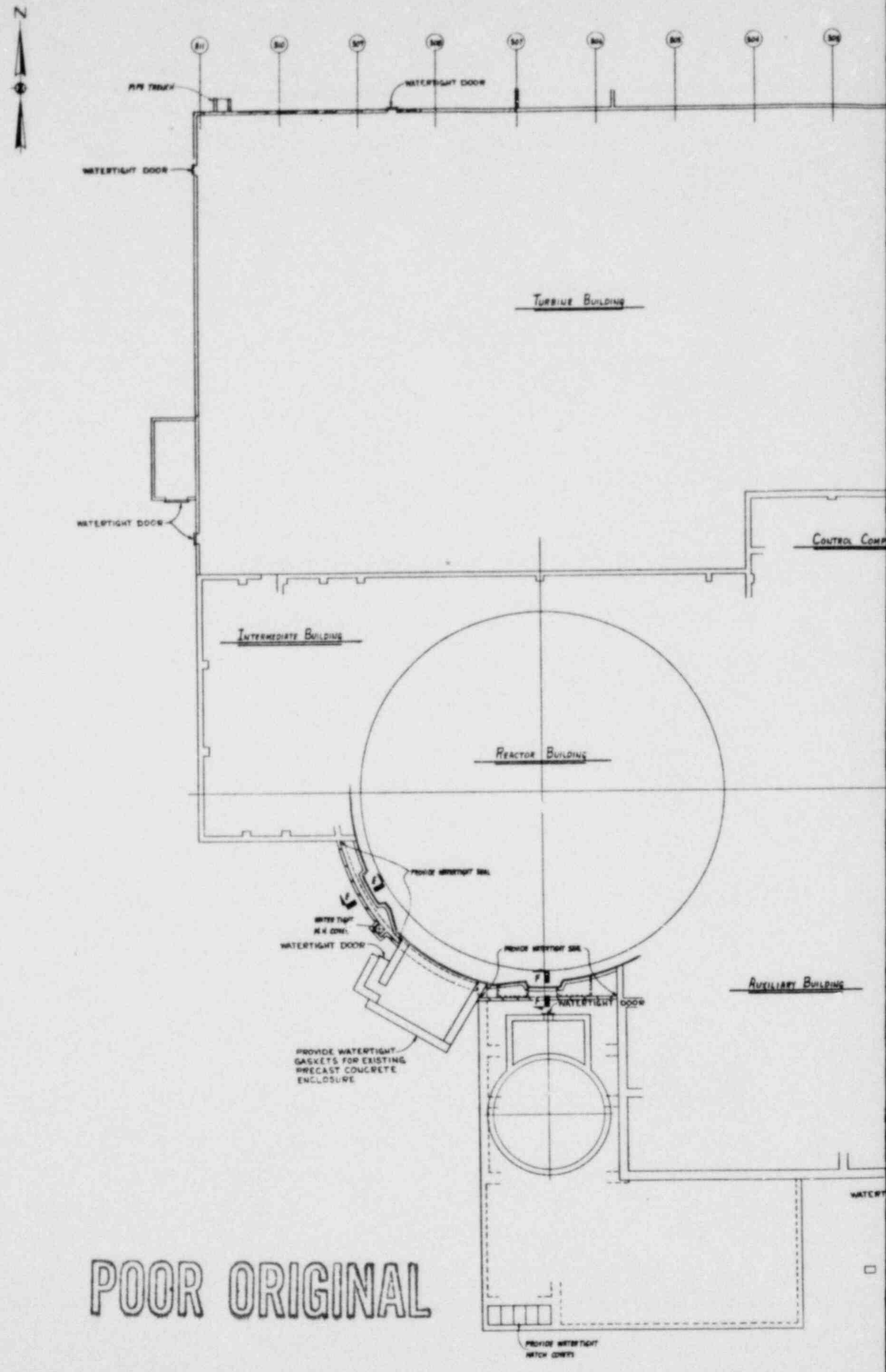
CRYSTAL RIVER UNIT 3
WIND AND WAVE CHARACTERISTICS VS. TIME
FIGURE 5

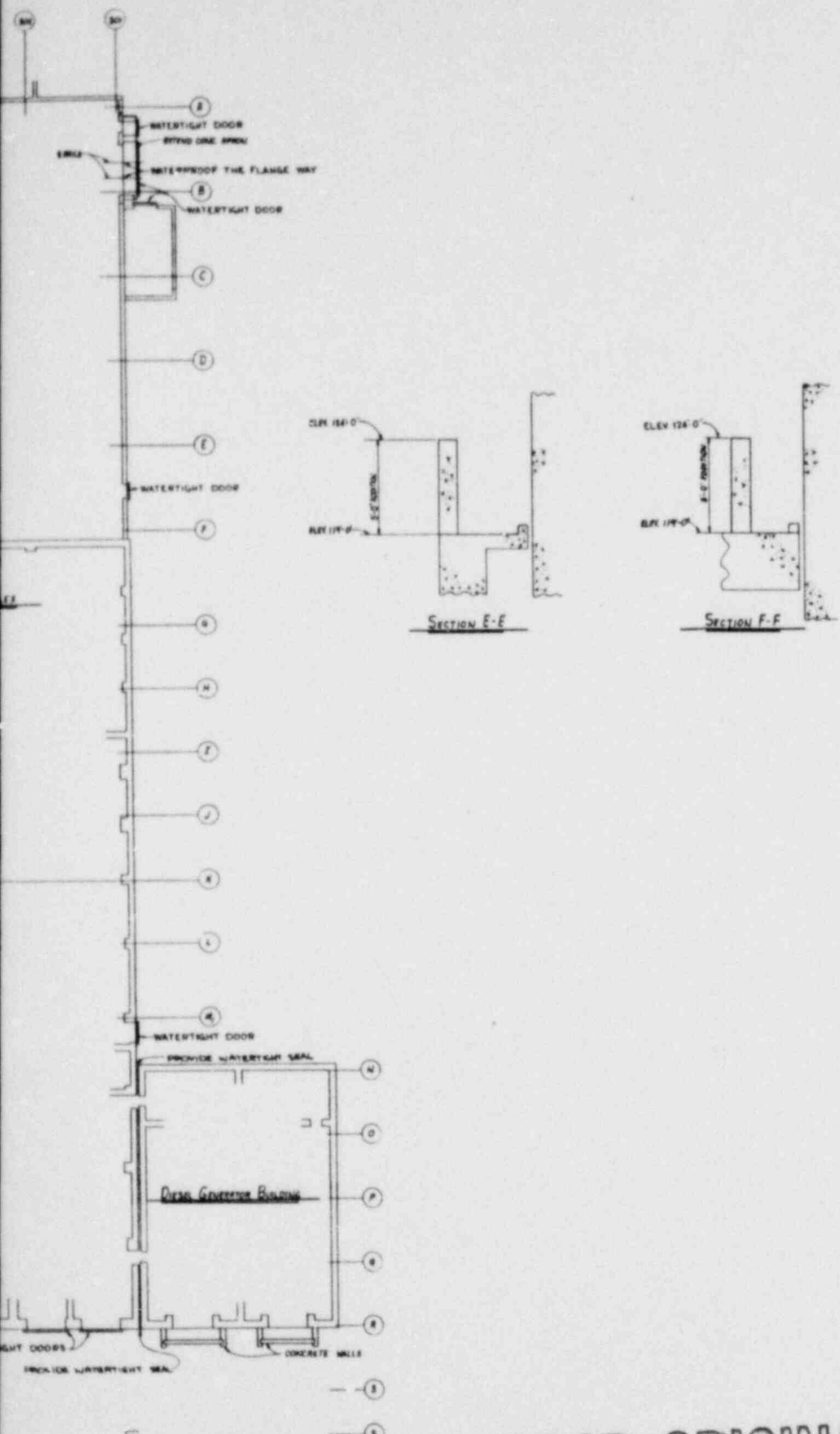


POOR ORIGINAL



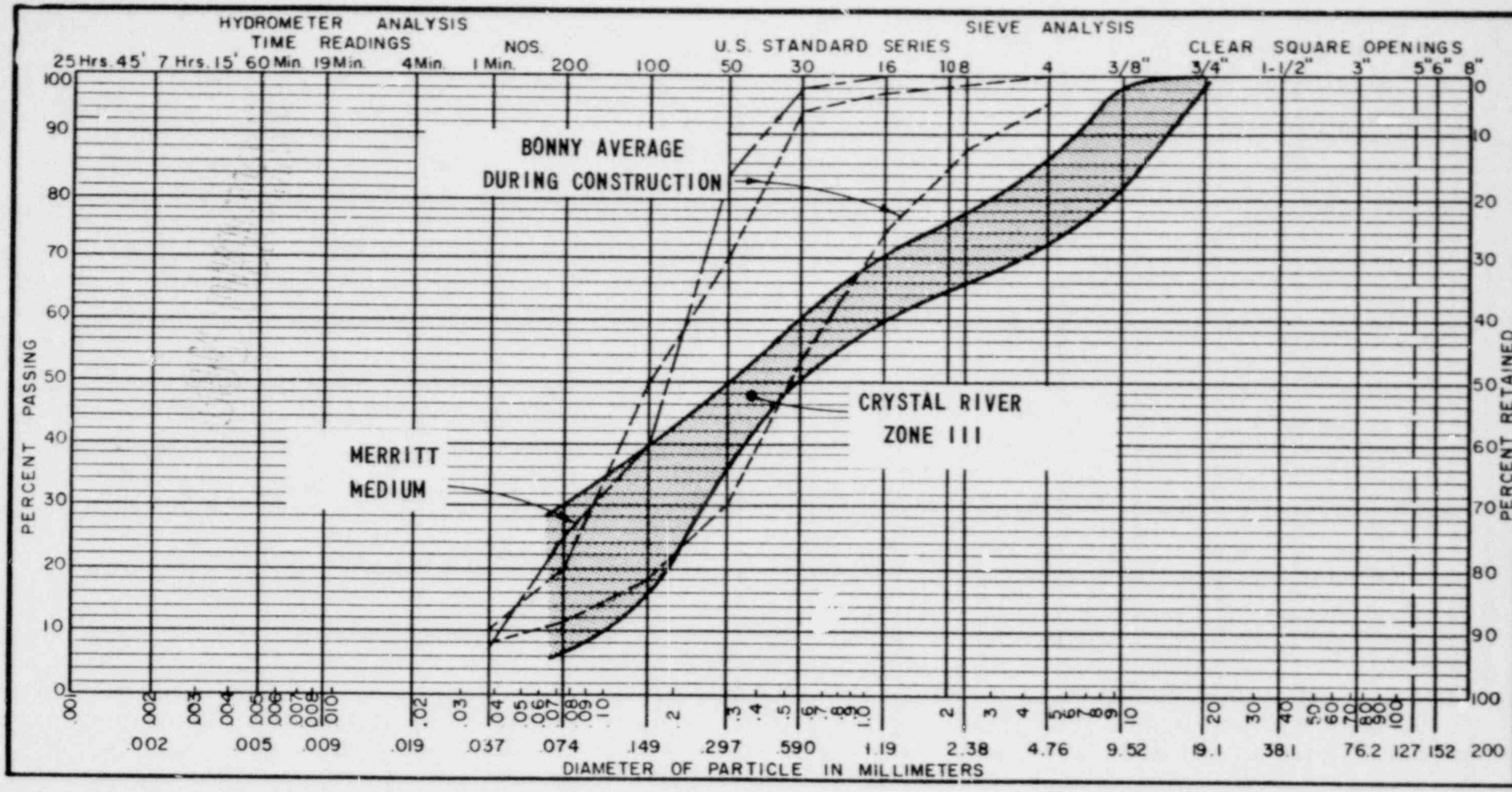
CRYSTAL RIVER UNIT 3
PLOT PLAN
FIGURE 7


N
PIPE TRENCH
WATERTIGHT DOOR
WATERTIGHT DOOR
WATERTIGHT DOOR
Turbine Building
Control Comp
Intermediate Building
Reactor Building
Auxiliary Building
PROVIDE WATERTIGHT SEAL
WATERTIGHT DOOR
PROVIDE WATERTIGHT SEAL
WATERTIGHT DOOR
PROVIDE WATERTIGHT GASKETS FOR EXISTING PRECAST CONCRETE ENCLOSURE
PROVIDE WATERTIGHT HATCH COVERS

