ROCHESTER GAS AND ELECTRIC CORPORATION • 89 EAST AVENUE, ROCHESTER, N.Y. 14649

JOHN E. MAIER
Vice President

TELEPHONE
AREA CODE 716 546-2700

April 19, 1982

Director of Nuclear Reactor Regulation
Attention: Mr. Dennis M. Crutchfield, Chief
Operating Reactors Branch No. 5
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: SEP Topic II-4.D, Stability of Slopes
R. E. Ginna Nuclear Power Plant
Docket No. 50-244

Dear Mr. Crutchfield:

This letter is in response to your February 19, 1982 letter regarding SEP Topic II-4.D. In your evaluation of this topic, it was noted that the NRC could not conclude that the slopes on the Ginna site are stable.

Although RG&E believes that the slopes are stable, we have performed an evaluation to determine the effects on safety-related structures even if the slopes did fail. As noted in the attachment, the postulated failure of the slopes would not affect any safety-related structures, systems, or components. Thus, stability of slopes as related to the Ginna site is not considered a safety concern.

Very truly yours,

John E. Maier

John E. Maier

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Attachment: Calculation of Internal Friction Angle, Cohesion and Travel Distance of a Mud Wave Generated From Slope Failure for SEP Topic II-4.D, Stability of Slopes, R. E. Ginna

Two onsite slopes at the R. E. Ginna Nuclear Power Plant have been identified whose failure may be a safety concern. Boring #1 and Boring #3 of the PSAR subsurface investigation have been accepted to represent the subsurface soil conditions of these slopes. The stratum that is the focus of primary concern has been designated as the CL class material. The CL material taken from Boring #101 was subjected to two Triaxial Compression Tests to determine the properties of the CL material. This mode of testing produces a failure plane that more closely resembles an actual failure plane than the results of the direct shear tests conducted on the CL material obtained from Boring #1 and #3. The results of the Triaxial Compression Tests can be seen on the copy of Boring Log #101 attached to this submittal. A copy of the method of performing the Triaxial Compression Test is also attached.

In the first test, a constant normal pressure, σ, of 1200 pounds per square foot was applied until failure was achieved. The shearing strength was calculated to be 600 pounds per square foot for the CL material during this test. In the second test, the constant normal pressure σ was increased to 1800 pounds per square foot, when failure occurred. The shearing strength of this sample also was calculated to be 600 pounds per square foot.

The shearing resistance S of the soil subjected to the Triaxial Compression Test is equal to the constant of cohesion C plus the product of the normal stress on the surface of sliding, σ, and the tangent of the angle of internal friction ϕ. In other words, $S = C + \sigma \tan \phi$, known as Coulomb's equation. The log of Boring #101 shows us that although the sample is subjected to two different normal stresses, the calculated shearing resistance of the soil in both tests is the same. The shearing resistance S of this material can only be calculated after ϕ and C have been experimentally determined. Since the values of ϕ and C were not stated on the Boring Logs, the actual slope stability due to the onsite CL material cannot be derived.

However, possible combinations of the cohesion C and the internal angle of friction ϕ that correspond to a factor of safety along different possible failure surfaces can be evaluated. For instance, it can be shown that if the shearing resistance of the material can be fully attributed to cohesion, that is if the soil is a very soft clay, then the internal angle of friction equals zero. In this case $600 = C + \tan 0°$ or $C = 600$ psf. On the other hand, if the material is cohesionless, that is, a dry

sandy material with C = 0, then 600 = 0 + 1800 x tan ϕ or ϕ = 18.4°. These two examples define the extreme limit combinations of cohesion C and the internal angle of friction ϕ.

A relationship between the critical height h_{CR}' of different slopes and the cohesion C of clay was developed by Gregory P. Tschebetarioff in Foundations, Retaining and Earth Structures © 1973, McGraw-Hill Inc. This relationship has been developed into charts from which one can estimate the length of travel of a possible mud slide which may occur in the case of slope failure. A copy of these charts is included as Figure 1. The full range of the possible developed cohesion C was evaluated to estimate the length of travel that each failure would incur for the corresponding calculated internal friction angles. The results of these calculations for both triaxial tests are included with this evaluation.

These calculations show that even in a worse case slope failure mode, the maximum distance traveled by the generated mud slide will be only 18.0 feet. Slope failure on the eastern slope will generate a mud wave that will propogate no closer than 95.0 feet from the east side of the screenhouse. Slope failure on the western slope will generate a mud wave that will propogate no closer than 210.0 from the northwest corner of the turbine building. Based upon this evaluation, we conclude that the onsite slopes at the R. E. Ginna plant present no threat to the safe operation of the facility.

