September 7, 1982

Docket No. 50-29

LS05-82-09-027

LICENSEE: YANKEE ATOMIC ELECTRIC COMPANY (YAECO)

FACILITY: YANKEE NUCLEAR POWER STATION (YANKEE)

SUBJECT: SUMMARY OF JULY 27-29, 1982 MEETING CONCERNING SEP TOPICS FOR YANKEE

On July 27 - 29, 1982, representatives of the NRC staff met with representatives of YAECO and New England Power Company to tour the Yankee plant site, the Harriman Dam, and the Sherman Dam , and to review documentation concerning the stability of slopes (SEP Topic II-4.D) and the settlement of foundations (Topic II-4.F) for Yankee. The staff toured the plant site and the two dams on July 27, and met with Yankee representatives on July 28 and 29 at the licensees' headquarters in Framingham, Massachusetts. The meetings consisted of technical discussions concerning the properties of the soils in and around the Yankee site. At the end of the meeting the 1 licensee agreed to provide additional information regarding the soil which was not available for the meeting. That information was subsequently transmitted to the staff and is enclosed.

Original signed by

Ralph Caruso, Project Manager
Operating Reactors Branch #5
Division of Licensing

Enclosure:
As stated

SE01

DSU USE(11)

cc w/enclosure:
See next page

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PDR ADOCK 05000029
P PDR

OFFICE	DL: ORB #5	DL: ORB #5					
SURNAME	RCaruso:cc	DCrutchfield					
DATE	9/3/82	9/7/82					

cc
Mr. James E. Tribble, President
Yankee Atomic Electric Company
25 Research Drive
Westborough, Massachusetts 01581

Chairman
Board of Selectmen
Town of Rowe
Rowe, Massachusetts 01367

Energy Facilities Siting Council
14th Floor
One Ashburton Place
Boston, Massachusetts 02108

U. S. Environmental Protection
Agency
Region I Office
ATTN: Regional Radiation Representative
JFK Federal Building
Boston, Massachusetts 02203

Resident Inspector
Yankee Rowe Nuclear Power Station
c/o U.S. NRC
Post Office Box 28
Monroe Bridge, Massachusetts 01350

Ronald C. Haynes, Regional Administrator
Nuclear Regulatory Commission, Region I
631 Park Avenue
King of Prussia, Pennsylvania 19406

D. Gupta 8/19/8?

INFORMATION ON THE ROCK KNOLL

The following is provided as further information concerning the small wooded knoll which lies directly east of the plant. The information and figures were obtained from a Weston Geophysical report titled, "Geology and Seismology, Yankee Rowe Nuclear Power Plant", dated January 29, 1979.

Figure G2-1B describes the surficial geology of the knoll as a thin ground moraine with frequent bedrock outcrops. Figure G2-2A depicts the bedrock geology of the knoll as a well-bedded albite gneiss. The bedrock of the knoll is also inferred from geologic profile C-C' on Figure G2-4 which terminates at the northwest extent of the knoll as shown on Figure G2-3A. Again, an albite gneiss is shown to lie at or slightly below the surface underlying a thin deposit of moraine.

In conclusion, the wooded knoll directly east of the plant consists predominantly of an albite gneiss overlain by a thin deposit of ground moraine.