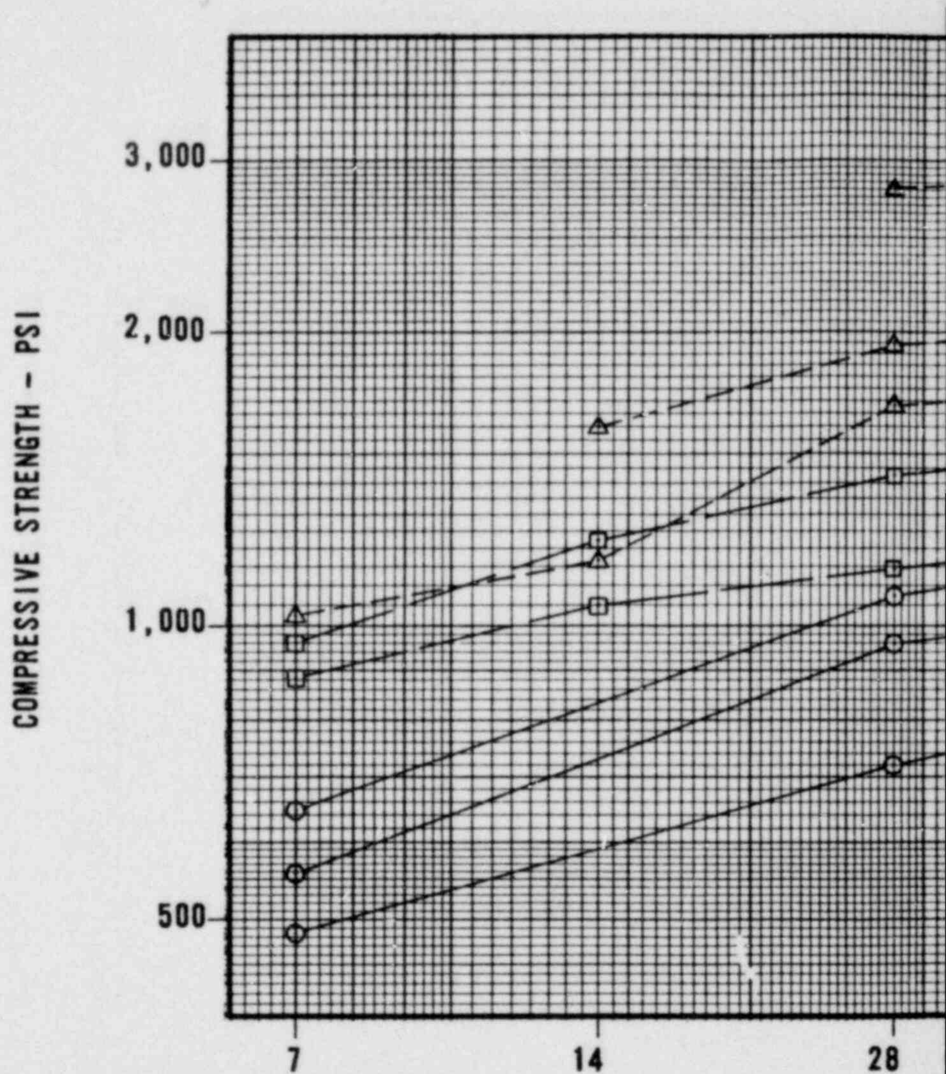


POOR ORIGINAL

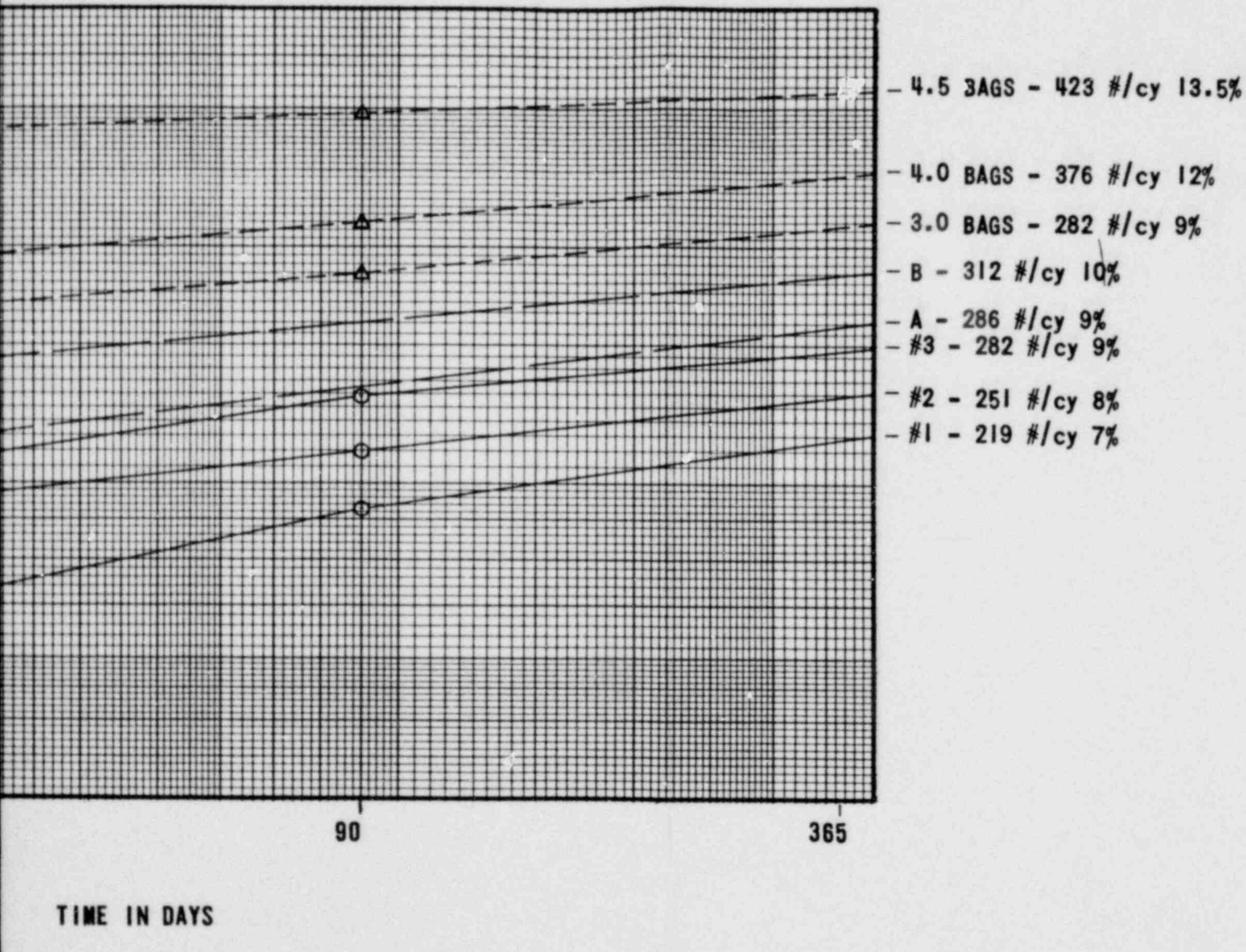
CRYSTAL RIVER UNIT 3
PLAN – WATER SEALS
WATER-TIGHT DOOR LOCATIONS
FIGURE 8



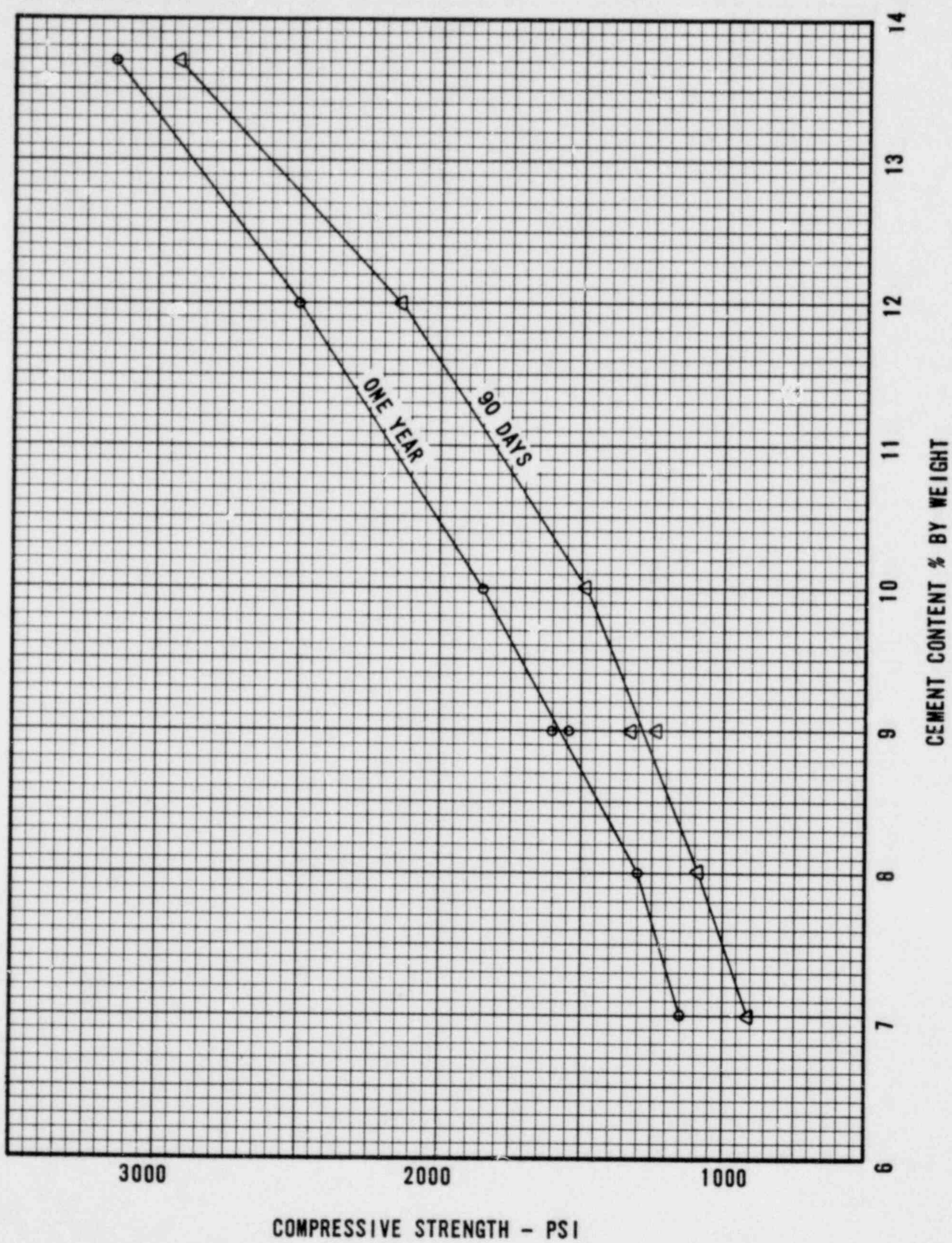
CRYSTAL RIVER UNIT 3
SOIL – CEMENT MIXES
SOIL GRADATION CURVES
FIGURE 9



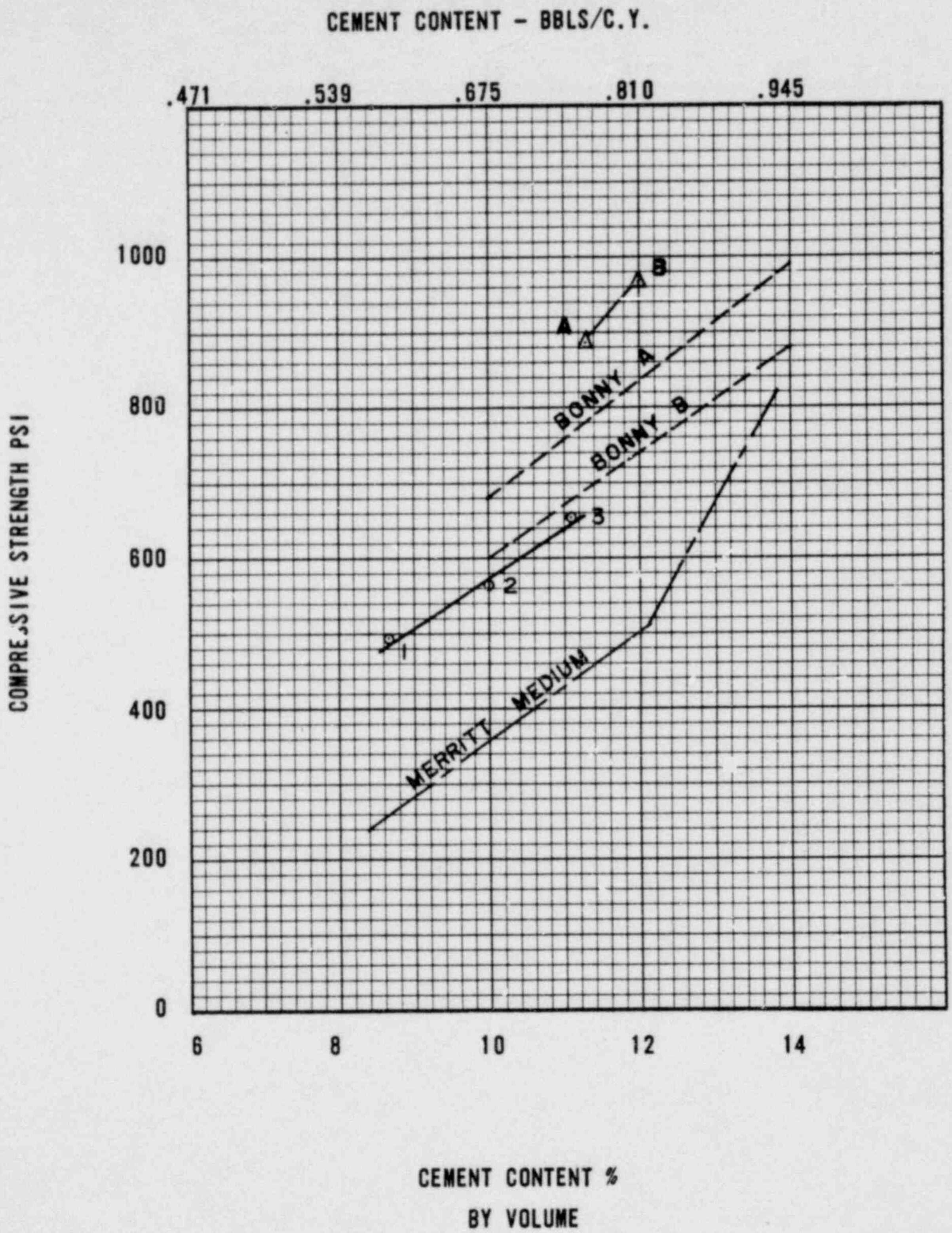
COMPRESSIVE STRENGTH - PSI
3,000
2,000
1,000
500
7
14
28



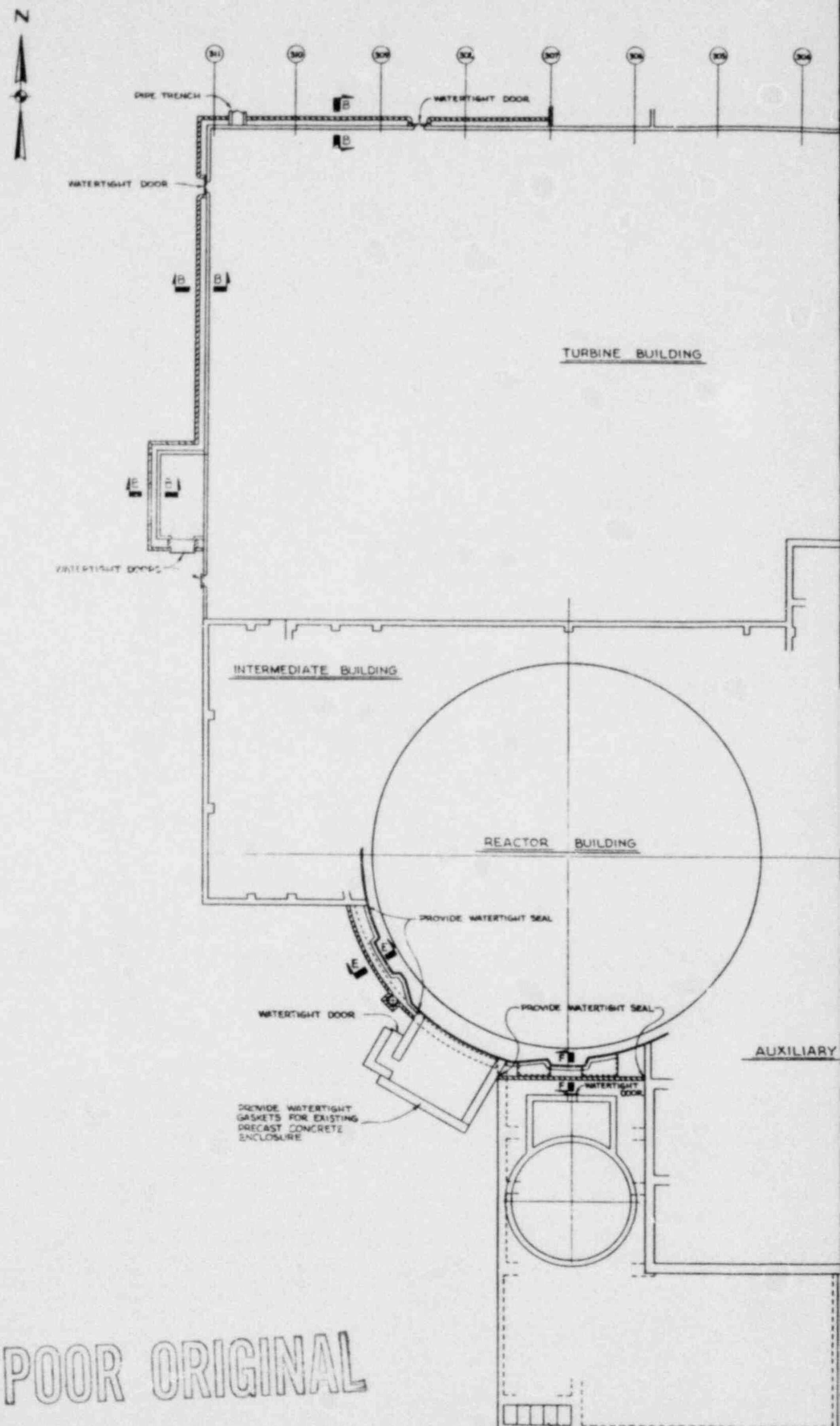
CRYSTAL RIVER UNIT 3
SOIL – CEMENT DESIGN MIXES
FIGURE 10



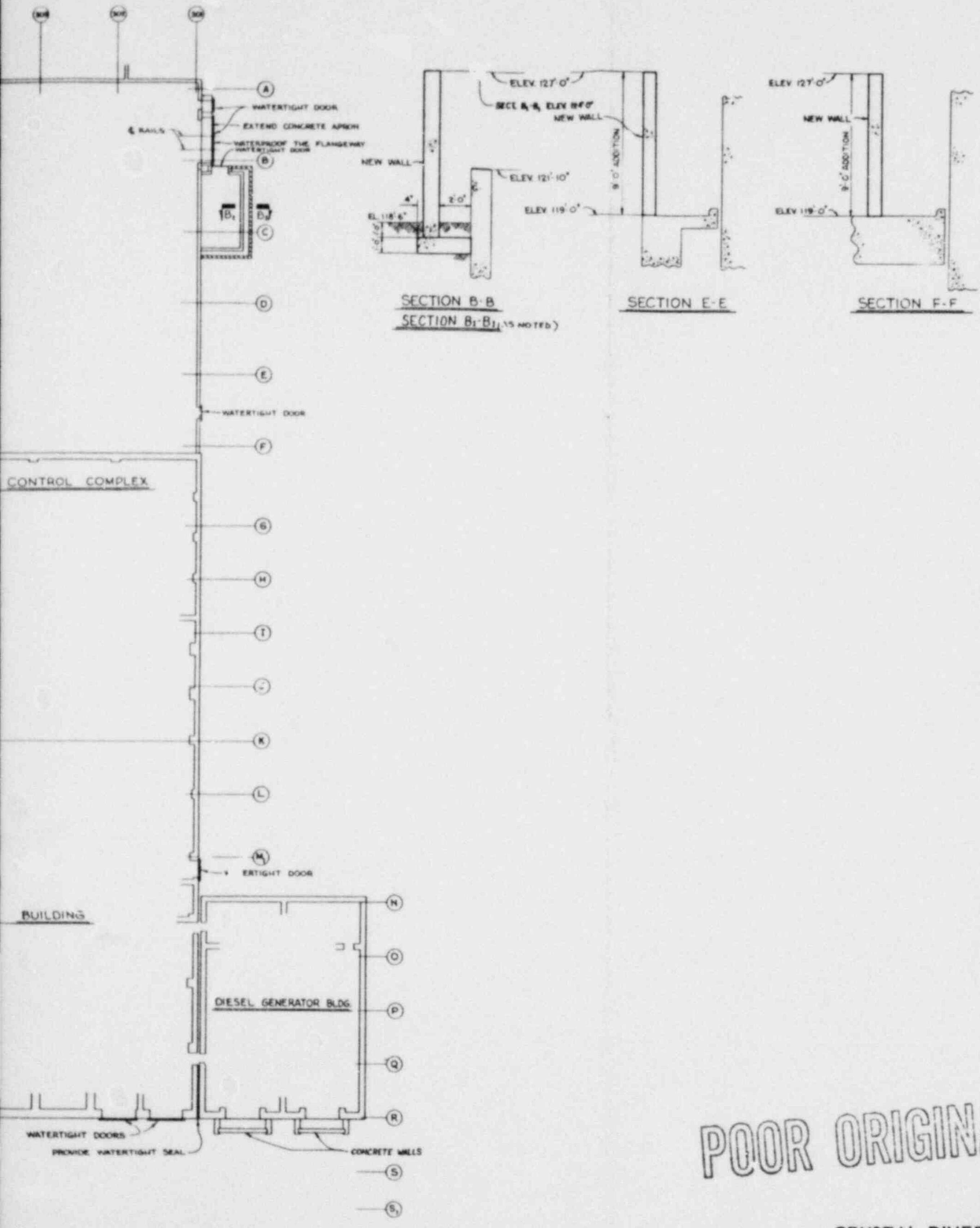
CRYSTAL RIVER UNIT 3
SOIL - CEMENT DESIGN MIXES
FIGURE 11



CRYSTAL RIVER UNIT 3
SOIL - CEMENT MIXES
7 DAY COMPRESSIVE STRENGTH
FIGURE 12



N
PIPE TRENCH
WATERTIGHT DOOR
WATERTIGHT DOOR
WATERTIGHT DOORS
TURBINE BUILDING
INTERMEDIATE BUILDING
REACTOR BUILDING
PROVIDE WATERTIGHT SEAL
WATERTIGHT DOOR
PROVIDE WATERTIGHT SEAL
AUXILIARY
WATERTIGHT DOOR
PROVIDE WATERTIGHT GASKETS FOR EXISTING PRECAST CONCRETE ENCLOSURE
PROVIDE WATERTIGHT HATCH COVERS