Boring #101 - Triaxial Compression Test 26 ft. deep

$S = 600$ psf $\quad W = 22.6\%$

$\sigma = 1800$ psf $\quad \gamma_D = 106$ pcf

$\gamma_T = 137.0$ pcf

Angle of repose, $\phi' = 7.59°$
Normal stress, $\sigma' = 10$ ft. 137.0 pcf = 1370 psf
Effective cohesion, C' = Apparent cohesion, C

Shearing strength $S = C + \sigma \tan \phi$ or $C = S - \sigma \tan \phi$
Shearing stress $t = C' + \sigma' \tan \phi'$
Safety factor, $S.F. = \frac{s}{t} = \frac{C + \sigma \tan \phi}{C' + \sigma' \tan \phi'}$

$\phi°$	C psf	S.F.	Distance Traveled* Upon Failure
26.6°	---	---	---
24°	---	---	---
21°	---	---	---
18.4°	0	3.28	0'
17°	50	2.58	4.6
15°	118	2.00	9.2
12°	217	1.50	18.0'
10°	283	1.29	16.0'
8°	347	1.13	8.8'
5°	443	0.96	0
0°	600	0.77	0

*Figure 7-6a II, pg. 284, Foundations Retaining and Earth Structures, Gregory P. Tschebetarioff © 1973 McGraw-Hill.

Boring #101 - Triaxial Compression Test 20 ft. deep

S = 600 psf W = 24.5%

σ = 1200 psf γ_D = 102 pcf

γ_T = 135.1 pcf

Angle of repose, $\phi' = 7.59°$
Normal stress, σ' = 10 ft. 135.1 pcf = 1351 psf
Effective cohesion, C' = Apparent cohesion, C

Shearing strength $S = C + \sigma \tan \phi$ or $C = S - \sigma \tan \phi$
Shearing stress $t = C' + \sigma' \tan \phi'$

Safety factor, $S.F. = \frac{s}{t} = \frac{C + \sigma \tan \phi}{C' + \sigma' \tan \phi'}$

$\phi°$	C	S.F.	Distance Traveled* Upon Failure
26.6°	0	3.76	0
24°	66	2.44	3'
21°	139	1.88	7.4
18°	210	1.54	14.8'
17°	233	1.45	16'
15°	278	1.31	11.1'
12°	345	1.14	9.8'
7°	453	0.95	0
4°	516	0.86	0
0°	600	0.77	0

*Figure 7-6a II, pg. 284, Foundations Retaining and Earth Structures, Gregory P. Tschebetarioff © 1973 McGraw-Hill.