GEOLOGY AND SEISMOLOGY

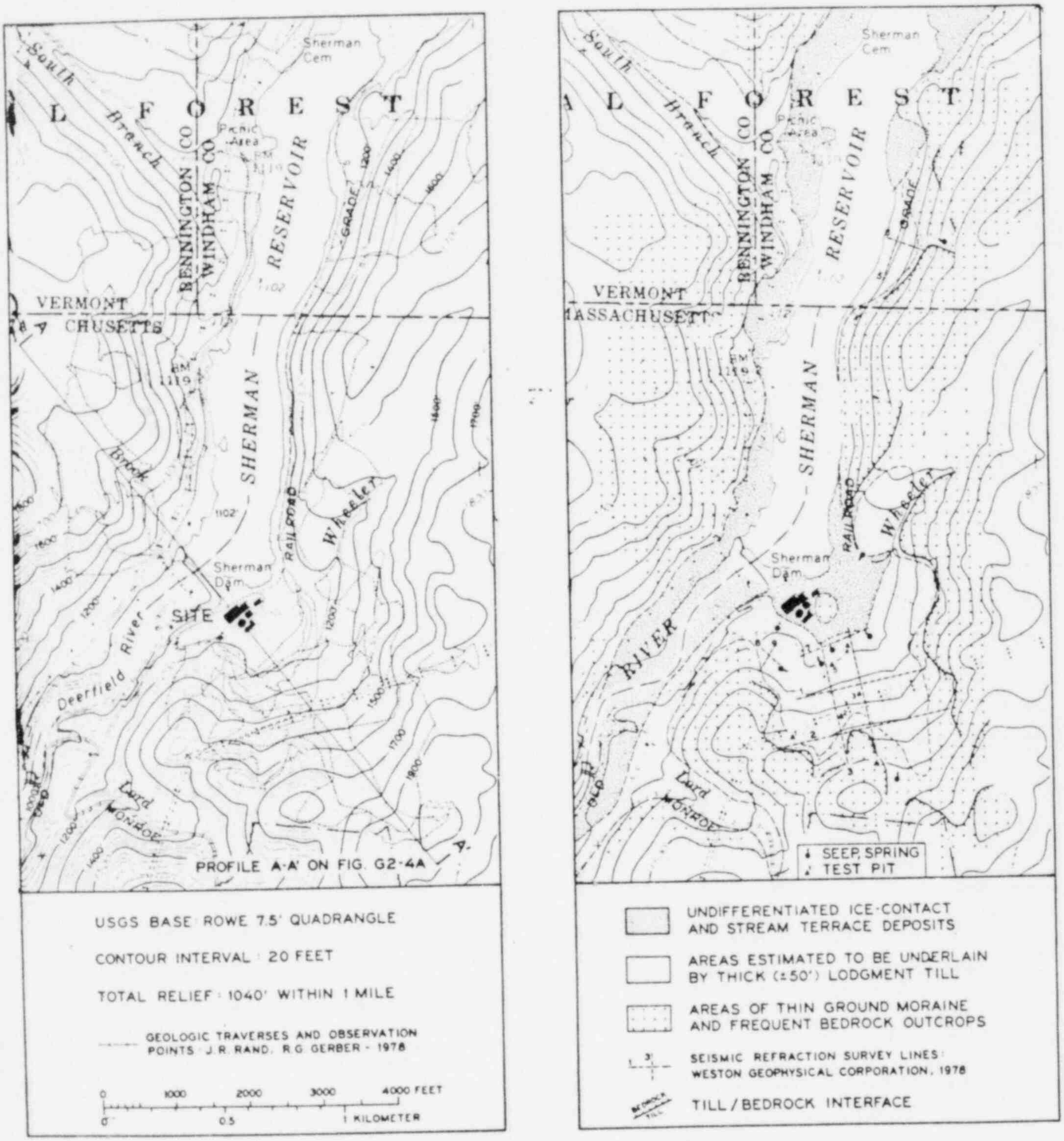
YANKEE ROWE NUCLEAR POWER PLANT

prepared for

YANKEE ATOMIC ELECTRIC COMPANY

January 29, 1979

Weston Geophysical
CORPORATION