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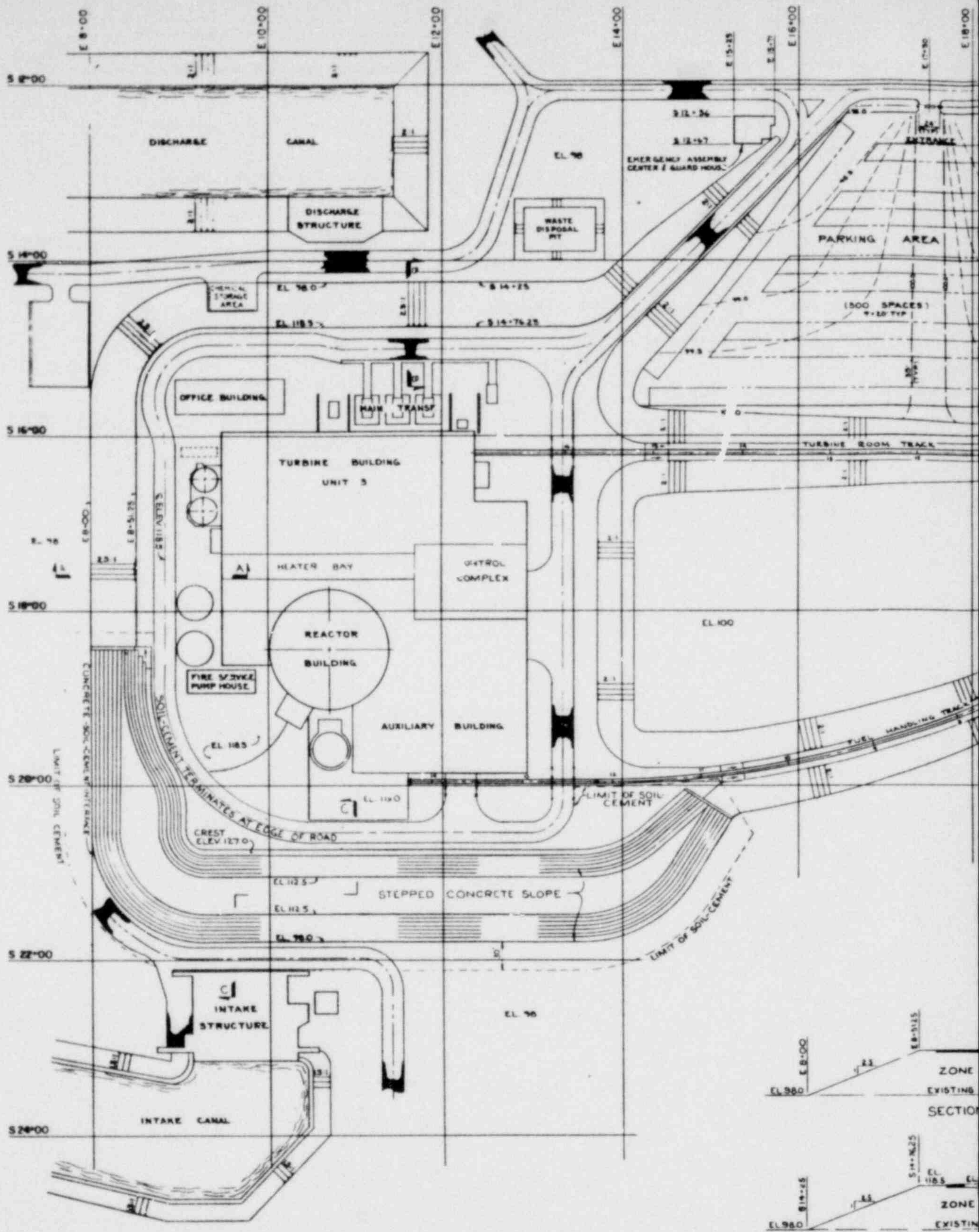
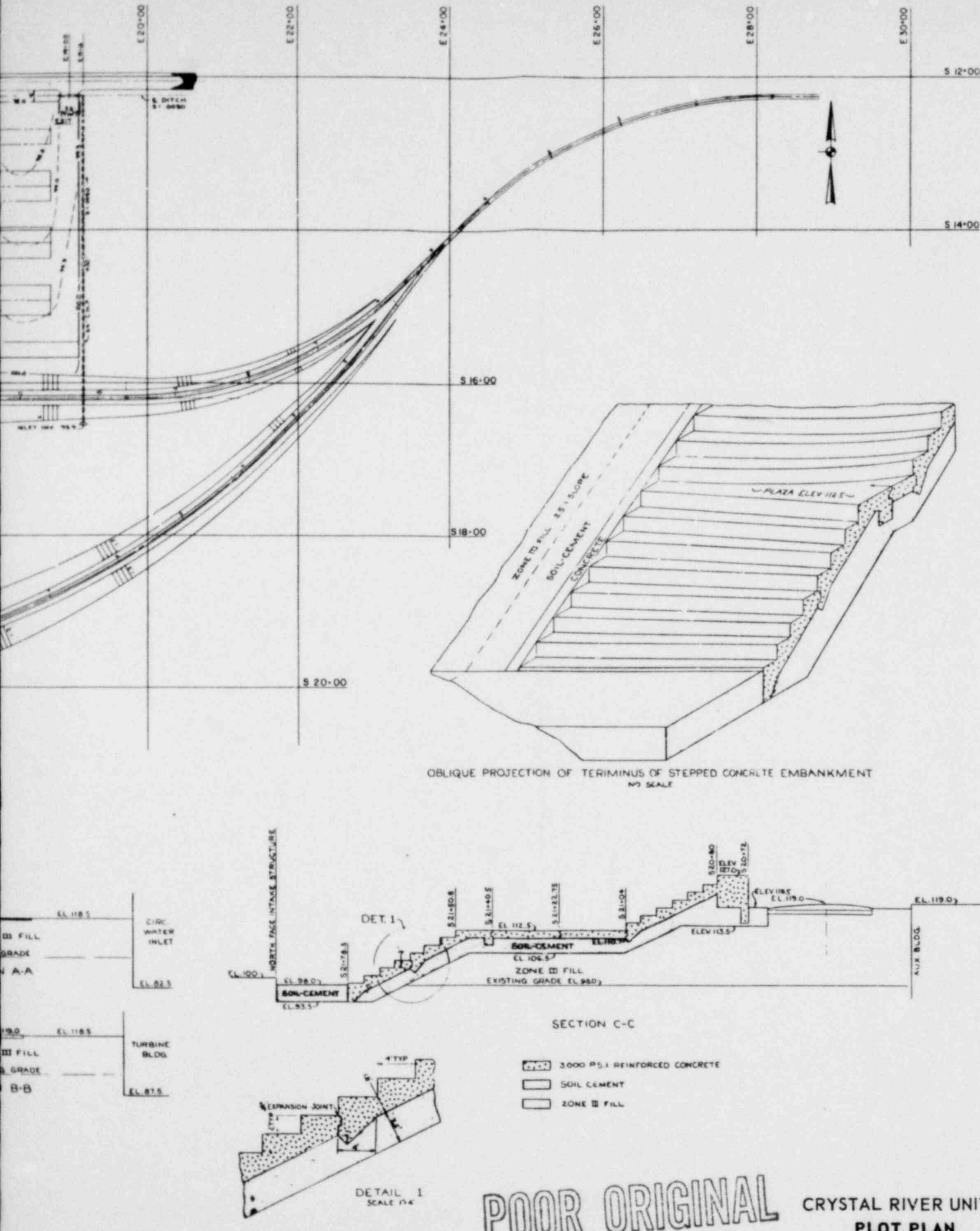
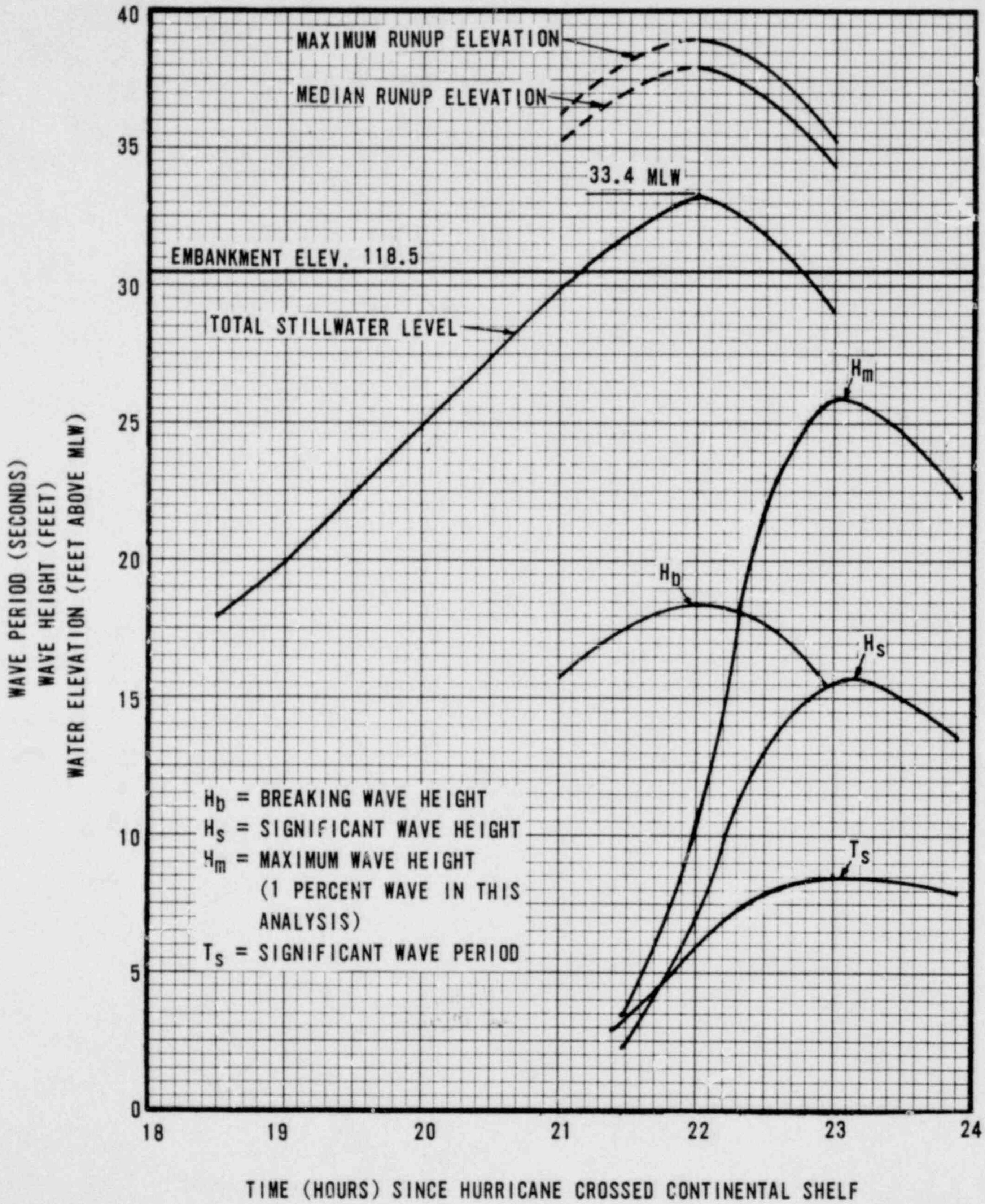
CRYSTAL RIVER UNIT 3
PLAN – WATER SEALS
WATER-TIGHT DOOR LOCATIONS
FIGURE 13


E 8+00
E 10+00
E 12+00
E 14+00
E 16+00
E 18+00
S 12+00
S 14+00
S 16+00
S 18+00
S 20+00
S 22+00
S 24+00
DISCHARGE CANAL
DISCHARGE STRUCTURE
WASTE DISPOSAL PIT
EMERGENCY ASSEMBLY CENTER & GUARD HOUSE
ENTRANCE
PARKING AREA
(500 SPACES)
CHEMICAL STORAGE AREA
OFFICE BUILDING
MAIN TRANSF
TURBINE BUILDING UNIT 3
HEATER BAY
CONTROL COMPLEX
REACTOR BUILDING
FIRE SERVICE PUMP HOUSE
AUXILIARY BUILDING
TURBINE ROOM TRACK
FUEL HANDLING TRACK
EL. 100
EL. 98
SOIL CEMENT TERMINATES AT EDGE OF ROAD
CREST ELEV 127.0
LIMIT OF SOIL CEMENT
STEPPED CONCRETE SLOPE
INTAKE STRUCTURE
INTAKE CANAL
ZONE
EXISTING
SECTION



CRYSTAL RIVER UNIT 3
PLOT PLAN
FIGURE 14



CRYSTAL RIVER UNIT 3

DESIGN WAVES AND WATER LEVELS VS TIME

FIGURE 15

ATTACHMENTS

FLORIDA POWER CORP.

CRYSTAL RIVER UNIT 3

SOIL-CEMENT TESTS

OCTOBER, 1969

MIX A - 286 #/CY
MIX B - 312 #/CY

Bob McKnight ___ 2/9/70

PROPOSED METHODS FOR SOIL CEMENT TESTING

It was felt that "PSAR" strength of 2000 psi compressive strength was not entirely applicable mix design criteria considering the berm usage for wave protection. Most soil cement mixtures which will satisfactorily meet the requirements of ASTM D559 and D560* will be found to have adequate compressive strength for use as a berm or subbase materia

Our suggestion is to design the mix using the durability when exposed to wetting-drying test (ASTM D559) as the controlling criteria. This should give at least 500 psi compressive strength when tested in accordance with ASTM D1633 We feel the above method will give a more than adequate berm.

Rather than use a 6x12 mold & ASTM D1632, we would use 4" split compaction mold and laboratory compaction effort of 22,350 ft-lb/ft³ (same as PSAR requirement). For more detail see PTL report dated Oct. 28, 1969, regarding method of molding specimens.

E. Froats

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* not applicable for this climate.

Soil Cement Design

Materials

Soil was native excavated material consisting of weathered limerock and infilled material sampled at random from the fill area to the southeast of unit 3 and from the banks of drainage canals. Approx. 500 lbs. of total material was sampled at 6 different points and a composite made up. No attempt was made to grade selectively other than to screen material through a 3/4" screen for purposes of laboratory testing. Gradations on the material used were run and are attached.

Cement used was Type II, Moderate heat obtained at West Coast Concrete Batch Plant.

Compaction Methods

Three different laboratory compaction methods were tried, all based on the recommended effort in the PSAR of 22,350 ft/lb./c.f

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A curve showing the results and total % spread between maximum densities obtained is attached. The deviation of any results, at the maximum one from the other, is within allowable field compaction limitations. Method "C" as shown on the curve was chosen because it would permit utilizing a 4" split compaction mold for molding each compression specimen with the same dynamic ~~[illegible]~~ compactive effort. This eliminates one of the biggest problems with the ASTM D-1632 method, that of varying densities of specimens. A list of densities and moistures on each specimen molded is attached to this report. It is necessary to use a 4" mold in lieu of a 6" mold because ASTM C-42 only provides correction factors for compression specimens with L/D ratios greater than 1:1. Our specimen ration for these tests is 1.15 and a correction factor of 0.925 has been applied to all compression test strength results.

Design

Two trial mixes were made,

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Mixes A & B. Mix A employs 286# of cement per cubic yard of compacted soil at maximum density and a moisture of optimum + 1%-2% Mix B has 312# of cement per cubic yard of soil. Results of both strength (compressive) and splitting tension tests are attached.

It should be pointed out that the compressive strength of this material at 28 days would seem to more than adequately meet the requirements while attached comparison between 1500 PSI fill concrete and the soil cement mixture in splitting tension indicates at least equal if not greater strength in shear for the soil cement. This material, of course, is much cheaper to use as structural fill, than is the 1500 PSI concrete.

W F Hurst

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(For Intra Office Use Only)

[Cli]ent's [Or]der No. ____________ Date 10-28-69 PTL Order No. TA7732

[C]lient F.P.C. - CRYSTAL RIVER, FLA. - UNIT #3

[D]escription of Inspection GRADATION OF ON SITE EXCAVATED

At MATERIAL USED IN SOIL CEMENT MIX DESIGN

% PASSING

SIZE	SAMP #1	SAMP #2	
1½"			
1"			
¾"	100.0	100.0	
½"	98.8	99.9	
⅜"	96.3	95.5	
#4	87.0	86.2	
#8	79.4	77.8	
#16	71.2	70.0	
#30	61.8	60.0	
#50	46.5	44.0	
#100	23.0	18.9	
#200	2.7	8.5	
F.M.			

THIS MATERIAL DOES, DOES NOT MEET SPECIFICATIONS

Time: ____________ hours

Mileage: ____________ Miles

Other Expenses $ ____________

W F Hurst
Inspector

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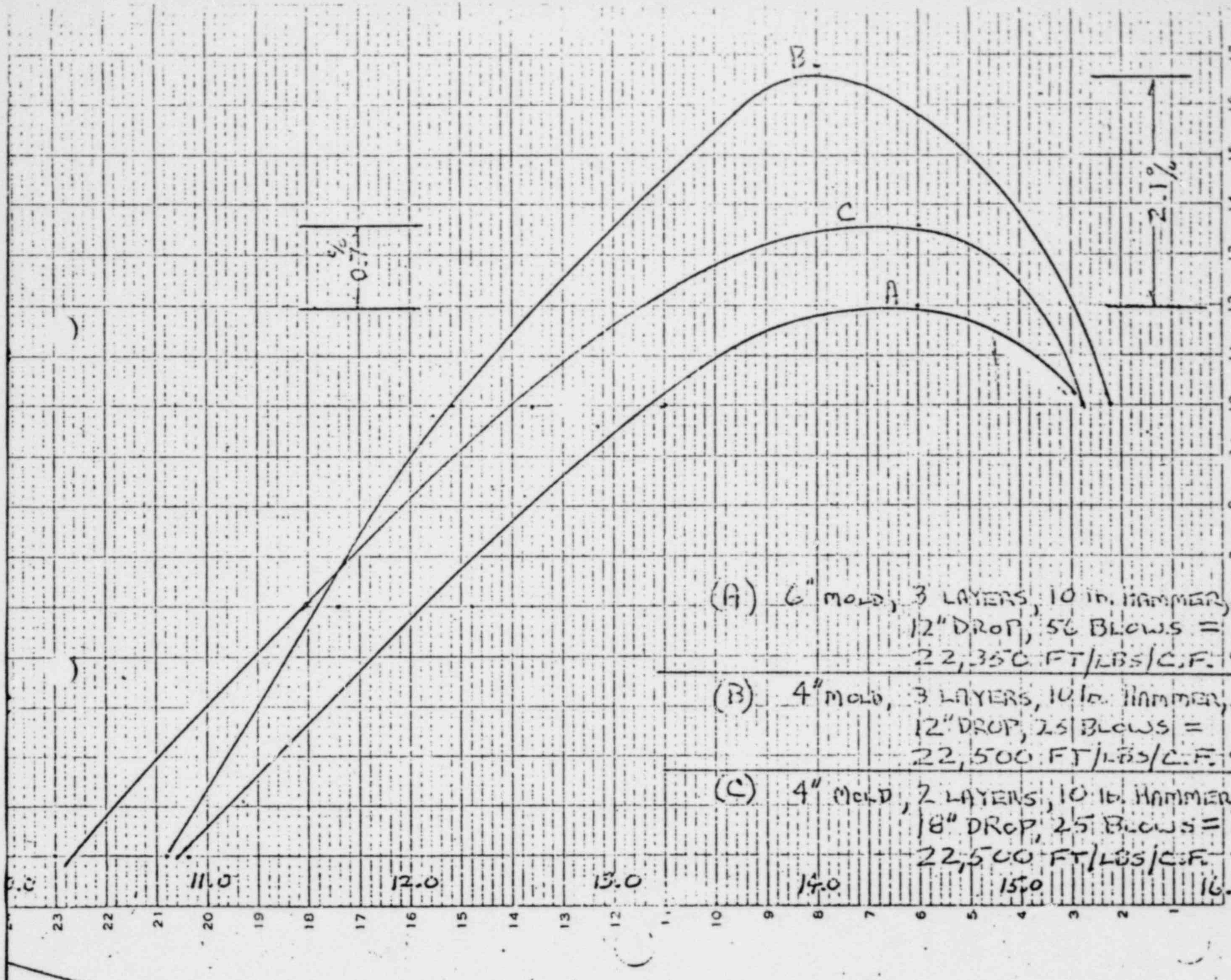
B
C
A
0.7%
2.1%
(A) 6" MOLD, 3 LAYERS, 10 lb. HAMMER, 12" DROP, 56 BLOWS = 22,350 FT/LBS/C.F.
(B) 4" MOLD, 3 LAYERS, 10 lb. HAMMER, 12" DROP, 25 BLOWS = 22,500 FT/LBS/C.F.
(C) 4" MOLD, 2 LAYERS, 10 lb. HAMMER, 18" DROP, 25 BLOWS = 22,500 FT/LBS/C.F.
11.0
12.0
13.0
14.0
15.0
16.0

(For Intra Office Use Only)

No.: ____________ Date 10-28-69 PTL Order No. ____________

FLA. POWER CORP. - UNIT #3 - CRYSTAL RIVER, FLA.

Description of Inspection MAX. DENSITY AND OPTIMUM MOISTURE CONTENT OF SOIL CEMENT TRIAL MIXES.

DESIGN "A"		MAX. DRY DENSITY	OPTIMUM MOISTURE
7 DAY SPECIMENS:	1-	113.1 LBS/CU.FT.	14.9 %
	2-	112.6 "	14.5 %
	3-	112.6 "	14.5 %
14 DAY SPECIMENS	1-	113.3 LBS/CU.FT.	14.9 %
	2-	113.3 "	14.9 %
	3-	113.5	14.7 %

DESIGN "B"			
7 DAY SPECIMENS	1-	113.0 LBS/CU.FT.	14.7 %
	2-	113.1 "	14.7 %
	3-	112.8 "	14.8 %
14 DAY SPECIMENS	1-	111.1 LBS/CU.FT.	14.8 %
	2-	110.9 "	15.2 %
	3-	111.4 "	15.6 %

Time: ____________ hours

Mileage: ____________ Miles

Other Expenses $ ____________

W. J. [illegible]

Inspector

POOR ORIGINAL

(For Intra Office Use Only)

...'s No. ______ Date 11-13-69 ______ PTL Order No. ______

...ent F.P.C. - CRYSTAL RIVER

...scription of Inspection COMPARISON OF STRENGTHS OF SOIL CEMENT & 1500 PSI CONCRETE IN SPLITTING TENSION IN ACCORDANCE WITH ASTM C-496.

SPECIMEN	AGE (DAYS)	SIZE	STRENGTH (PSI)
1500 PSI CONC.	7	6" X 12"	156
SOIL CEMENT (MIX B)	14	4" X 4.6"	216
1500 PSI CONC.	7	6" X 12"	175
SOIL CEMENT (MIX A)	14	4" X 4.6"	185
MIX B	28		235
MIX A	28		210

Time: ______ hours

Mileage: ______ Miles

Other Expenses $ ______

W. P. Hurst
Inspector

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Order No. TA-7732

Report No. ________

Date 11-11-69

REPORT

REPORT OF TEST ON CONCRETE CYLINDERS
6" DIAMETER BY 12" LENGTH

REPORTED TO: FLORIDA POWER CORP.
PROJECT: CRYSTAL RIVER PLANT UNIT NO. 3
Concrete Supplier: West Coast Concrete, Inc.
Arch-Engineer: Gilbert Assoc., Inc.
General Contractor: J. A. Jones

Location of Concrete Placement SOIL CEMENT

Date Cast 10-28-69 Concrete Class ______ P.S.I. Area 28.27 Sq. In.