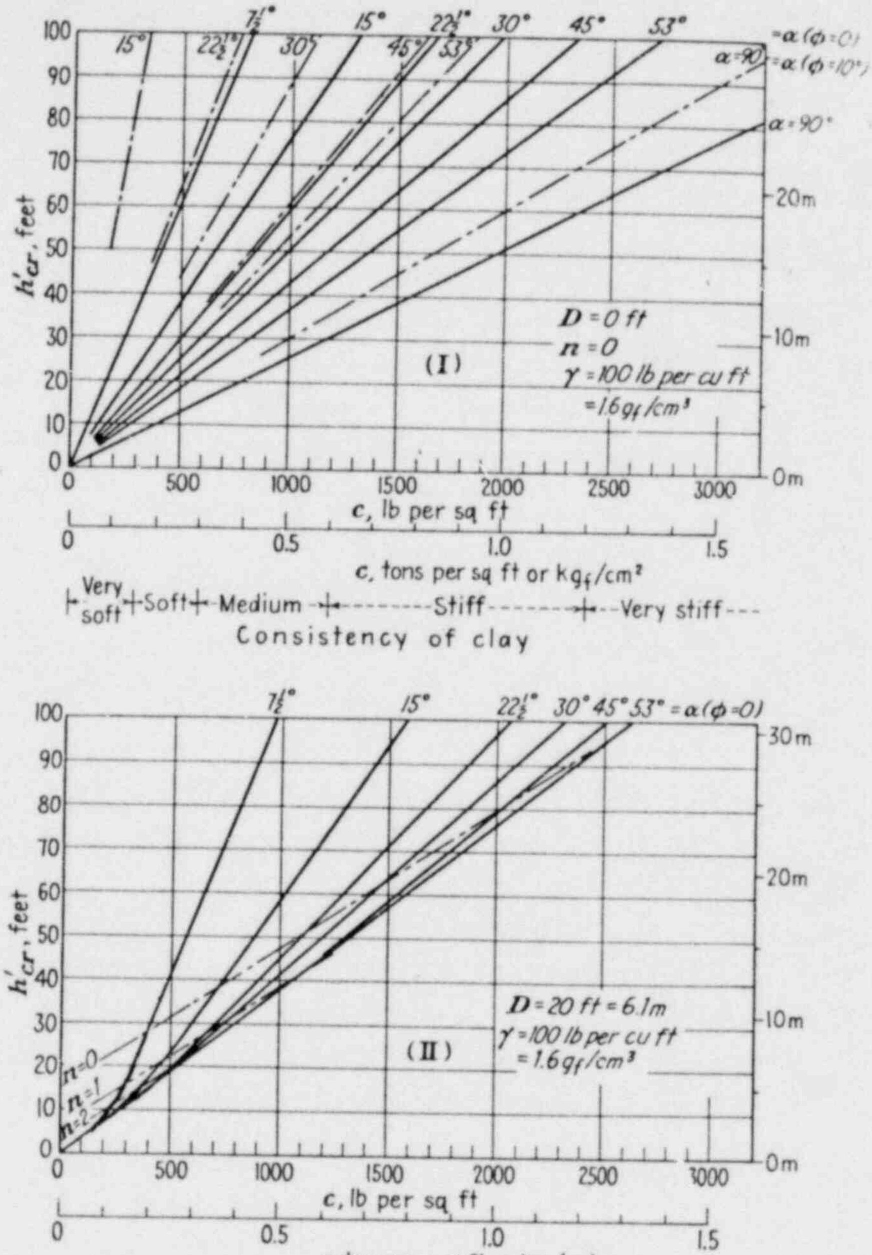


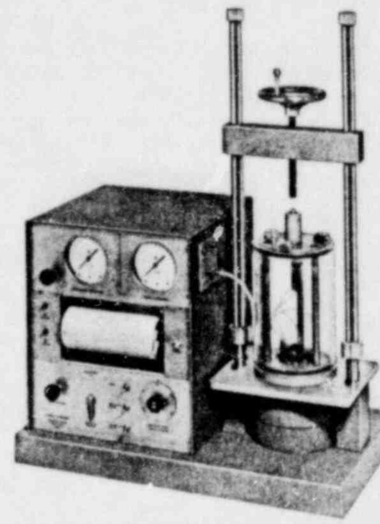
Fig. 7-6a. Four diagrams illustrating the relationship between the critical height h_{cr}' of different slopes and the cohesion c of a clay weighing 100 lb/ft³ (1.6 gf/cm³). (*Developed from Taylor, Ref. 330.*)

Figure 1

Methods of Performing Unconfined Compression and Triaxial Compression Tests

The shearing strengths of soils are determined from the results of unconfined compression and triaxial compression tests. In triaxial compression tests the test method and the magnitude of the confining pressure are chosen to simulate anticipated field conditions.

Unconfined compression and triaxial compression tests are performed on undisturbed or remolded samples of soil approximately six inches in length and two and one-half inches in diameter. The tests are run either strain-controlled or stress-controlled. In a strain-controlled test the sample is subjected to a constant rate of deflection and the resulting stresses are recorded. In a stress-controlled test the sample is subjected to equal increments of load with each increment being maintained until an equilibrium condition with respect to strain is achieved.



TRIAXIAL COMPRESSION TEST UNIT

Yield, peak, or ultimate stresses are determined from the stress-strain plot for each sample and the principal stresses are evaluated. The principal stresses are plotted on a Mohr's circle diagram to determine the shearing strength of the soil type being tested.

Unconfined compression tests can be performed only on samples with sufficient cohesion so that the soil will stand as an unsupported cylinder. These tests may be run at natural moisture content or on artificially saturated soils.