A. TOPOGRAPHIC MAP - SITE LOCALE

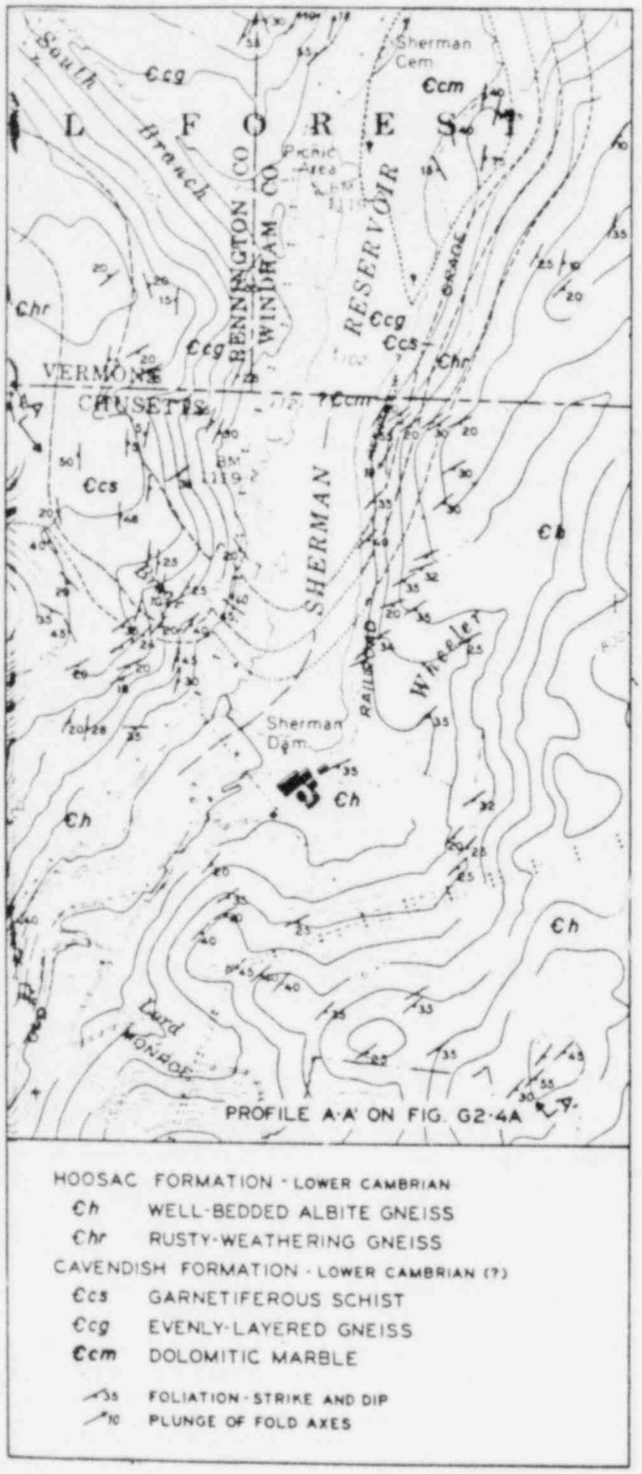
B. SURFICIAL GEOLOGY - SITE LOCALE

FIGURE G2-1

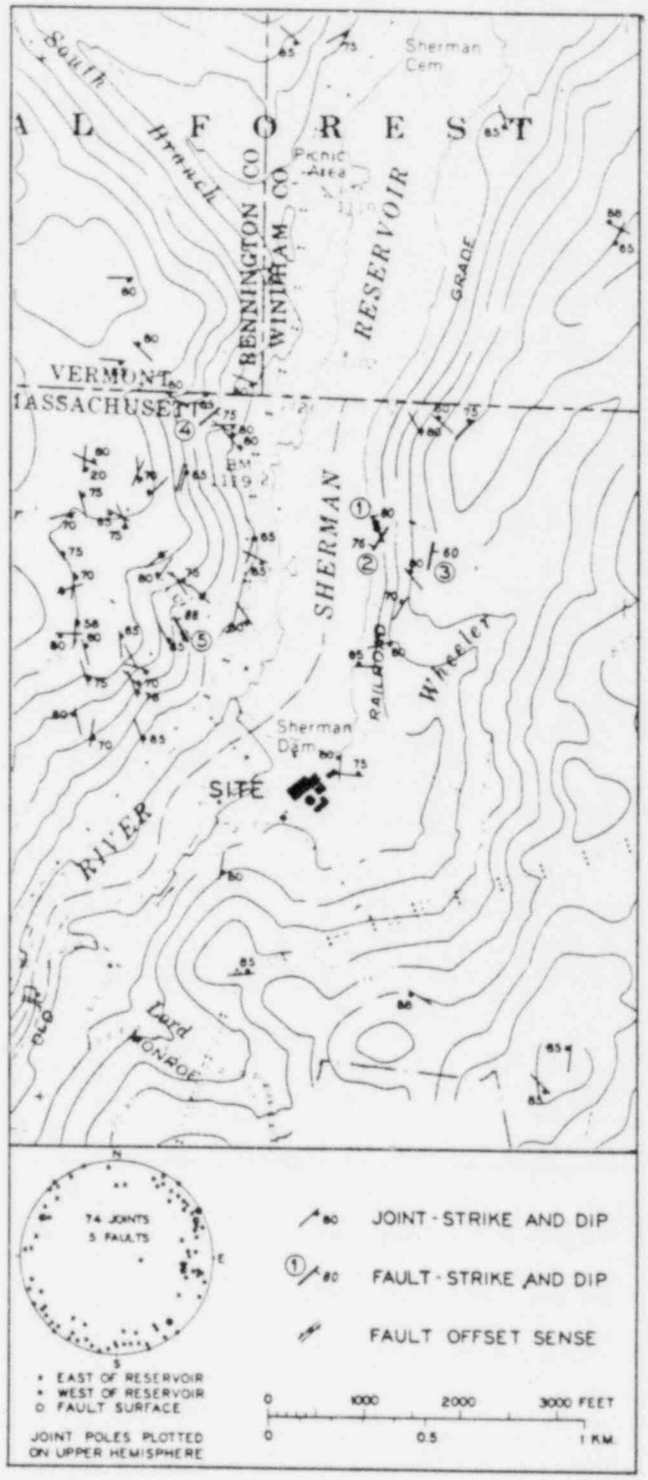
SITE LOCALE

TOPOGRAPHY
and