Cement Factor ______ Brand ______ Type ______ Aggregate Size ______

DEL. S/N.	CYLINDER IDENT.	W/C GAL.-SACK	SLUMP INCHES	AIR %	CONCRETE TEMP. °F.	DATE TESTED	AGE DAYS	TOTAL LOAD LBS.	COMP. ST. P.S.I.	SPECIFIC LOCATION
	Mix A					11-4	7		865	
	"					"	7		955	
	"					"	7		845	
	"					11-11	14		1070	
	"					"	14		1030	
	"					"	14		1040	
	Mix B					11-4	7		885	
	"					"	7		1040	
	"					"	7		960	
	"					11-11	14		1250	
	"					"	14		1170	
	"					"	14		1230	
	Mix A						28		1150	
	"						"		1095	
	"						"		1180	
	Mix B						"		1420	
	"						"		1360	
	"						"		1475	

REMARKS:

1 – Florida Power Corp., H. C. Bennett
2 – Florida Power Corp., E. E. Froats
– PTL – Tampa
– PTL – W. T. Hurst
1 – PTL – Pgh.

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PITTSBURGH TESTING LABORATORY

W. T. Hurst

FLORIDA POWER CORP.

CRYSTAL RIVER UNIT 3

SOIL-CEMENT TESTS

NOVEMBER, 1969

MIX 1 - 3 Bags Cement/CY
MIX 2 - 4 Bags Cement/CY
MIX 3 - 4.5 Bags Cement/CY

FLORIDA TESTING LABORATORIES, INC.

Phone 531-1446 — P. O. Box 11064
St. Petersburg, Florida 33733

Client Florida Power Corporation
Submitted By FPC
Project Crystal River, Unit #3
Sampled By FPC
Material Soil Cement, Specimen 6"x12"
Sampled From On Site material
Identification Marks As Shown
Date Sampled
Quantity Represented
Date Received
Contractor
Date Tested
Source of Supply Native Limerock on Site
Date Reported

Lab. No. 3425

TEST RESULTS

Tested By

3.0 Bags/cu.Yd.			4.0 bags/cu. Yd.			4.5 Bag/Cu. Yd		
Molded Weight	Moisture	Psi @ 7 day	Molded Weight	Moisture	Psi @ 7 day	Molded Weight	Moisture	Psi 7 Da
118.1	11.7	972	121.0	11.1	2369	-	-	-
118.1	11.2	1007	120.5	11.9	1786	-	-	-
116.8	10.7	1098	119.5	11.5	1821	-	-	-
		Psi @ 14 days			Psi @ 14 days			Psi @ 14 da
116.0	11.1	1131	120.6	11.6	1697	-	-	-
117.2	10.7	1202	119.4	11.9	1379	-	-	-
116.2	11.0	843	119.7	11.9	1732	-	-	-
		Psi @ 28 days			Psi @ 28 days			Psi @ 28 da
116.2	12.3	990	120.7	11.2	2758	121.0	10.7	3129
118.9	10.3	1520	120.3	11.1	1962	119.8	10.9	2440
118.6	10.6	1856	120.1	11.0	1909	119.4	10.1	2829
		Psi @ 90 days			Psi @ 90 days			Psi @ 90 da
118.8	10.7	1839	121.8	11.0	2298	120.2	11.2	3041
118.8	10.6	1927	119.3	11.7	2033	120.1	11.2	2846
118.2	11.1	1786	119.9	11.5	2122	120.0	11.2	2988

By Norris A. French
FLORIDA TESTING LABORATORIES, INC.

cc: FPC/Edw Froats (3)

FORM FTL-4 REV.

FLORIDA TESTING LABORATORIES, INC.
ST. PETERSBURG, FLORIDA

SOIL COMPACTION CURVE SHEET

Laboratory No.: 3425 Tested By: ______ Date: ______

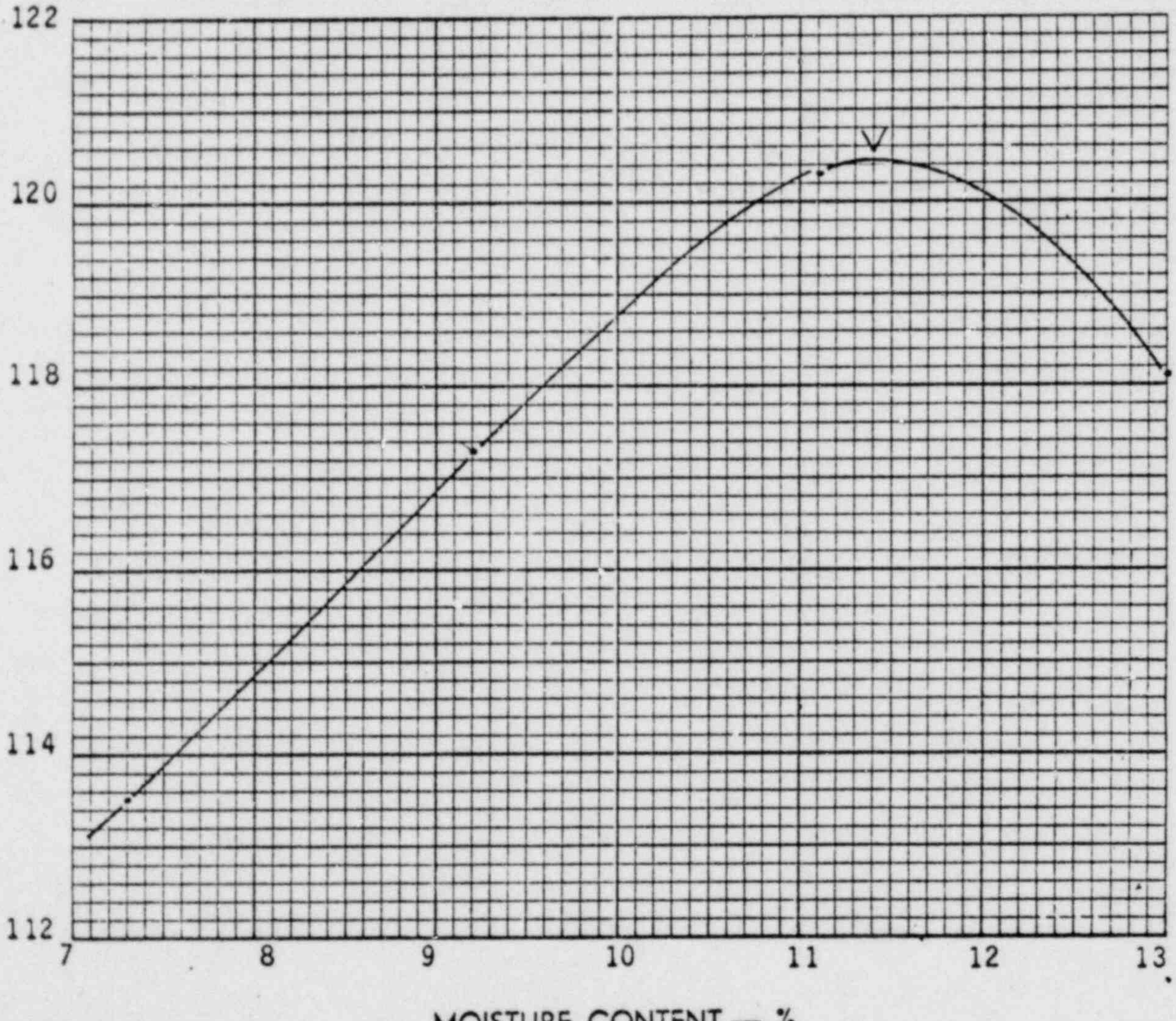
Client: Florida Power Corporation

Project: Crystal River, Unit #3

Location: Soil Cement Berm

Material: Soil Cement

Method of Compaction: AASHO T-180 Mod.



Maximum Dry Density: 120.4 #/ft^3 Optimum Moisture Content: 11.4%

cc: FPC/Edw.Froats (3)

FLORIDA TESTING LABORATORIES, INC.

By [signature]

RECEIVED

DEC 1 1 1969

FLORIDA POWER CORP.
CRYSTAL RIVER CONSTRUCTION

FLORIDA TESTING LABORATORIES, INC.

Phone 531-1446 — P. O. Box 11064
St. Petersburg, Florida 33733

Client Florida Power Corporation
Project Crystal River, Unit #3
Material On Site Limerock
Identification Marks
Quantity Represented
Contractor
Source of Supply On Site

Submitted By FPC
Sampled By FPC
Sampled From
Date Sampled
Date Received
Date Tested
Date Reported

Lab. No. 3425

TEST RESULTS

Tested By

GRADATION

Sieve	% Passing
3/4"	100.0
1/2"	99.8
3/8"	99.1
#4	97.0
#10	92.9
#40	73.7
#60	61.1
#140	25.9
#200	22.2
#270	20.4

LIMEROCK ANALYSIS

Carb. of C_a & Mg = 98.55

ABSORPTION

29.5%

Material run through crusher prior to delivery to Laboratory.

cc: Florida Power Corp. (3)

By Norris A. French
FLORIDA TESTING LABORATORIES, INC.

FORM FTL-4 REV.

ORDER NO. TA-7586

LABORATORY NO. 697743

DATE 10/30/69

FINAL REPORT — TESTS OF CEMENT

FOR Florida Testing Laboratories, Incorporated

PROJECT Crystal River, Unit #3

BRAND Portland FROM

SPECIFICATION ASTM C-150 TYPE II

RESULTS OBTAINED

PHYSICAL TESTS

SAMPLE NO.	SOUNDNESS AUTOCLAVE EXPANSION %	TIME OF SETTING (Vicat) INITIAL	TIME OF SETTING (Vicat) FINAL	COMPRESSIVE STRENGTH AVE. 3 SPEC.—P.S.I. 3 Days	7 Days	28 Days	AIR CONTENT % BY VOL.	SPECIFIC SURFACE (BLAINE) SQ. CM/GM.
1	.02	2 Hrs.	10 Mins.	2660	3970	6370	8.8	3922
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
SPEC. REQ.	[illegible]	[illegible] Mins.	Minimum	1000 Min.	1800 Min.	3500 Min.	12.0 Max.	2600 Min.

CHEMICAL ANALYSIS

	[illegible]	[illegible]	Fe_2O_3	[illegible]	[illegible]	Loss on Ign.	Insol. Res.	C_3S	C_2S	C_3A
1	[illegible]	[illegible]	2.40	[illegible]	[illegible]	[illegible]	[illegible]	57.2		6.[illegible]
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
SPEC. REQ.	21.0 Min.	6.0 Max.	6.0 Max.	5.0 Max.	2.5 Max.	3.0 Max.	.75 Max.	50.0 Max.		[illegible]

REMARKS:

MATERIAL DOES [ ] DOES NOT [ ] COMPLY WITH SPECIFICATIONS.

REG/mb 4-Florida Testing Laboratories, Inc.
mb 2-FTL, Tampa

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

R. E. Gardner, Manager,
Cement & Concrete Department

FLORIDA POWER CORP.

CRYSTAL RIVER UNIT 3

SOIL-CEMENT DESIGN MIX TESTS

JULY 29, 1970

MIX 1 - 219 #/CY
MIX 2 - 251 #/CY
MIX 3 - 282 #/CY

PITTSBURGH TESTING LABORATORY

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PITTSBURGH, PA.

REPORT

SOIL-CEMENT DESIGN FOR WAVE PROTECTION BERM

Florida Power Corporation
Crystal River Unit #3

Index:

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
PITTSBURGH, PA.

CLIENTS No. P.O. No. PR3-1106 REPORT ORDER No. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Soil Cement Design Mix.

DATE: 7-29-70

At the request of Florida Power Corporation, a series of tests were performed to develop a soil-cement design acceptable for the wave protection berm on Crystal River Unit #3.

The PSAR section on soil-cement berm construction references the Bonny Dam project of the US Bureau of Reclamation on which a test section of soil-cement was constructed for wave erosion protection in 1951. (Report EM-630) Therefore it was felt that research into the design and testing methods used by BUREC on that project should provide guidance for the berm construction on Unit #3.

An in-depth look at the BUREC reports on the Bonny Dam and a subsequent project, the Merritt Dam, revealed a basically different approach from that detailed in the PSAR. The PSAR establishes compressive strength of the soil cement mixture as the basic design criteria while BUREC on the above projects follows basically the Portland Cement Association recommendations for design of soil-cement mixtures subject to erosive action. This involves a series of wetting and drying and/or freezing and thawing tests to establish minimum acceptable cement content. The Merritt Dam report, written in 1961, states, "Compressive strength is generally considered supplementary to the freeze-thaw and wet-dry soil-cement tests." (EM 611, p.7)

One very basic difference between the Bonny Dam project and Crystal River Unit #3 is the nature of the soil from which the soil cement will be manufactured. On the Bonny Dam job ,soil used was quartz sand with 20%-35% silt. This basically forms a matrix type structure with the cement and develops its coherence through an internal physical type bond between the individual soil particles and the hydrated cement gel. Conversely, the material to be used on this project as a soil base is native excavated,well weathered limestone with a very high calcium carbonate content. This composition permits the soil base itself to react chemically in the soil-cement mixture, forming cement-like bonds between individual particles and, along with the cement gel, developing an extremely coherent internal structure. Results of the wetting and drying tests which were performed verify this point quite graphically.

One further point concerning the compressive strength criteria of the PSAR section should be made. Reference is made in that section to the 2,000 PSI strength of cores taken from the Bonny Dam test section. It should be clearly understood that these cores were taken some 10 years after construction of the test section and that normal PCA procedures require only that compressive strength of a soil-cement mix be at least 450 PSI and increasing at age 7 days. 28 day strength at Bonny Dam were in the range of 900-1100 PSI, which is roughly comparable to 28 day strengths obtained here at Crystal River.

In consideration of the above, it was decided that the PCA recommendations for soil-cement wave protection design would produce an acceptable structure with a more economical mix than the original PSAR recommendations.

Portland Cement Association design criteria as followed by BUREC on the Bonny Dam project are as follows:

1. Soil-cement losses during either 12 cycles of wet-dry or of freeze-thaw tests shall not exceed 14 percent.

2. The maximum volume change during either the wet-dry or freeze-thaw tests shall not exceed 2 percent of the volume at the time of molding.

3. The maximum moisture content during the wet-dry or freeze-thaw test shall not exceed that quantity which will completely fill the voids of the specimen at the time of molding.

4. Compressive strengths shall increase with age and with increases in cement content.

5. The cement content, as indicated by the criteria of 1 through 4 above which were formulated for highway purposes, shall be increased 2 percent by volume to provide a surface resistant to water erosion.

Test methods for mix design were as follows:

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REPORT

Soil samples were taken from excavated native limerock material stockpiled at the site. No attempt was made to selectively grade the samples, other than to screen all material over a 3/4" sieve for purposes of laboratory testing.

Cement contents of 7%, 8%, and 9% by weight were used in preparation of the test specimens. These mixes were designated respectively for test purposes as Mix #1 containing 219 lbs. of cement per. cu. yd. of soil-cement mixture, Mix#2 containing 251 lbs. per.cu. yd., and Mix#3 containing 282 lbs. per. cu. yd.

Maximum density and optimum moisture of soil-cement mixtures were determined in accordance with ASTM D558, except compactive effort was modified to 5 layers, 25 blows with ~~10~~ 5.5 lb. rammer falling 18".

Test specimens were prepared at moisture contents slightly above optimum and compacted in 4" diameter molds in the above described manner. These samples were then removed from the mold and cured for 7 days in a temperature and humidity controlled curing room, after which they were used for wetting and drying and compressive strength tests.

Results of the wetting and drying tests show extremely low weight losses for all specimens, indicating a very coherent and durable structure. Compressive strengths at 90 days were in the range of 1000 PSI or higher and steadily increasing. Several specimens of Mix #3 were were arbitrarily tested for splitting tensile strength in accordance with ASTM C-496 and developed a range of 200 PSI, which is higher than concrete of comparable quality. As all specimens easily met the PCA requirements, the one with lowest cement factor,i.e. Mix #1, was selected as a base for design. This then was adjusted for cement content on the bases of 2% by volume in accordance with recommended procedure.

In consideration of the above, it is recommended that the soil-cement design mix consist of 270 lbs of type I cement per cu. yd. of native limerock material excavated on-site. This corresponds to a cement content of 10.6% by volume and 8.5% by weight. This mixture would be placed and compacted to at least 98% of maximum density as determined by ASTM method D-558 as modified in this report.

Report prepared by:
W.T. HURST, PTL

P.E. Kornman

P.E. Kornman,P.E.

Respectfully submitted,
PITTSBURGH TESTING LABORATORY

H. J. McGillivray

H.J. McGillivray, Manager
Tampa District

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

CLIENTS No. P.O. No. PR3-1106 REPORT ORDER NO. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Soil Cement Design Mix

DATE: 7-29-70

SUMMARY OF SOIL TESTS

Gradation	#4	79.6% PASSING
	#40	47.4% PASSING
	#200	4.7% PASSING
ATTERBERG LIMITS	L.L.	26
	P.I.	NP
SPECIFIC GRAVITY	2.57	
MAXIMUM DENSITY	116.2 PCF	
OPTIMUM MOISTURE	13.0 %	
VOID RATIO (COMPACTED)		0.380
COEFFICIENT OF PERMEABILITY (COMPACTED)		8×10^{-4} CM/SEC. or 2.4 Ft./YR
SOIL TYPE	AASHO	A-1-b

cc: 1 Client, Mr. Bennett
2 Client, Mr. Froats
1 Wm. T. Hurst

P.E. Kornman
P.E. Kornman, P.E.

Respectfully submitted,
PITTSBURGH TESTING LABORATORY

H. J. McGillivray
H.J. McGillivray, Manager
Tampa District

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

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PITTSBURGH, PA.

Clients No. P.O. No. PR3-1106 REPORT ORDER No. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Soil Cement Design Mix.

DATE: 7-29-70

SUMMARY OF TESTS ON SOIL-CEMENT

	MIX #1.	MIX #2.	MIX #3.
PERCENT CEMENT BY VOLUME	8.6	9.9	11.1
PERCENT CEMENT BY WEIGHT	7	8	9
MAXIMUM DENSITY PCF	117.3	118.5	117.9
OPTIMUM MOISTURE %	13.5	13.4	13.7
WET-DRY TEST			
MAXIMUM MOISTURE CONTENT %	----	----	14.2
SATURATION MOISTURE %	----	----	14.8
MAXIMUM VOLUME CHANGE %	1.2	1.1	1.1
WEIGHT LOSS %	1.53	1.18	0.61
COMPRESSIVE STRENGTH			
7 DAY (PSI-Avg.)	490	560	650
28 DAY (PSI-Avg.)	720	960	1080
90 DAY (PSI-Avg.)	930	1100	1240

Respectfully submitted,
PITTSBURGH TESTING LABORATORY

P.E. Kornman

H. J. McGillivray

cc: 1 Client, Mr. Bennett P.E. Kornman, P.E.
2 Client, Mr. Froats
1 Wm. T. Hurst

H.J. McGillivray, Manager
Tampa District

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
PITTSBURGH, PA.