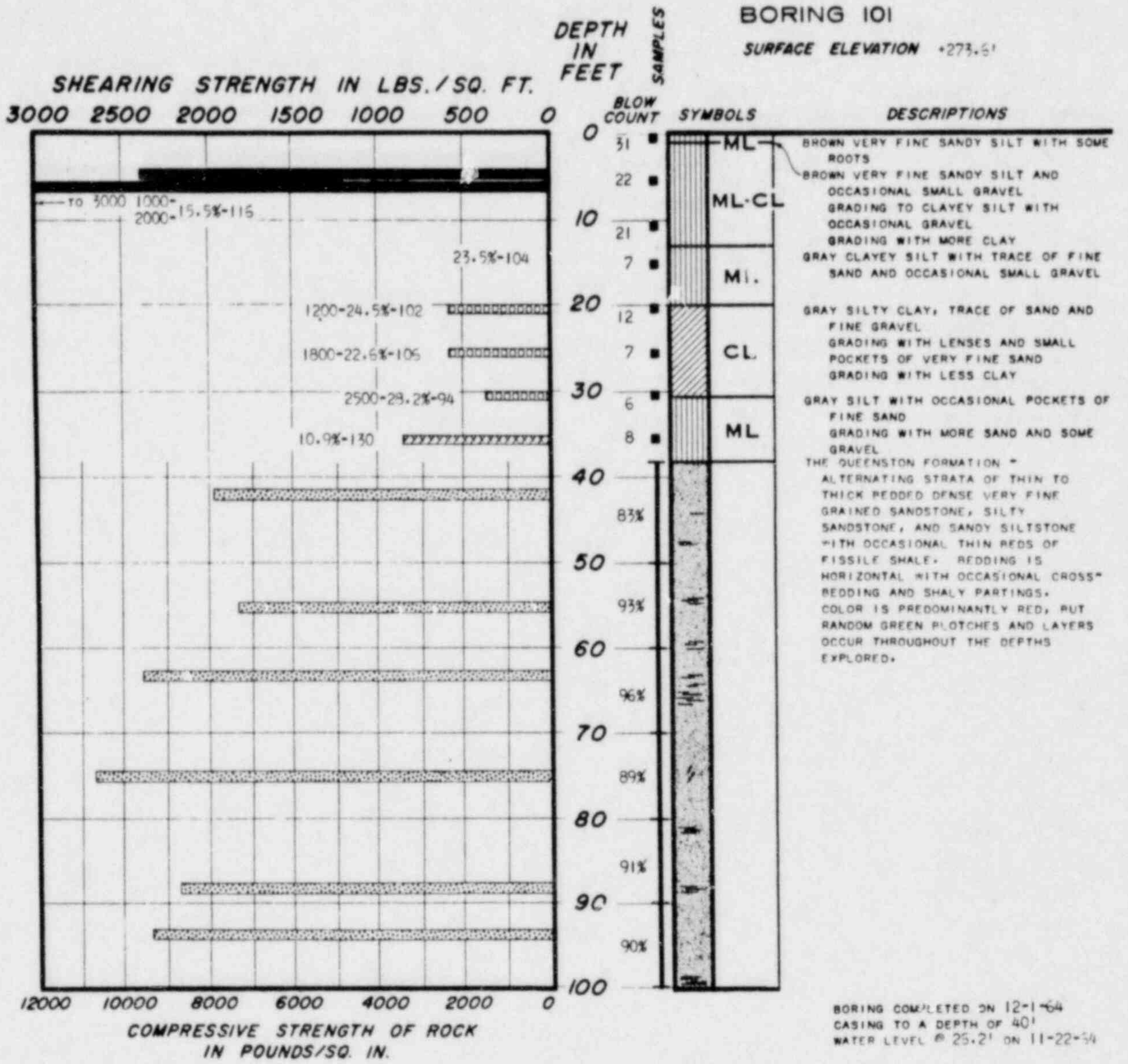
In a triaxial compression test the sample is encased in a rubber membrane, placed in a test chamber, and subjected to a confining pressure throughout the duration of the test. Normally, this confining pressure is maintained at a constant level, although for special tests it may be varied in relation to the measured stresses. Triaxial compression tests may be run on soils at field moisture content or on artificially saturated samples. The tests are performed in one of the following ways:

Unconsolidated-undrained: The confining pressure is imposed on the sample at the start of the test. No drainage is permitted and the stresses which are measured represent the sum of the intergranular stresses and pore water pressures.

Consolidated-undrained: The sample is allowed to consolidate fully under the applied confining pressure prior to the start of the test. The volume change is determined by measuring the water and/or air expelled during consolidation. No drainage is permitted during the test and the stresses which are measured are the same as for the unconsolidated-undrained test.

Drained: The intergranular stresses in a sample may be measured by performing a drained, or slow, test. In this test the sample is fully saturated and consolidated prior to the start of the test. During the test, drainage is permitted and the test is performed at a slow enough rate to prevent the buildup of pore water pressures. The resulting stresses which are measured represent only the intergranular stresses. These tests are usually performed on samples of generally non-cohesive soils, although the test procedure is applicable to cohesive soils if a sufficiently slow test rate is used.

An alternate means of obtaining the data resulting from the drained test is to perform an undrained test in which special equipment is used to measure the pore water pressures. The differences between the total stresses and the pore water pressures measured are the intergranular stresses.



BORING COMPLETED ON 12-1-64
CASING TO A DEPTH OF 40'
WATER LEVEL @ 25.2' ON 11-22-64

LOG OF BORING

KEY TO ROCK SYMBOLS:

- QUEENSTON FORMATION
- QUEENSTON FORMATION WITH SHALY PHASES
- QUEENSTON FORMATION WITH VERTICAL FRACTURES

DAMES & MOORE

4774-001 BY ADB DATE 12-10-64 CHECKED BY DATE