CLIENTS No. P.O. No. PR3-1106 — **REPORT** — ORDER No. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Wetting and Drying Test (ASTM-D559)

DATE: 7-29-70

CYCLE	1.	2.	3.	4.	5.	6.
MIX #1						
VOLUME CHANGE	0.8%	1.0%	0.9%	0.7%	0.6%	1.2%
WEIGHT LOSS	0.22%	0.34%	0.45%	0.56%	0.67%	0.84%
MIX #2						
VOLUME CHANGE	0.9%	0.9%	0.6%	1.1%	0.7%	1.0%
WEIGHT LOSS	0.39%	0.62%	0.84%	0.84%	0.90%	0.96%
MIX #3						
VOLUME CHANGE	1.0%	0.8%	0.7%	0.9%	0.5%	1.1%
WEIGHT LOSS	0.17%	0.39%	0.50%	0.56%	0.56%	0.56%

CYCLE	7.	8.	9.	10.	11.	12.
MIX #1						
VOLUME CHANGE	0.9%	0.7%	0.9%	0.3%	0.5%	0.7%
WEIGHT LOSS	0.95%	0.9[illegible]%	1.35%	1.40%	1.46%	1.53%
MIX #2						
VOLUME CHANGE	0.9%	0.5%	0.3%	0.5%	0.5%	0.3%
WEIGHT LOSS	1.07%	1.07%	1.12%	1.12%	1.18%	1.18%
MIX #3						
VOLUME CHANGE	1.1%	0.5%	0.7%	0.3%	0.4%	0.6%
WEIGHT LOSS	0.61%	0.61%	0.61%	0.61%	0.61%	0.61%

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

cc: 1 Client, Mr. Bennett
2 Client, Mr. Froats
1 Wm. T. Hurst

P.E. Kornman

P.E. Kornman, P.E.

H. J. McGillivray

H.J. McGillivray, Manager
Tampa District

PITTSBURGH TESTING LABORATORY

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Order No. TA-7732

Report No. ____________

Date 3-27-70

REPORT

REPORT OF TEST ON CONCRETE CYLINDERS
6" DIAMETER BY 12" LENGTH

REPORTED TO: FLORIDA POWER CORP.
PROJECT: CRYSTAL RIVER PLANT UNIT NO. 3
Concrete Supplier: West Coast Concrete, Inc.
Arch-Engineer: Gilbert Assoc., Inc.
General Contractor: J. A. Jones

Location of Concrete Placement SOIL CEMENT DESIGN MIX

Date Cast 3-27-70 Concrete Class ------------- P.S.I. Area 12.57 Sq. In.

Cement Factor ------------- Brand ------------- Type ------------- Aggregate Size ---------------

DEL. S/N.	CYLINDER IDENT.	W/C GAL.-SACK	SLUMP INCHES	AIR %	CONCRETE TEMP. °F.	DATE TESTED	AGE DAYS	TOTAL LOAD LBS.	COMP. ST. P.S.I.	SPECIFIC LOCATION
	Mix #1					4-4	7	6400	510	
	"					"	7	5900	470	
	Mix #2					"	7	6700	535	
	"					"	7	7300	580	
	Mix #3					"	7	7600	605	
	"					"	7	8800	700	
	Mix #1					4-25	28	8600	685	
	"					"	28	9500	755	
	Mix #2					"	28	11500	915	
	"					"	28	13700	1010	
	Mix #3					"	28	12600	1000	
	"					"	28	14500	1155	
	Mix #1					6-26	90	12300	980	
	"					"	90	11100	885	
	Mix #2					"	90	12900	1030	
	"					"	90	14800	1180	
	Mix #3					"	90	16200	1290	
	"					"	90	15000	1195	

REMARKS:

1 – Florida Power Corp., H. C. Bennett
2 – Florida Power Corp., E. E. Froats
1 – PTL – Tampa
1 – PTL – W. T. Hurst
1 – PTL – Pgh.

P.E. Kornman
P.E. Kornman, P.E.

PITTSBURGH TESTING LABORATORY
H. J. McGillivray
H. J. McGillivray, Manager

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

CLIENTS No. P.O. No. PR3-1106 REPORT ORDER No. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Maximum Density Tests on Soil-Cement Mixtures.

DATE: 7-29-70

Test for maximum density and optimum moisture on Mixes #1, #2, #3 were performed in accordance with ASTM D-558 modified as follows: Compactive effort was increased to 5 layers, 25 blows per layer with 10 lb. rammer falling 18".

Results were as follows:

Mix #1 ---------------117.3 PCF @ 13.5%

Mix #2----------------118.5 PCF @ 13.4%

Mix #3----------------117.9 PCF @ 13.7%

cc: 1 Client, Mr. Bennett
2 Client, Mr. Froats
1 Wm. T. Hurst

P.E. Kornman
P.E. Kornman, P.E.

H. J. McGillivray
H.J. McGillivray, Manager
District, Tampa

PITTSBURGH TESTING LABORATORY

PROCTOR DENSITY TESTS

Lab No.: ______

TA No.: 7732

CLIENT: FLORIDA POWER CORPORATION

REPORT OF: *~~(Standard)~~ Proctor Density Test of Limerock for soil-cement
(Modified) *Mark out one

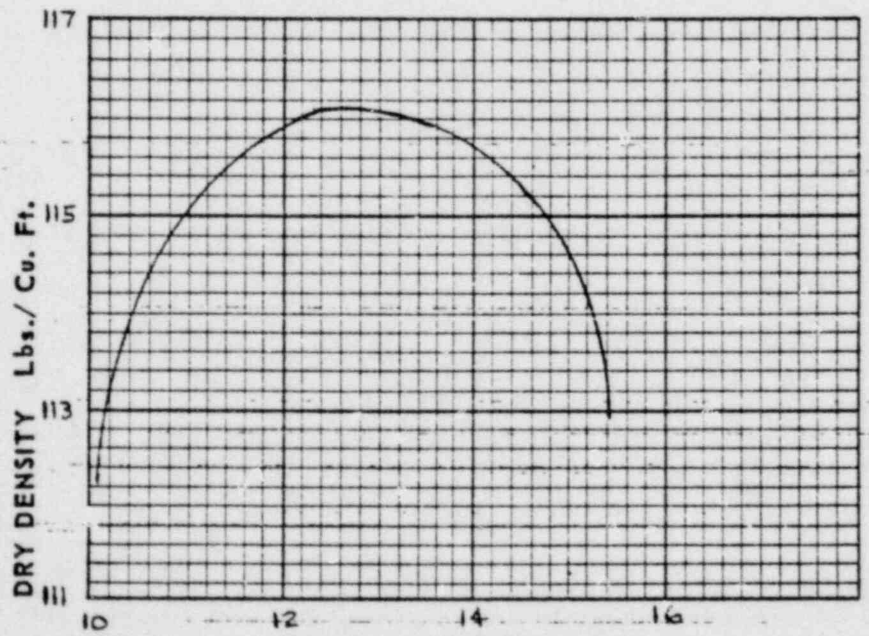
PROJECT: Crystal River, Unit #3

SAMPLED BY: Wm. T. Hurst DATE: 7-29-70

SPECIFICATION: **(See below) ASTM D1557 Method: C

RESULTS:

	Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
1.	123.5	10.1	112.3
2.	130.5	12.3	116.1
3.	132.2	14.6	115.5
4.	128.0	15.0	111.3



OPTIMUM MOISTURE 13.0 MAXIMUM DENSITY 116.2

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
INSPECTING ENGINEERS AND CHEMISTS
512 NORTH DELAWARE AVE.
TAMPA, FLORIDA 33606
AREA CODE 813
PHONE: 253-3485

November 22, 1972 PTL Ref: TA-7732

Mr. James Harris
Gilbert Associates
525 Lancaster Avenue
Reading, Pennsylvania 19602

Re: Compaction Data
Florida Power Corporation

Dear Mr. Harris:

Enclosed you will find data on laboratory compaction and field density testing of the limerock fill compacted as Zone III material. Figure I summarizes compaction to date.

The figure #2 entitled Compaction Characteristics was made up for J. A. Jones Company at the request of Gus Packos when the compaction difficulties arose with the only available limerock material. After consideration of the data on this figure, we advised J. A. Jones to use a tamping foot roller for initial compaction followed by the 30 ton rubber tyne roller. The field results have borne out the conclusions drawn from the laboratory testing; in other words, that high stress kneeding type compaction would be required to achieve the required density. All of the field data indicates that the compaction water contents are well wet of optimum for the high energy compaction now being used, but drying the material has not be practical.

The laboratory study indicates that the controlling perameter in the moisture-density relationship for any limerock sample appears to be the specific gravity of particles. This factor seems to be a highly variable quantity and determines the maximum density to which that sample can be compacted. We also found that the specific gravity of the particles scalped according to ASTM D-1557, Method C, was higher than the specific gravity of particles for the remaining material. The reduced field average specific gravity leads to the need for increased compaction effort in order to reach the requirements established in terms of Method C.

The results of triaxial testing are being sent under seperate cover. We apologize for the delay, but the requirement for the measurement of pore pressures in compacted samples made our own equipment inadequate for accurate testing. Due to the necessity for high back pressures in order to prevent pore water cavitation due to dialation, none of our usual sources were adequate either. Since we were in the process of up-grading our own equipment, we decided to rush the re-building rather than try to squeeze in on someone else's time schedule for tests in which little faith could be put in the results. The test series is now complete and will follow this data immediately.

We appreciate the opportunity of handling this testing for you and shall be glad to discuss the results with you at any time.

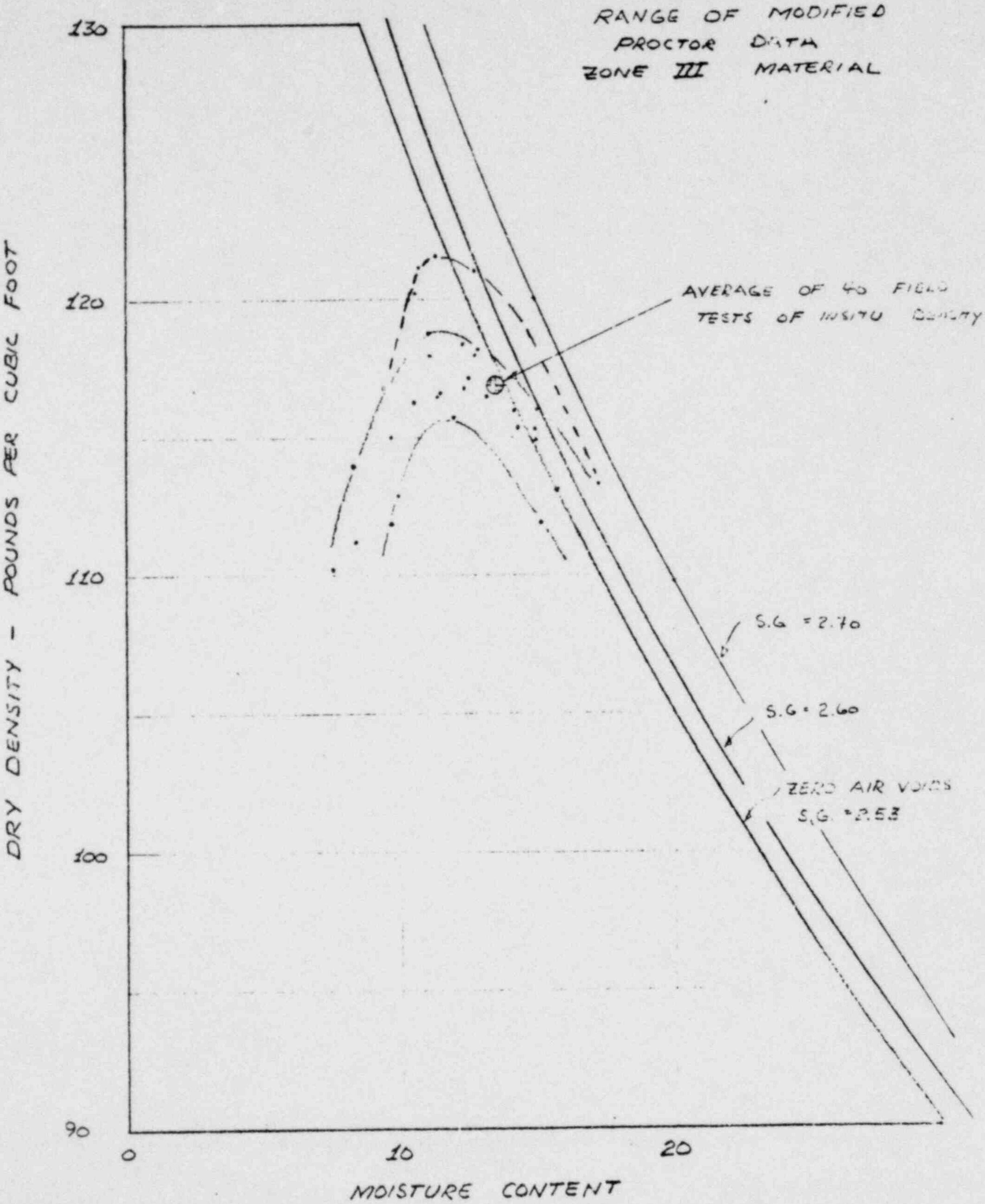
Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

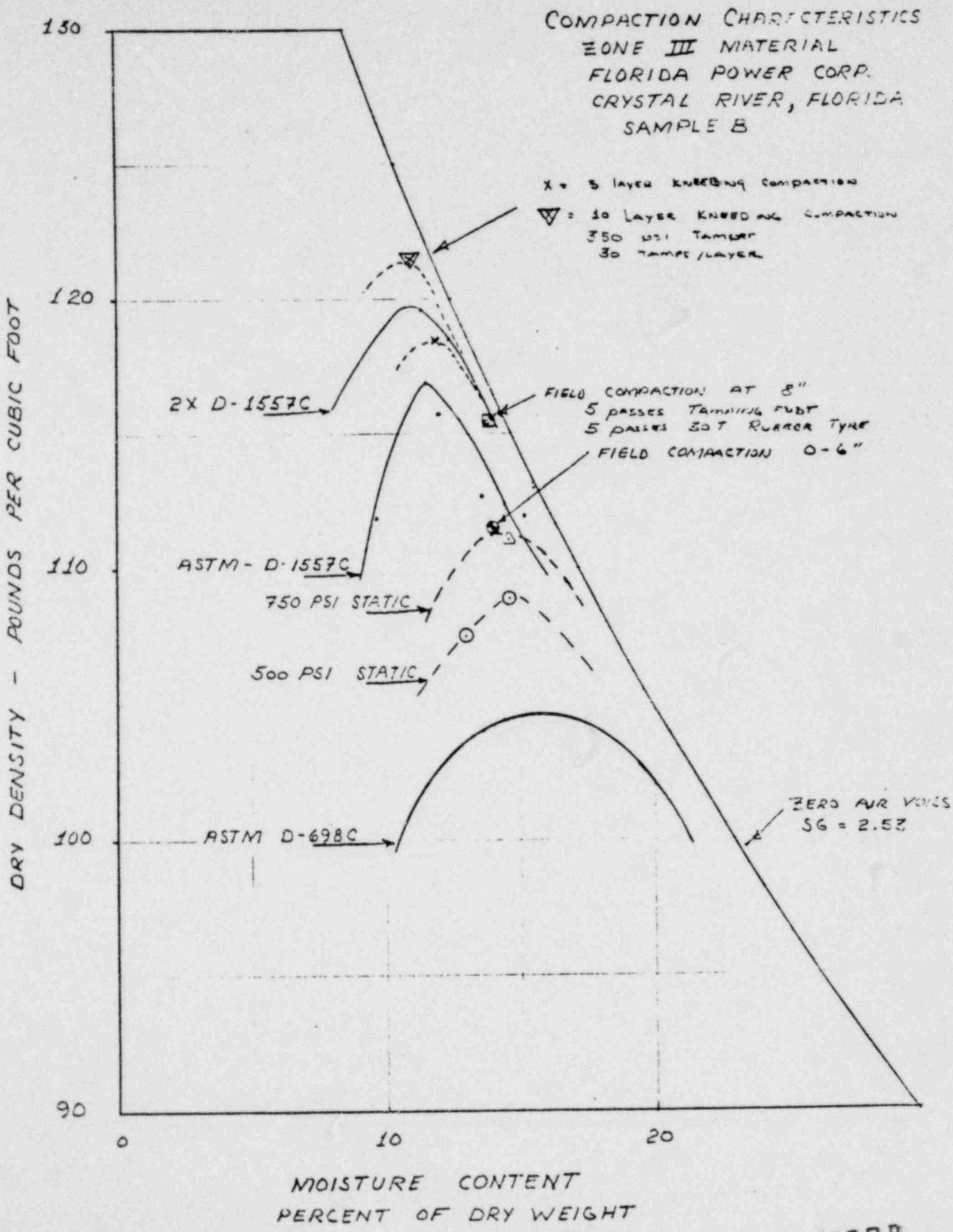
Ross T. McGillivray, P.E.
Tampa District Engineer

COMPACTION DATA
FLA. POWER CORP , CRYSTAL RIVER , FLORIDA



PITTSBURGH TESTING LABORATORY
TAMPA , FLORIDA

POOR ORIGINAL



PITTBURGH TESTING LABORATORY
TAMPA, FLORIDA

POOR ORIGINAL

FORM . ' REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-66

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Sieve Analysis of Zone III LimeRock Fill
Sample "B"

TESTED BY: J. Byrne, PITTSBURGH TESTING LABORATORY

DATE TESTED: March 22, 1972

Sieve Analysis:

Sieve Size	Cumulative % Passing
3/4"	97.0
1/2"	91.8
3/8"	84.4
No. 4	73.5
No. 10	65.0
No. 20	55.3
No. 40	49.8
No. 60	44.8
No. 80	40.7
No. 140	34.7
No. 200	30.8
Pan	0

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

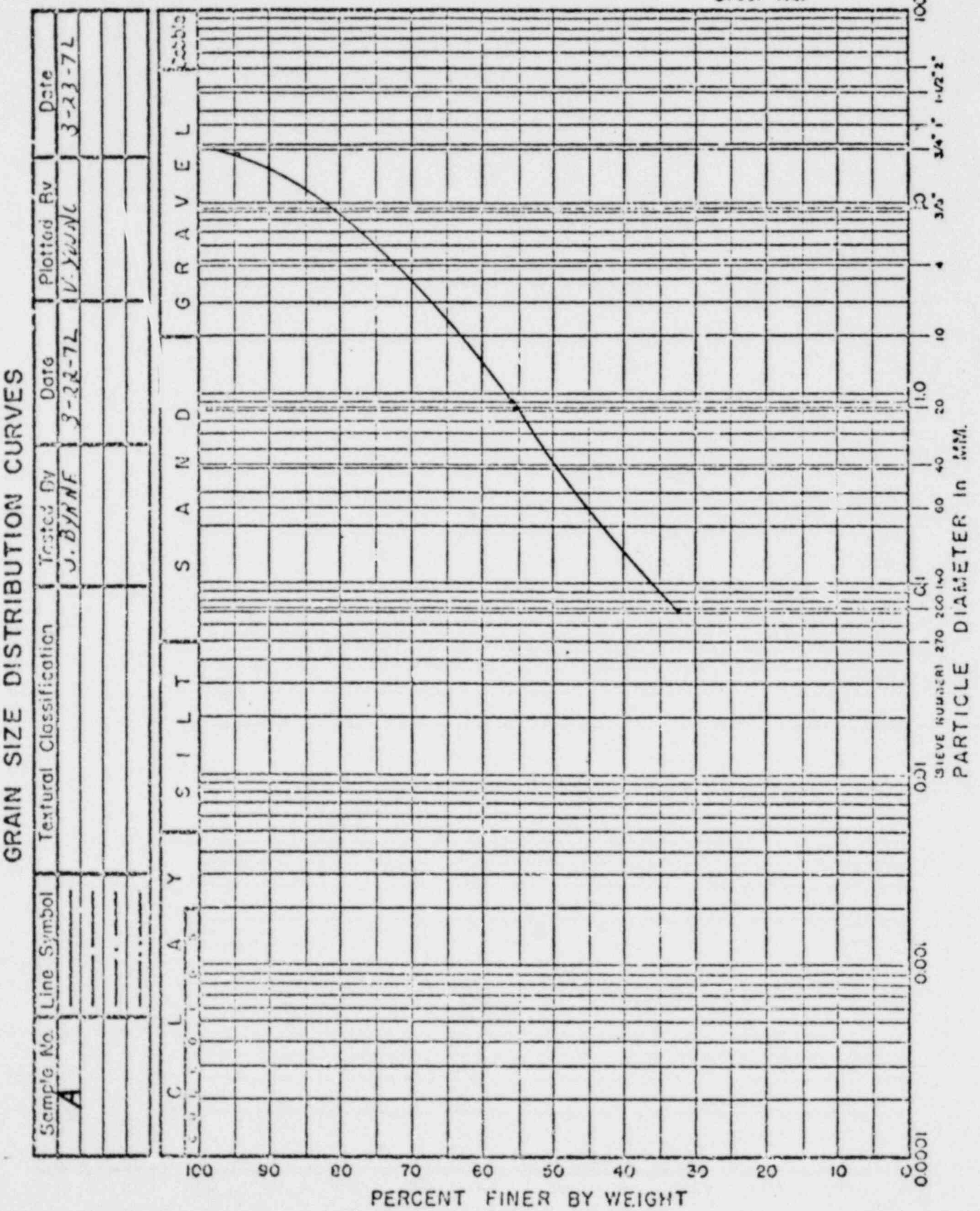
Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

PITTSBURGH TESTING LABORATORY

Order No.



POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-65

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Sieve Anylsis of Zone III Lime Rock Fill
Sample "A"

TESTED BY: J. Byrne, PITTSBURGH TESTING LABORATORY

DATE TESTED: March 22, 1972

Sieve Analysis:

Sieve Size	Cumulative % Passing
3/4"	98.4
1/2"	87.5
3/8"	80.7
No. 4	72.4
No. 10	65.2
No. 20	55.8
No. 40	50.4
No. 60	46.3
No. 80	42.2
No. 140	36.3
No. 200	33.2
Pan	0

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

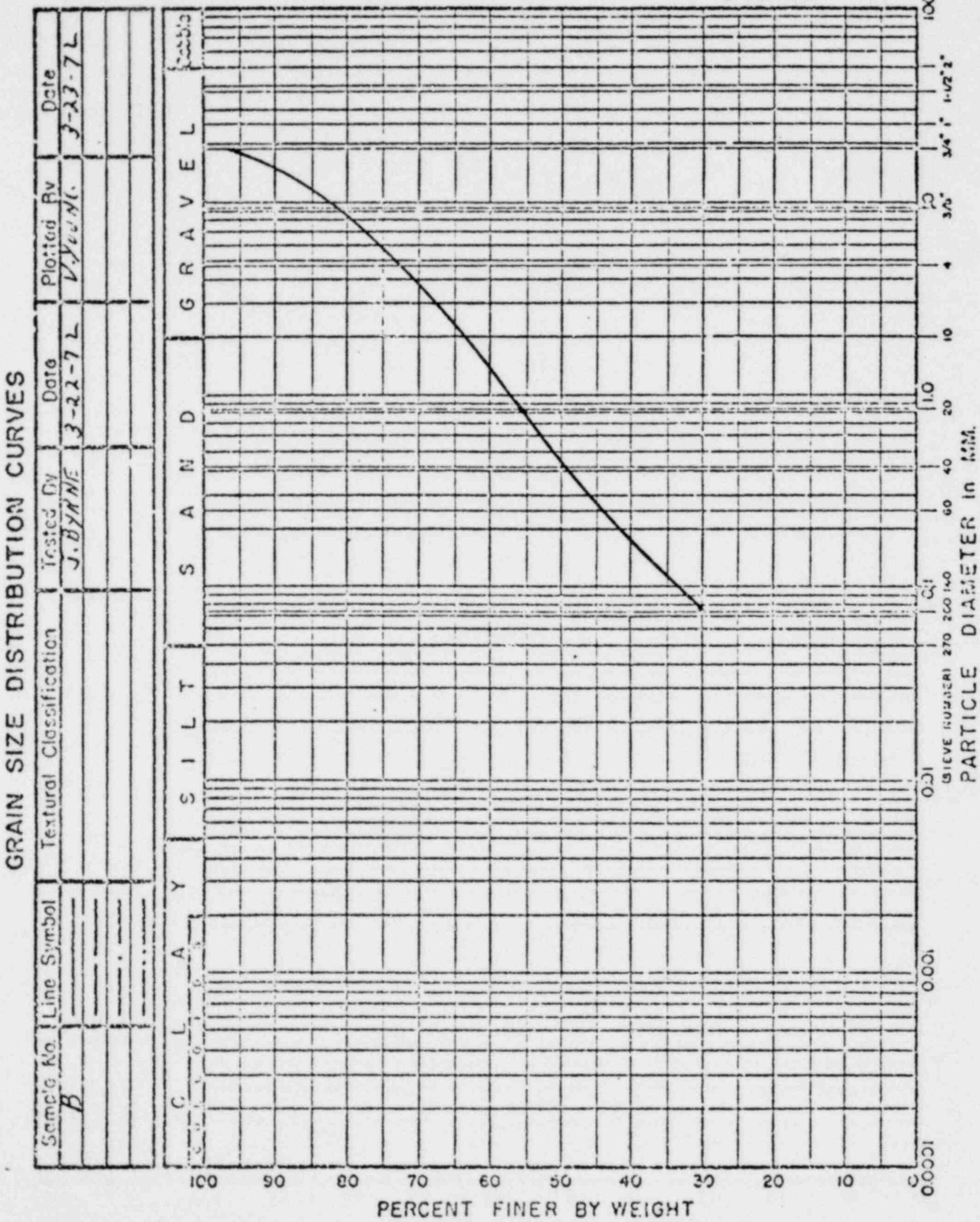
Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

PITTSBURGH TESTING LABORATORY

Order No.



POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-60

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT: Florida Power Corporation
P. O. Boc 276
Crystal River, Florida 32629

REPORT OF: Modified Proctor Density Test of Lime Rock Sample "A"

PROJECT: Florida Power

SAMPLED BY: R. McGillivray, PITTSBURGH TESTING LABORATORY

SPECIFICATIONS: ASTM D-1557-70 Method "C"

DATE: March 21, 1972

RESULTS:

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
122.7	9.7	111.9
129.6	12.0	115.7
129.0	15.2	111.9

Maximum Dry Density 115.8 pounds per cubic foot.

Optimum Moisture 12.5 per cent.

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-64

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Double Energy Modified Proctor Density Test of Lime Rock Sample "A"

PROJECT: Florida Power

SAMPLED BY: R. McGillivray, PITTSBURGH TESTING LABORATORY

SPECIFICATIONS: 2X ASTM 1557-70 Method "C"

DATE: March 21, 1972

RESULTS:

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
132.6	11.4	119.0

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-63

ORDER No. TA-7732

CLIENT'S No.

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Standard Proctor Density Test of Lime Rock
Sample "A"

PROJECT: Florida Power

SAMPLED BY: R. McGillivray, PITTSBURGH TESTING LABORATORY

SPECIFICATIONS: ASTM D-698-58T Method "C"

DATE: March 21, 1972

RESULTS:

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
118.8	13.6	104.6

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne

sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-59

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT:	Florida Power Corporation P. O. Box 276 Crystal River, Florida 32629
REPORT OF:	Modified Proctor Density Test of Lime Rock Sample "B"
PROJECT:	Florida Power
SAMPLED BY:	R. McGillivray, PITTSBURGH TESTING LABORATORY
SPECIFICATION:	ASTM D-1557-70 Method "C"
DATE:	March 21, 1972

RESULTS:

Wet Density Lbs./Cu. Ft.	Moisture Per Cent	Dry Density Lbs./Cu. Ft.
123.3	12.0	110.1
132.6	13.6	116.7
130.8	15.7	113.1

Maximum Dry Density 116.7 pounds per cubic foot.

Optimum Moisture 13.7 per cent,

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-62

CLIENT'S No.

ORDER No. TA-7732

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Double Energy Modified Proctor Density Test of Lime Rock Sample "B"

PROJECT: Florida Power

SAMPLED BY: R. McGillivray, PITTSBURGH TESTING LABORATORY

SPECIFICATIONS: 2X-1557-70 Method "C"

DATE: March 21, 1972

RESULTS:

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
133.2	12.4	118.5

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. E. Froats
1 G. B. Browne
sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 11-61

ORDER No. TA-7732

CLIENT'S No.

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Standard Proctor Density Test of Lime Rock Sample " B"

PROJECT: Florida Power

SAMPLED BY: R. McGillivray, PITTSBURGH TESTING LABORATORY

SPECIFICATIONS: ASTM D698-58T

DATE: March 21, 1972

RESULTS:

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
119.4	14.4	104.4

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

R. T. McGillivray, P. E.
Tampa District Engineer

cc: 1 Client ATTN: H. L. Bennett
1 Client ATTN: E. F. Froats
1 G. B. Browne
sa: 11-13-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

ORDER NO. TA-7732
LAB NO. 11-101

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Unit Weight by Static Compaction

PROJECT: Florida Power

DATE: March 23, 1972

SAMPLE A

	5 Layers at 500 PSI	3 Layers at 500 PSI
Weight of Materials	8.09	8.09
Weight of Mold	4.04	4.04
Weight of Soil	4.05	4.05
Wet Unit Weight	121.5	121.5
Percent Moisture	13.0	13.0
Dry Unit Weight	107.5	107.5

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

Ross T. McGillivray, P. E.
Tampa District Engineer

cc: 3 Client
sa: 11-21-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

ORDER NO. TA-7732
LAB NO. 11-102

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Unit Weight by Static Compaction

PROJECT: Florida Power

DATE: March 25, 1972

SAMPLE A

5 Layers at 500 PSI

Wet Unit Weight	Moisture Percent	Dry Unit Weight
124.8	14.6	108.9

3 Layers at 750 PSI

Wet Unit Weight	Moisture Percent	Dry Unit Weight
127.2	14.6	111.1

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

Ross T. McGillivray, P. E.
Tampa District Engineer

cc: 3 Client
sa: 11-21-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881
PITTSBURGH, PA.

ORDER NO. TA-7732
LAB NO. 11-104

REPORT

CLIENT: Florida Power Corporation
P. O. Box 276
Crystal River, Florida 32629

REPORT OF: Kneeding Compaction
10 Layers
350 PSI Tamper
30 Tamps/layer

SPECIFICATIONS: ASTM D-1557 Method C

RESULTS: SAMPLE B

Wet Density Lbs/Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
116.9	1.2	115.1
122.8	5.4	116.2
128.5	9.3	117.3
135.0	11.6	121.0
130.9	14.0	114.6

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

Ross T. McCillivray, P. E.
Tampa District Engineer

cc: 3 Client
sa: 11-21-72

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881
PITTSBURGH, PA.

ORDER NO. TA-7732
LAB NO. 11-103

REPORT

CLIENT: Florida Power Corporation
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Kneeding Compaction
5 Layers
350 PSI Tamper
30 Tamps/layer

SPECIFICATIONS: ASTM D-1557 Method C

RESULTS:

Wet Density Lbs/ Cu. Ft.	Moisture Per Cent	Dry Density Lbs/Cu. Ft.
113.5	1.4	111.8
118.4	5.7	112.0
126.7	9.8	115.5
132.4	12.3	117.9
128.5	14.3	113.3

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

Ross T. McGillivray, P. E.
Tampa District Engineer

cc: 3 Client
sa: 11-21-72

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

CLIENTS No. P.O. NO. PR3-1106 REPORT ORDER No. TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Modified Proctor Density Test of ZONE III.

PROJECT: Crystal River Unit #3, Nuclear Generating Plant.

SAMPLED BY: G.B. Browne, PITTSBURGH TESTING LABORATORY

SPECIFICATION: ASTM D 1557, Method C.

DATE: 10-24-70

RESULTS:

WET DENSITY lbs/ Cu. Ft.	MOISTURE Per Cent	DRY DENSITY lbs/ Cu. Ft.
128.9	10.5	116.3
130.0	11.5	116.6
132.1	12.6	117.2
132.0	13.2	116.5
132.1	14.3	115.3

Maximum Dry Density 117.2 Pounds per cubic foot.
Optimum Moisture 12.6%

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

H.J. McGillivray, Manager
Tampa District

P.E. Kornman

P.E. Kornman, P.E.

cc: 1 Client, Mr. Bennett
2 Client, Mr. Froats
1 Wm. T. Hurst

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

CLIENTS No. P.O. NO PR3-1106 **REPORT** ORDER NO: TA-7732

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Florida 32629

REPORT OF: Modified Proctor Density Test of ZONE III

PROJECT: Crystal River Unit #3, Nuclear Generating Plant.

SAMPLED BY: G.B. Browne

SPECIFICATION: ASTM D 1557, Method C.

DATE: 3-1-71

RESULTS:

WET DENSITY lbs/ Cu. Ft.	MOISTURE Per Cent	DRY DENSITY lbs/ Cu. Ft.
132.6	10.4	120.0
133.0	10.6	120.2
134.6	10.8	121.2
136.9	11.4	121.6
136.5	12.8	121.0

Maximum Dry Density 121.6 PCF

Optimum Moisture 11.4%

cc: 1 Client: Mr. Bennett
2 Client: Mr. Froats
1 Wm. T. Hurst

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

H. J. McGillivray

H.J. McGillivray, Manager
Tampa District

P.E. Kornman

P.E. Kornman, P.E.

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No.

CLIENT'S No.

ORDER No.

REPORT

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Fla., 32629

REPORT OF: Modified Proctor Density Test of ZONE III

PROJECT: CRYSTAL RIVER UNIT #3, Generating Plant

SAMPLED BY: G.B. BROWNE

SPECIFICATION: ASTM D 1557, Method C

DATE: 8-8-72

* *

RESULTS:

WET DENSITY lbs/Cu Ft.	MOISTURE Per Cent	DRY DENSITY lbs/ Cu.Ft.
118.6	7.6	110.2
123.4	8.3	114.0
127.8	9.4	116.8
133.1	12.4	118.4
133.4	12.9	118.2
132.6	15.0	115.3

Maximum Dry Density: 118.4 PCF

Optimum Moisture 12.4%

cc; 1 Client: Mr. H.L.Bennett
1 Client: Mr. E.E. Frosts
1 P.T.L.

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

E.J. WAMSLEY, P.E.

H.J. McGILLIVRAY, MANAGER

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No.

CLIENT'S No.

ORDER No.

REPORT

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Fla., 32629

REPORT OF: Modified Proctor Density Test of ZONE III

PROJECT: CRYSTAL RIVER UNIT #3, Generating Plant

SAMPLED BY: G.B. Browne, P.T.L.

SPECIFICATION: ASTM D 1557, Method C

DATE: 4-3-72

* *

RESULTS:

WET DENSITY lbs/Cu Ft.	MOISTURE Per Cent	DRY DENSITY Lbs/ Cu.Ft.
120.5	8.4	111.2
126.2	9.7	115.0
129.8	11.4	116.5
131.3	12.4	116.8
132.5	14.2	116.0

Maximum Dry Desnity: 116.8 PCF

Optimum Moisture: 12.4

cc: 1 Client: Mr. H.L. Bennett
1 Client: Mr. E.E. Froats
1 P.T.L.

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

E.J. Wamsley

E.J. WAMSLEY, P.E.

H.J. McGillivray

H.J. McGILLIVRAY, MANAGER

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No.

CLIENT'S No.

ORDER No.

REPORT

REPORT FOR: FLORIDA POWER CORPORATION
P.O. Box 276
Crystal River, Fla. 32629

REPORT FOR: Modified Proctor Density Test of ZONE III

PROJECT: CRYSTAL RIVER UNIT #3, Generating Plant

SAMPLED BY: G.B. BROWNE

SPECIFICATION: ASTM D 1557, Method C

DATE: 9-28-72

** ** * * * * * * * * *

REDULTS:

WET DENSITY lbs/Cu Ft.	MOISTURE Per Cent	DRY DENSITY lbs/Cu Ft.
129.6	10.0	117.8
132.0	11.1	118.8
132.3	15.1	114.9
132.9	17.3	113.3
133.1	12.8	118.0

Maximum Dry Density: 118.8 PCF

Optimum Moisture: 11.1%

cc; 1 Client: Mr. H.L. Bennett
1 Client: Mr. E.E. Froats
1 P.T.L.

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

E.J. WAMSLEY, P.E.

H.J. McGILLIVRAY, MANAGER

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-4-77

Material: Zone III

Sampled by: Timothy Gibboney

Date: 10-4-77

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. east of Column Line 301, ------ ft. ---- of Column Line B

Elev: 105'6" Field dry den 119.50 Field moist 16.00 % compaction 100.5%

Test No. 2 45' ft east of Column Line 301, — ft. — of Column Line B.

Elev: 107'6" Field dry den 116.75 Field moist. 16.25 % compaction 98.4%

Test No. 3 55' ft. east of Column Line 301, – ft. — of Column Line A

Elev: 106'10" Field dry den 117.75 Field moist 117.75 ? % compaction 99%

Test No. 4 _____ ft _____ of Column Line ____, ____ ft. _____ of Column Line _____

Elev: _________

All above test meet specification requirement.

Test ~~No.~~ [illegible] and _____ meet specification requirement after recompaction.

Maximun Dry Density: (~~[illegible]~~) (111.2) PCF

118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-11-72

Material: Zone III

Sampled by: Timothy Gilboney

Date: 10-11-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 45' ft. north of Column Line A, ------ft. ---- of Column Line 30.7

Elev: 101 Field dry den 118.75 Field moist 13.25 % compaction 99.9%

Test No. 2 45' ft north of Column Line A, - ft. — of Column Line 30.7.

Elev: 101 Field dry den 119.70 Field moist 12.80 % compaction 100.7%

Test No. 3 ____ft. ____of Column Line ____, ____ft. ____of Column Line ____

Elev: ____

Test No. 4 ____ft ____ of Column Line ____, ____ft. ____of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~120.0~~ 118.4) (11 2) PCF

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-10-72

Material: Zone III

Sampled by: Timothy Gibbons

Date: 10-10-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 45 ft. east of Column Line 301, ------ft. ---- of Column Line A

Elev: 111

Field dry den 117.3 | Field moist. 12.30 | % compaction 98.9

Test No. 2 30' ft east of Column Line 301, 10' ft. south of Column Line A.

Elev: 112

Field dry den 118.20 | Field moist. 12.30 | % compaction 99.5%

Test No. 3 18' ft. east of Column Line 301, 10' ft. south of Column Line B

Elev: 113

Field dry den 115.85 | Field moist. 13.75 | % compaction 97.8%

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~121.0~~ 118.8) (111.2) PCF

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-10-72

Material: Zone III

Sampled by: Timothy Gibbons

Date: 10-10-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. south of Column Line 5, ------ft. ---- of Column Line 302

Elev: 106 Field dry den 118.00 Field moist. 12.25 % compaction 99.5%

Test No. 2 20' ft east of Column Line 301, 35 ft. north of Column Line 5.

Elev: 106 Field dry den 116.75 Field moist. 12.25 % compaction 98.3%

Test No. 3 40 ft. east of Column Line 301, 40' ft. north of Column Line 5

Elev: 106 Field dry den 116.4 Field moist. 11.10 % compaction 98.1%

Test No. 4 ______ft ______ of Column Line ______, ______ft. ______of Column Line ______

Elev: ______

All above test meet specification requirement.

Test No. ______ and ______ meet specification requirement after recompaction.

Maximun Dry Density: (~~[illegible]~~) (111.2) PCF
118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-10-72

Material: Zone III

Sampled by: Timothy Gibboney

Date: 10-10-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (~~Sand-Cone~~)

Test No. 1 35' ft. east of Column Line 301, 10' ft. south of Column Line C

Elev: 110 Field dry den 116.75 Field moist 12.75 % compaction 98.3%

Test No. 2 20' ft east of Column Line 301, — ft. — of Column Line E.

Elev: 110 Field dry den 117.50 Field moist 12.00 % compaction 99%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~121.0~~) (111.2) PCF

118.8

POOR ORIGINAL

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PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-10-72

Material: Zone III

Sampled by: Timothy Gibbons

Date: 10-10-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (~~Sand Cone~~)

Test No. 1 45' ft. north of Column Line A, 10' ft. east of Column Line 301

Elev: 101 Field dry den 116.06 Field moist 12.50 % compaction 97.8%

Test No. 2 45' ft north of Column Line A, — ft. — of Column Line 301

Elev: 101 Field dry den 117.00 Field moist 12.50 % compaction 98.7%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximun Dry Density: ([illegible]) (111.2) PCF

118.8

POOR ORIGINAL

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PITTSBURGH TESTING LABORATORY ENGINEERS AND CHEMISTS

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-9-72

Material: Zone III

Sampled by: ~~10-9-72~~ Timothy Gilbrey

Date: 10-9-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) ~~(Sand Cone)~~ no good, failed

Test No. 1 45' ft. north of Column Line A, ------ ---- of Column Line 301

Elev: 101 Field dry den 115.9 Field moist 11.60 % compaction 94.6%

Test No. 2 45' ft north of Column Line A, ___ ft. ___ of Column Line 302

Elev: 101 Field dry den 115.25 Field moist 12.50 % compaction 97%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ______ and ______ meet specification requirement after recompaction.

Maximum Dry Density: (~~121.0~~) (111.2) PCF

118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-9-72

Material: Zone III

Sampled by: Timothy Gilboney

Date: 10-9-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (~~Sand Cone~~)

Test No. 1 15 ft. east of Column Line 301, 10' ft. south of Column Line A

Elev: 112 Field dry den 118.25 Field moist. 16.75 % compaction 99.6%

Test No. 2 30' ft east of Column Line 301, 5' ft. north of Column Line B.

Elev: 111'8" Field dry den 118.4 Field moist 15.60 % compaction 99.7%

Test No. 3 40' ft. east of Column Line 301, — ft. — of Column Line A

Elev: 111 Field dry den 116.50 Field moist 16.50 % compaction 98%

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~121.0~~) (111.2) PCF
118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10/9/72

Material: Zone III

Sampled by: Timothy Gibboney

Date: 10/9/72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (~~Sand Cone~~)

Test No. 1 25---ft. --east-- of Column Line 301-, -40---ft. north of Column Line --A--

Elev:	Field Dry Den.	Field Moisture	% Compaction
100	119.10	10.50	100.1%

Test No. 2 10----ft West of Column Line 301, 45 ft. North of Column Line A.

Elev:	Field Dry Den.	Field Moisture	% Compaction
100	119.95	14.25	100.9%

Test No. 3 ____ft. ____of Column Line ____, ____ft. ____of Column Line ____

Elev: ____

Test No. 4 ____ft ____ of Column Line ____, ____ft. ____of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. 2 and 1 meet specification requirement after recompaction.

Maximum Dry Density: (~~121.8~~ 118.8) (111.2) PCF

118.4

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY
ENGINEERS AND CHEMISTS

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-9-72

Material: Zone III

Sampled by: Timothy Dibbins

Date: 10-9-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) ~~(Sand Cone)~~

Test No. 1 18' ft. east of Column Line 301, 10' ft. south of Column Line B

Elev: 110' Field dry den 116.50 Field moist. 13.00 % compaction 98%

Test No. 2 35' ft east of Column Line 301, — ft. — of Column Line B.

Elev: 109'10" Field dry den 116.25 Field moist. 12.25 % compaction 97.8%

Test No. 3 45' ft. east of Column Line 301, — ft. ___ of Column Line A

Elev: 109 Field dry den 118.5 Field moist. 12.5 % compaction 99.7%

Test No. 4 _____ ft _____ of Column Line ___, ___ ft. _____ of Column Line _____.

Elev: __________

All above test meet specification requirement.

Test No. _______ and ______ meet specification requirement after recompaction.

Maximun Dry Density: (~~121.0~~) (111.2) PCF

118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT Date 10-9-72

Material: Zone III

Sampled by: Timothy Gibbing

Date: 10-9-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) ~~(Sand Cone)~~

Test No. 1 40' ft. north of Column Line B, ------ft. --- of Column Line 30.3

Elev: 100' Field dry den. 109.4 Field moist 12.60 % compaction 92.3%

Test No. 2 40' ft north of Column Line B, — ft. — of Column Line 30.4

Elev: 100' Field dry den. 115.4 Field moist. 11.60 % compaction 97.3%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~121.6~~) (111.2) PCF
118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-6-72

Material: Zone III

Sampled by: Timothy Gibbney

Date: 10-6-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. east of Column Line 301, 10' ft. east of Column Line A

Elev: 109'8" Field dry den 116.00 Field moist. 13.00 % compaction 97.7%

Test No. 2 40' ft east of Column Line 301, — ft. — of Column Line B.

Elev: 109 Field dry den. 116.75 Field moist. 13.25 % compaction 98.2%

Test No. 3 40' ft. east of Column Line 301, — ft. — of Column Line B

Elev: 109 Field dry den 120.30 Field moist 13.30 % compaction 101.2%

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximun Dry Density: (~~121.0~~) (111.2) PCF
118.8

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY ENGINEERS AND CHEMISTS

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-6-77

Material: Zone III

Sampled by: Trinity Gillary

Date: 10-6-77

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. east of Column Line 301, —— ft. —— of Column Line B

Elev: 110'-6" Field dry den 116.75 Field moist 12.75 % compaction 98.2

Test No. 2 30' ft east of Column Line 301, — ft. — of Column Line B.

Elev: 104'10" Field dry den 106.50 Field moist 15.50 % compaction 96%

Test No. 3 40' ft. east of Column Line 301, — ft. — of Column Line A

Elev: 109 Field dry den 114.60 Field moist 13.00 % compaction 96.8%

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____.

Elev: ________

All above test meet specification requirement.

Test No. ______ and ______ meet specification requirement after recompaction.

Maximun Dry Density: (~~121.0~~) (111.2) PCF

118.4

field compaction

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY
ENGINEERS AND CHEMISTS

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-6-72

Material: Zone III

Sampled by: Timothy Gilbreig

Date: 10-6-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. east of Column Line 31, ------ft. ---- of Column Line E----

Elev: 10.8 Field dry den 117.9 Field moist 14.6 % compaction 99.2%

Test No. 2 35' ft east of Column Line 31, — ft. — of Column Line C.

Elev: 10.8 Field dry den 118.7 Field moist 15.25 % compaction 99.9%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximun Dry Density: (~~121.0~~) (111.2) PCF
118.8

POOR ORIGINAL

1

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-5-72

Material: Zone III

Sampled by: Timothy Mihaney

Date: 10-5-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) ~~(Sand Cone)~~

Test No. 1 55' ft. east of Column Line 301, 10' ft. north of Column Line B

Elev: 107 Field dry den 119.5 Field moist 17.00 % compaction 100.6%

Test No. 2 35' ft east of Column Line 301, — ft. — of Column Line A.

Elev: 105 Field dry den 116.7 Field moist 16.30 % compaction 98.2%

Test No. 3 20' ft. east of Column Line 301, — ft. — of Column Line B

Elev: 109 Field dry den 116.00 Field moist 17.00 % compaction 97.7%

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ________

All above test meet specification requirement.

Test No. ______ and ______ meet specification requirement after recompaction.

Maximum Dry Density: (~~[illegible]~~ 118.8) (111.2) PCF

POOR ORIGINAL

FORM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

REPORT

Date 10-4-72

Material: Zone III

Sampled by: Timothy Gibbons

Date: 10-4-72

Specification: 98% of Maximum Dry Density (Nuclear Guage) (Sand Cone)

Test No. 1 20' ft. east of Column Line 30', 10' ft. south of Column Line D

Elev: 108 Field dry den 116.00 Field moist. 16.00 % compaction 98%

Test No. 2 35' ft east of Column Line 30', — ft. — of Column Line C.

Elev: 107' Field dry den 119.00 Field moist. 14.00 % compaction 100.2%

Test No. 3 ____ ft. ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

Test No. 4 ____ ft ____ of Column Line ____, ____ ft. ____ of Column Line ____

Elev: ____

All above test meet specification requirement.

Test No. ____ and ____ meet specification requirement after recompaction.

Maximum Dry Density: (~~[illegible]~~) (111.2) PCF
118.8

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
INSPECTING ENGINEERS AND CHEMISTS
512 NORTH DELAWARE AVE.
TAMPA, FLORIDA 33606
AREA CODE 813
PHONE: 253-3485

December 7, 1972 PTL Ref: TA-7732

Mr. James Harris
Gilbert Associates
525 Lancaster Avenue
Reading, Pennsylvania 19602

Re: Triaxial Testing
Florida Power Corporation

Dear Mr. Harris:

Enclosed please find the results of triaxial testing conducted by us on Zone III compacted limerock fill. The triaxial sample size was 1.40" Ø x 2.985". Due to the small sample size, the air dried limerock was scalped on a #8 sieve and compacted by a 1/2" Ø kneading tamper in 5 layers. We believe that this will result in a conservative estimate of insitu strength since greater particle interlocking could be expected insitu.

Both drained and undrained tests with pore pressure measurements were run to assure the determination of the proper effective stress failure envelope. Difficulties were experienced with the undrained tests due to the extreme sample rigidity and negative pore pressure change generated by the shear stress.

The tests were run in a Wykeham-Farrance triaxial cell with a dead load lever apparatus used to apply the deviator stress. Pressure measurements were made using a Tyco 200 PSI piezoelectric pressure transducer.

The cell pressure and back pressure were applied through air-water interchange system with the pressure control supplied by Bellofram having a sensitivity of one part in 5000 air regulators. The minimum back pressure was 40 PSI.

Data reduction was done by programs written by PTL for the Wang 600 programmable calculator.

Figure 1 shows the triaxial test densities and their relationship to field density and compaction characteristics of the Zone III limerock. Figure 2 is the stress path results from the drained triaxial tests and Figure 3 the stress path results of undrained triaxial testing. Figures 4 and 5 are the stress-strain data.

The following is a summary by sample number of test conditions and results.

LR-1 Undrained triaxial test with pore pressure measurements

γ_d = 118.9 PCF
ω_m = 10.38%
ω_F = 14.9 %
σ_c = 60 PSI
u_B = 40 PSI
$(\sigma_1 - \sigma_3)_f$ = 12.73 TSF
$\bar{\phi}$ = 45.5°
$\bar{c}$ = 0
ϵ_f = 2.4%

LR-2 Drained triaxial test

γ_d = 113.08 PCF
ω_m = 11.2%
σ_c = 37.3 PSI
u_B = 29.6 PSI
$(\sigma_1 - \sigma_3)_f$ = 2.65 TSF
$\bar{\phi}$ = 45.5°
$\bar{c}$ = 0
ϵ_f = .82%

LR-3 Drained Triaxial Test

γ_d = 11[illegible].4 PCF
ω_m = 11.0%
ω_F = 17.6%
σ_c = 62.8 PSI
u_B = 30.4 PSI
$(\sigma_1 - \sigma_3)_f$ = 10.56 TSF
$\bar{\phi}$ = 45.5°
$\bar{c}$ = 0
ϵ_f = 1.74%

LR-4 Undrained Triaxial Test with Pore Pressure Measurements

γ_d = 115.9 PCF
ω_c = 13.9%
ω_f = 17.1%
σ_c = 61.17 PSI
u_B = 38.25 PSI
$(\sigma_1 - \sigma_3)_f$ = 15.22 TSF
$\bar{\phi}$ = 52.0°
$\bar{c}$ = 0
ϵ_f = 2.48%

Pore water cavitation results too high

LR-5 Unconfined Compression Test

γ_d = 114.6 PCF
$\omega_c = \omega_f$ = 10.1%
σ_c = 0
u_B = 0
$(\sigma_1 - \sigma_3)_f$ = 2.568 TSF
ϵ_f = 1.07%

Where γ_d is the dry unit weight of sample
ω_m is the moding water content
ω_f is the final water content
σ_c is the cell pressure
u_B is the back pressure
$(\sigma_1 - \sigma_3)_f$ is the maximum deviator stress
$\bar{\phi}$ is the effective stress angle of internal friction
$\bar{c}$ is the effective stress cohesion intercept
ϵ_f is the strain at failure

The angle $\bar{\alpha}$ from the stress path drawing is related to $\bar{\phi}$ by the following relationship $\tan\bar{\alpha} = \sin\bar{\phi}$ where $\bar{\phi}$ is the Mohr-Coulomb angle of internal friction used to define shear strength, i.e.

$$\tau = \bar{c} + \bar{\sigma}\tan\bar{\phi}$$

Where $\bar{\sigma}$ is the normal stress on any shear plane.

The test results can be summarized as follows:

$$\gamma_d = 113 \text{ to } 119 \text{ PCF}$$
$$\bar{c} = 0$$
$$\bar{\phi} = 45.5^{\circ}$$

The soil will generate negative pore pressure drops except under very high consolidation stresses when sheared undrained.

Test LR-4 shows pore pressure drops less than should have occurred due to incomplete saturation and should be corrected back to $\bar{\phi} = 35.5^{\circ}$.

We appreciate the opportunity of handling this testing for you and shall be glad to discuss the results with you at any time.

Respectfully submitted,

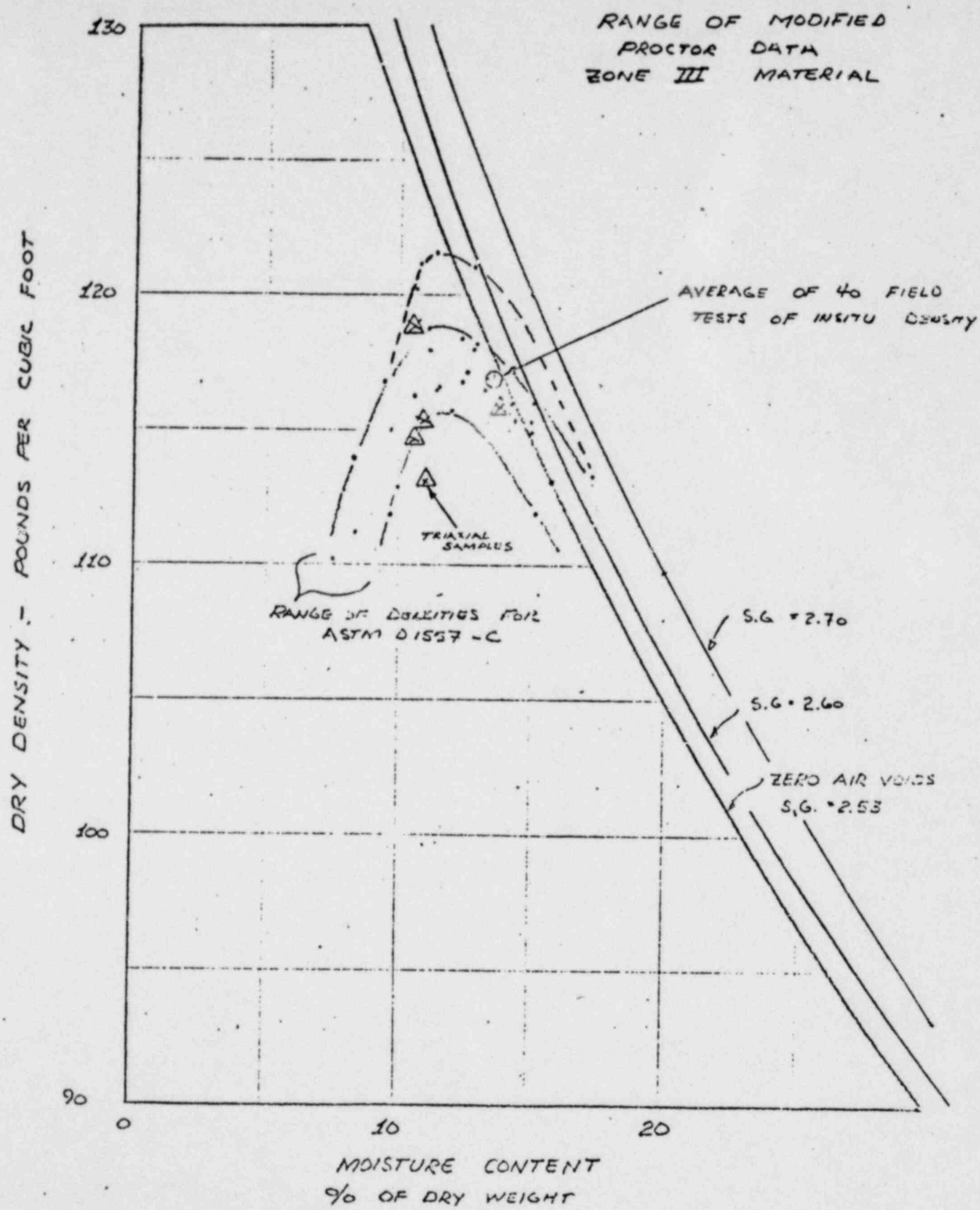
PITTSBURGH TESTING LABORATORY

Ross T. McGillivray

Ross T. McGillivray, P.E.
Tampa District Engineer

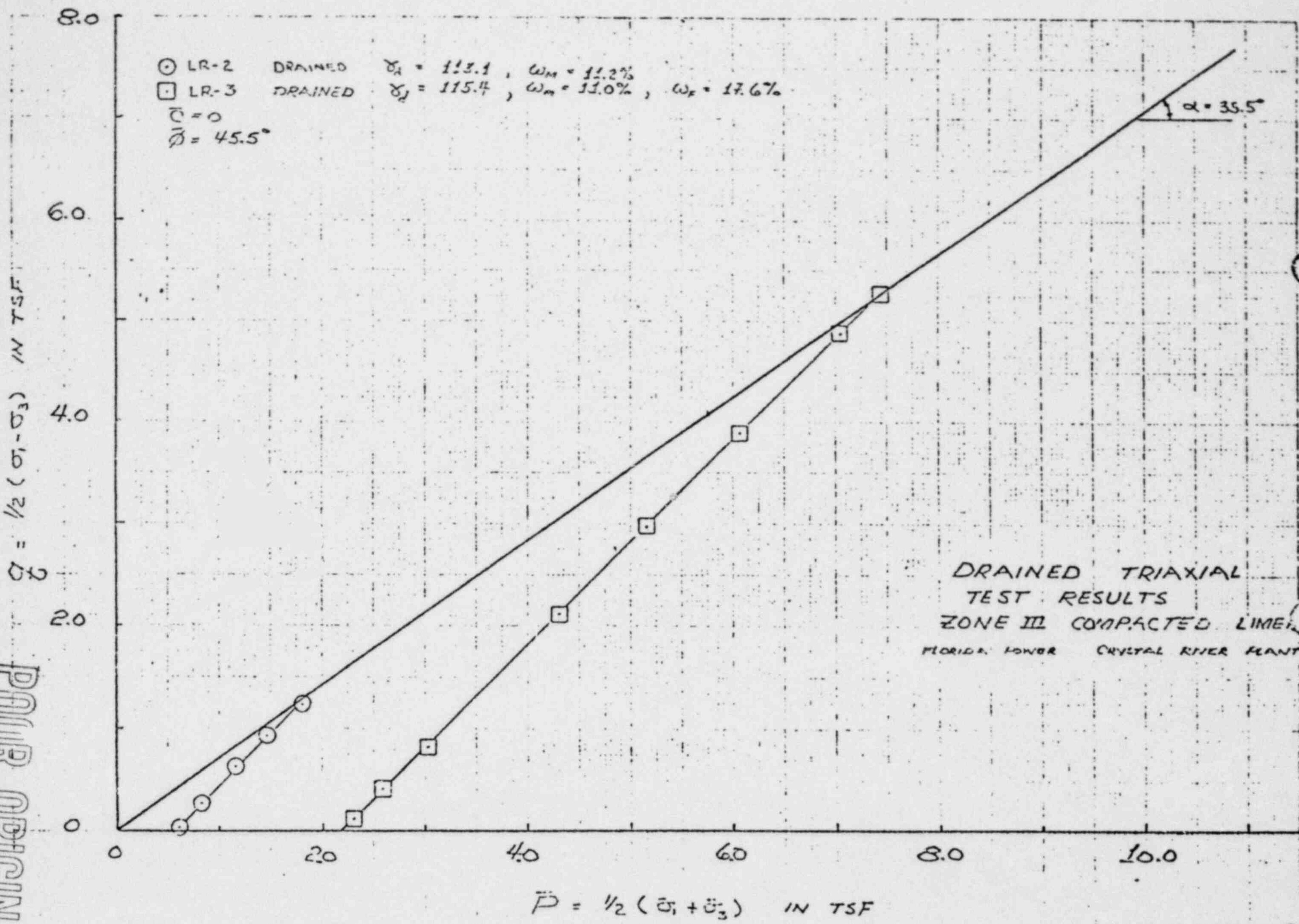
RTM/jhv

FLA POWER CORP, CRYSTAL RIVER, FLORIDA



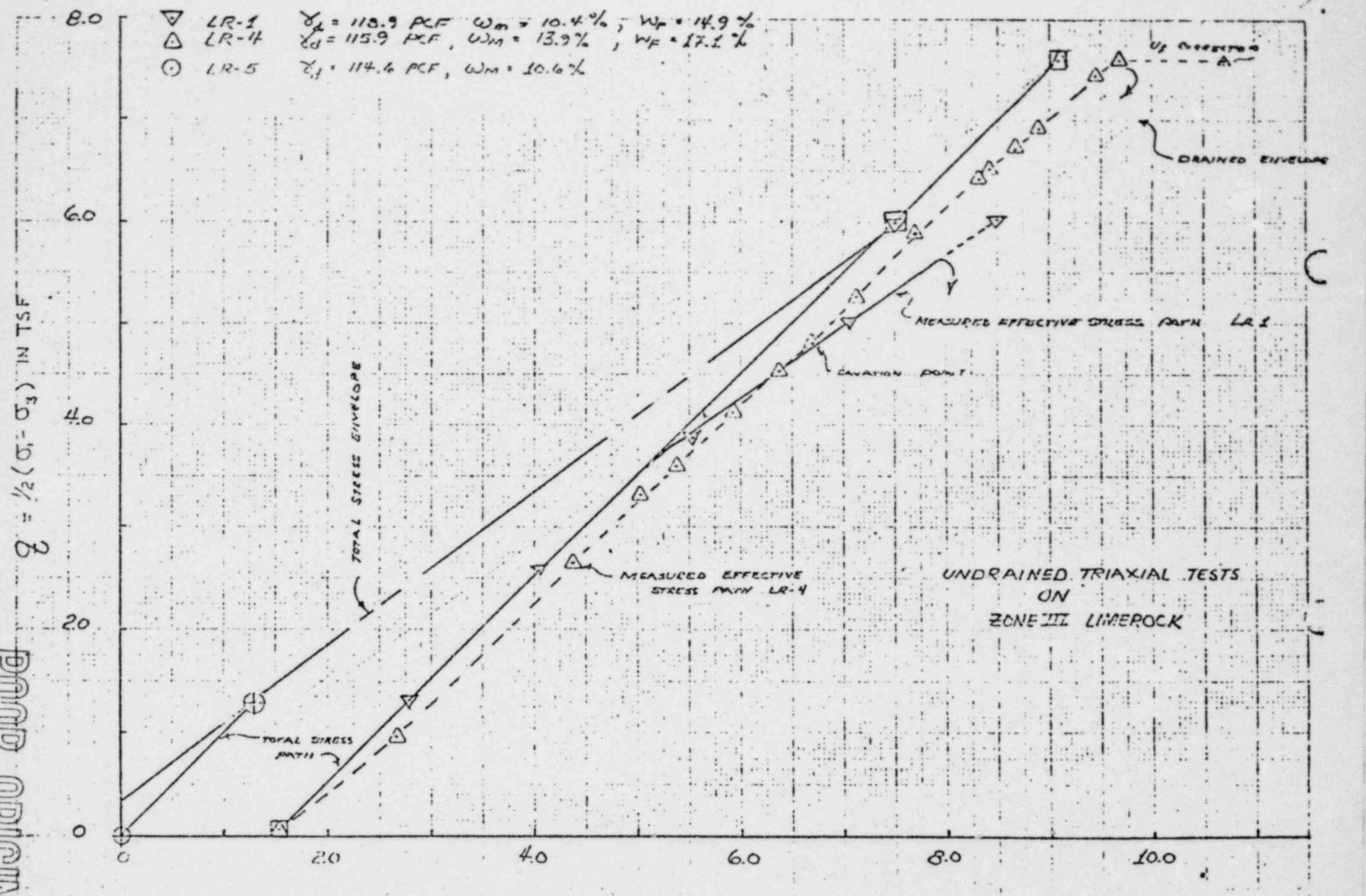
PITTSBURGH TESTING LABORATORY
TAMPA, FLORIDA

FIGURE 1



8.0
6.0
4.0
2.0
0
⊙ LR-2 DRAINED γd = 113.1 , ωm = 11.2%
⊡ LR-3 DRAINED γd = 115.4 , ωm = 11.0% , ωf = 17.6%
c̄ = 0
φ̄ = 45.5°
α = 35.5°
q̄ = ½ (σ̄1 - σ̄3) IN TSF
DRAINED TRIAXIAL
TEST RESULTS
ZONE III COMPACTED LIME
FLORIDA POWER CRYSTAL RIVER PLANT
0
2.0
4.0
6.0
8.0
10.0
P̄ = ½ (σ̄1 + σ̄3) IN TSF

FIGURE 2.



UNDRAINED TRIAXIAL TESTS ON ZONE III LIMEROCK

FIGURE 3

POOR ORIGINAL

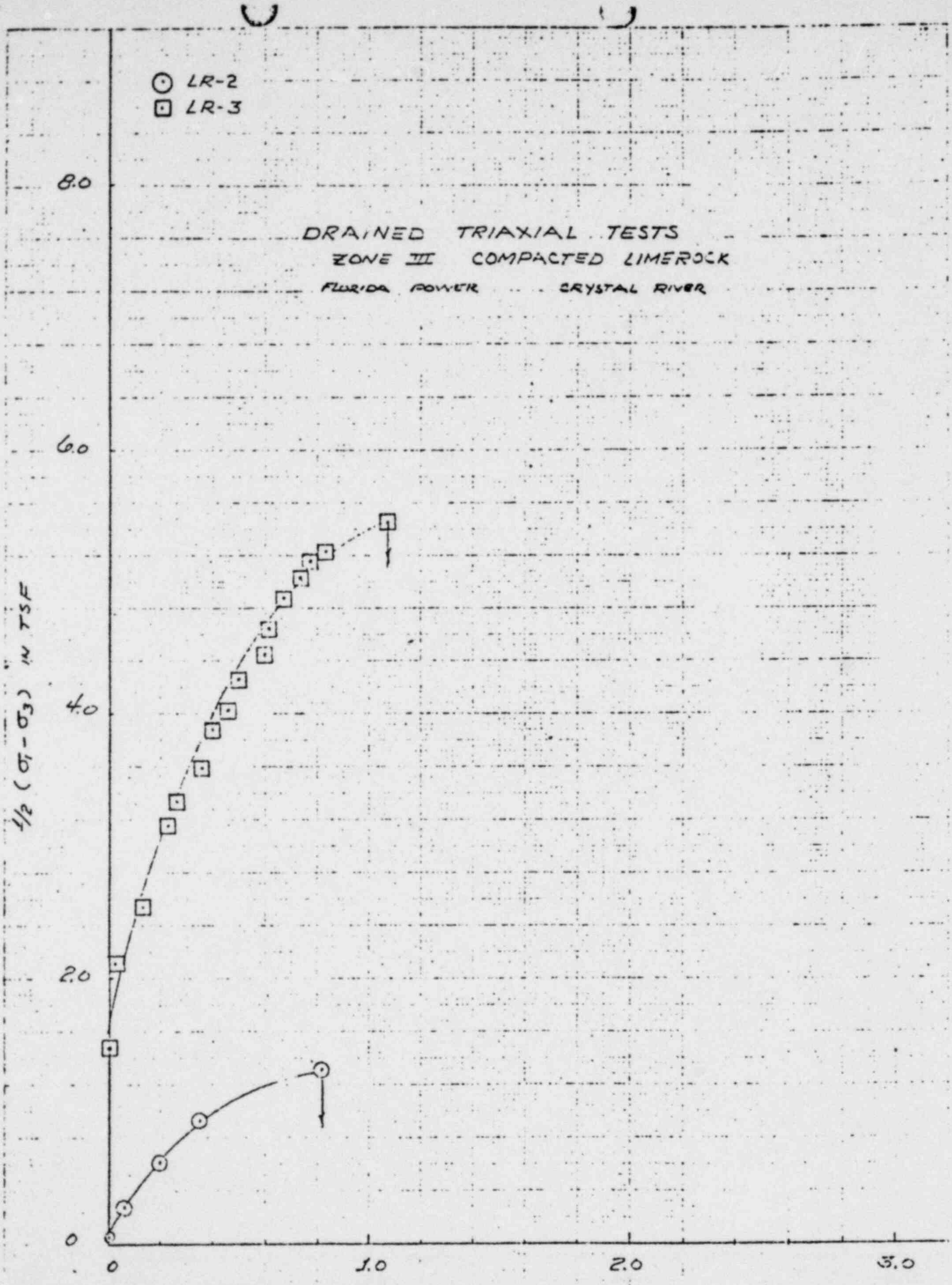
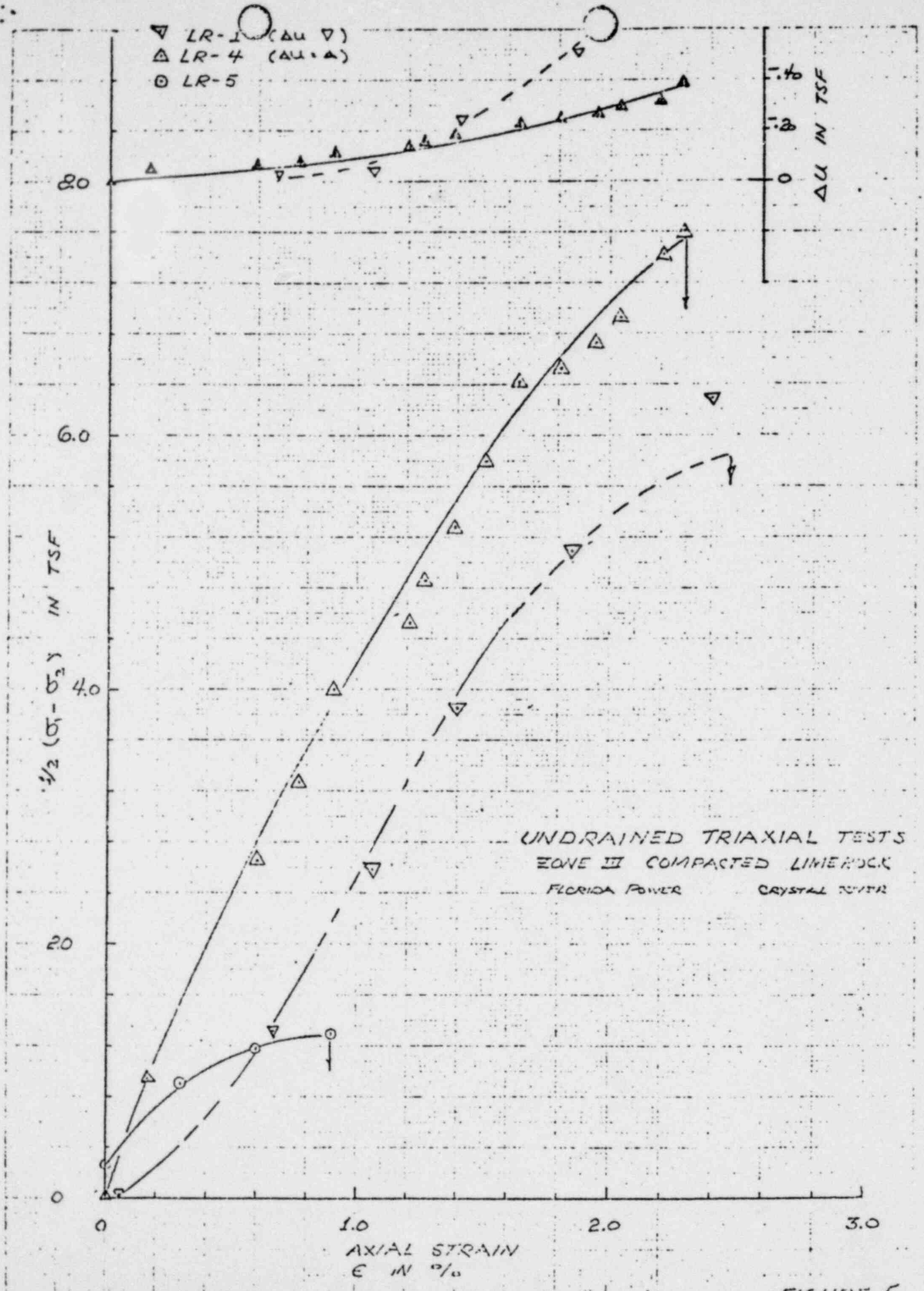


FIGURE 4

POOR ORIGINAL



LR-1 (Δu ▽)
LR-4 (Δu ▲)
LR-5
.40
.20
0
Δu IN TSF
8.0
6.0
4.0
2.0
0
½ (σ₁ - σ₃) IN TSF
0
1.0
2.0
3.0
AXIAL STRAIN
ε IN %
UNDRAINED TRIAXIAL TESTS
ZONE III COMPACTED LIMEROCK
FLORIDA POWER

FIGURE 5

COLLEGE OF ENGINEERING

COASTAL AND OCEANOGRAPHIC ENGINEERING DEPARTMENT
392-1436
AFTER 5:00 P.M. 372-5259
LABORATORY 392-0891

UNIVERSITY OF FLORIDA

GAINESVILLE, FLORIDA 32601
AREA CODE 904 PHONE 392-1436

April 3, 1972

Mr. Joel Caves
Gilbert Associates, Inc.
525 Lancaster Avenue
Reading, Pennsylvania

Dear Joel:

In our recent telephone conversation you requested that I comment on and clarify the types of waves generated in our study for Gilbert Associates, Inc. (GAI) as contrasted to a spectrum of waves. These comments pertain to results presented in our document "Report of Model Tests to Determine Extreme Runup at Florida Power Corporation Crystal River Site", dated April 1969.*

The motion of the wave generator in these tests was essentially periodic, with very small variations due to changes in line voltage, etc. The intent therefore was to generate a wave and runup system which, ideally, would have the same heights and elevations respectively, from one wave to the next. No attempt was made to generate a wave spectrum in which the wave system would comprise more than one fundamental period.

The variations indicated in our report, e.g. in Figure 1 and Table 1 (as indicated by the differences between median and maximum) are primarily due to the nonlinear effects in the runup processes; i.e. even if the waves were exactly repeated, the runup would vary from wave to wave. A secondary component of the variation in runup is due to the variation in the characteristics of the wave generated.

If I can provide any further information relative to our testing program, please let me know.

Sincerely yours,

Bob Dean

R. G. Dean, Chairman
Department of Coastal and
Oceanographic Engineering

RGD:mam

* FSAR Section 2, Reference (13), Report of Model Tests to Determine Extreme Run-up at Florida Power Corporation, Crystal River Site, Department of Coastal and Oceanographic Engineering, Florida Engineering and Industrial Experiment Station, University of Florida, April, 1969.

(Appendix 2C, Crystal River Unit 3, PSAR)

COLLEGE OF ENGINEERING

UNIVERSITY OF FLORIDA

GAINESVILLE, FLORIDA 32601
AREA CODE 904 PHONE 392-1436

COASTAL AND OCEANOGRAPHIC
ENGINEERING LABORATORY
392-1436
WAVE TANK - 392-0891

April 11, 1973

Mr. Heber Newton
Gilbert Associates, Inc.
P. O. Box 1498
Reading, Pennslyvania

Dear Mr. Newton,

This letter is in response to your request that I re-examine the results of our runup test programs carried out in 1967-1969 for the Crystal River site under the sponsorship of Gilbert Associates, Inc. These results are available in Project Reports (1) (2). Specifically, you requested that, on the basis of the available model data, I make a "best judgement" estimate of the median and maximum runup elevations that would result for the following conditions for Profile 5:

TABLE I

Condition	Storm Tide Level* (ft.)	Wave Height (ft.)
1	120.1	17.2
2	121.4	18.2

*Corresponds to a Mean Low Water Datum of 88 ft.

Although the maximum storm tide modeled in our test program was 118 ft. and the maximum corresponding wave height was 12.2 ft., it is possible to make estimates of the maximum runup for the higher storm tides and wave heights as discussed below. It is noted that our Profile 5 was extended above elevation 118.5 to prevent overtopping (see inset in Figure 1).

Discussion

The maximum storm tide elevation tested in our studies for Profile 5 was 118' as shown in the attached Figure 1 (Figure 5 of Reference 2). In this figure the maximum runup curve obtained by the experiments has a convex upward curvature. It therefore appears conservative (i.e. giving too large a runup) to extrapolate to the 120.1 and 121.4 ft. storm tide levels using straight line approximations (dashed lines, Figure 1) of the same slope as those occurring at a storm tide of 118 ft. For the storm tide levels of interest, the resulting runup values are presented in the table below.

TABLE II

Storm Tide Level (ft.)	Median Runup (ft.)	Maximum Runup (ft.)
120.1	124.9	125.9
121.4	126.0	127.1

It may be worthwhile to discuss the wave periods and heights used in the testing program by quoting from Reference 2, page 5: "The wave period (prototype) was selected by first running tests at several tide levels with the estimated most frequent wave period (prototype), 7.7 seconds, and then varying the period in 10% intervals above and below the estimated most frequent value. The period with which the most runup occurred was 70% of the estimated most frequent period or 5.4 seconds. The wave height was chosen by testing wave generator amplitude settings at several tide levels with a period (prototype) of 5.4 seconds. The amplitude setting which resulted in the most runup produced waves in the 10-15 ft. (prototype) height range prior to breaking over the south bank...".

It is concluded therefore that the runup curves in Figure 1 apply for the wave height and period resulting in the highest runup.

Conclusion

The median and maximum runup values corresponding to tide levels of 120.1 and 121.4 ft. are presented in Table II. These values represent my "best judgement" estimates (although believed to be slightly conservative) on the basis of the available model test data.

If you have any questions regarding the basis for obtaining these estimates, please advise me.

Sincerely yours,

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