SURFICIAL GEOLOGY

YANKEE NUCLEAR POWER STATION
Rowe, Massachusetts



A. BEDROCK GEOLOGY - SITE LOCALE



B. BEDROCK FRACTURES - SITE LOCALE

FIGURE G2-2

SITE LOCALE

BEDROCK STRATIGRAPHY
and
STRUCTURE

YANKEE NUCLEAR POWER STATION
Rowe, Massachusetts

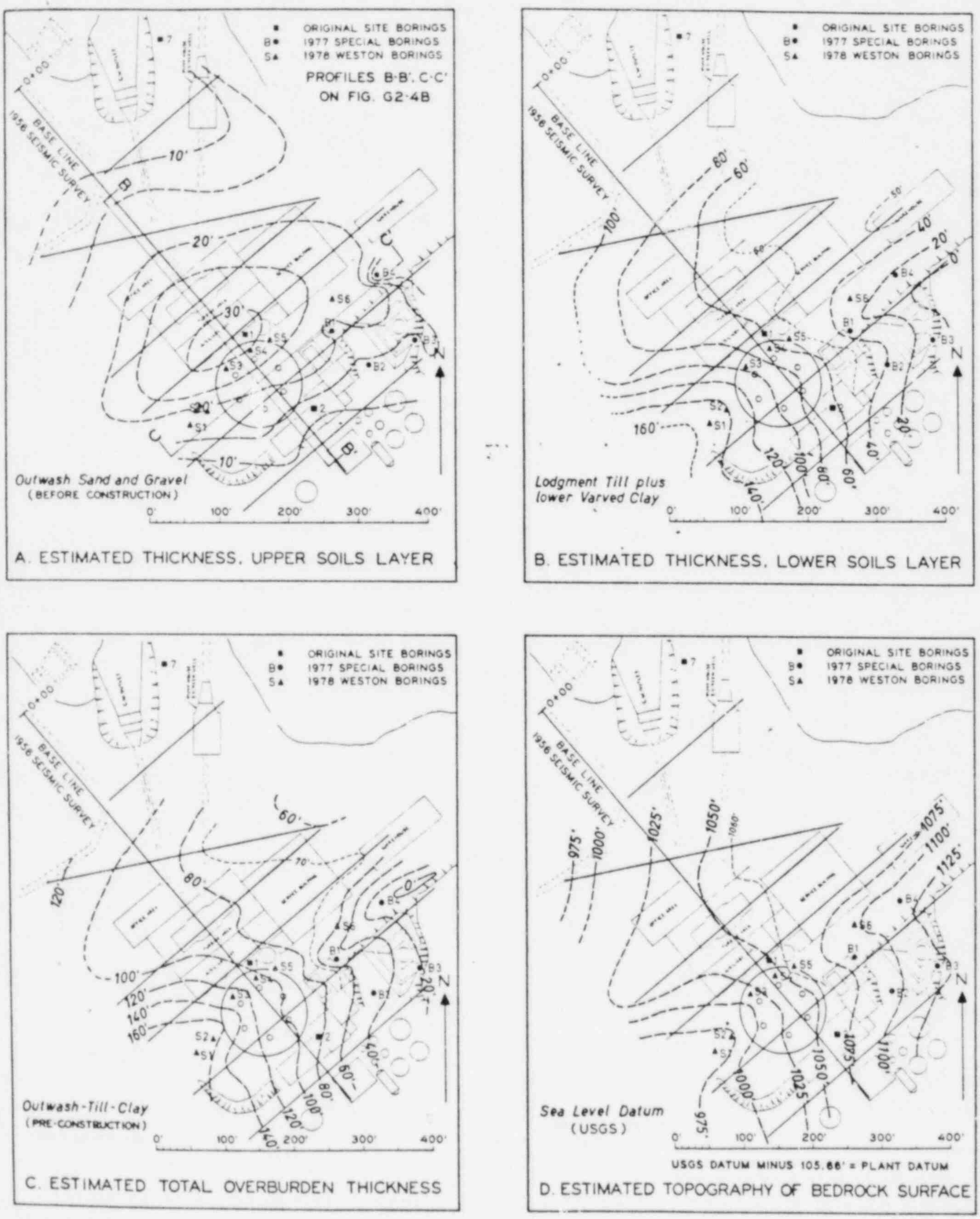
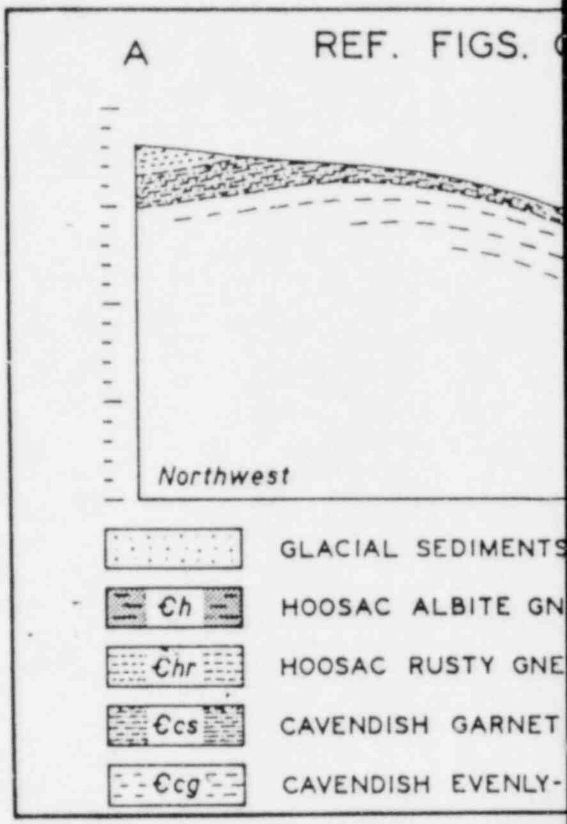
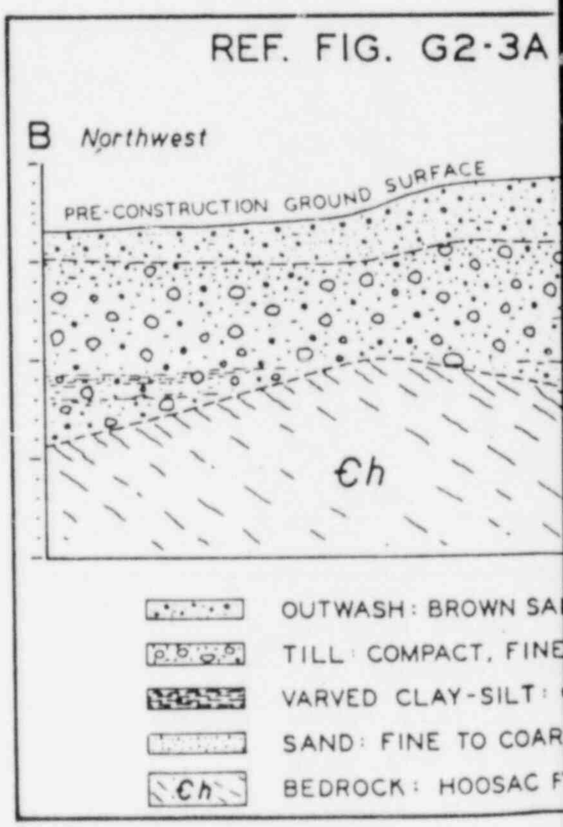


FIGURE G2-3

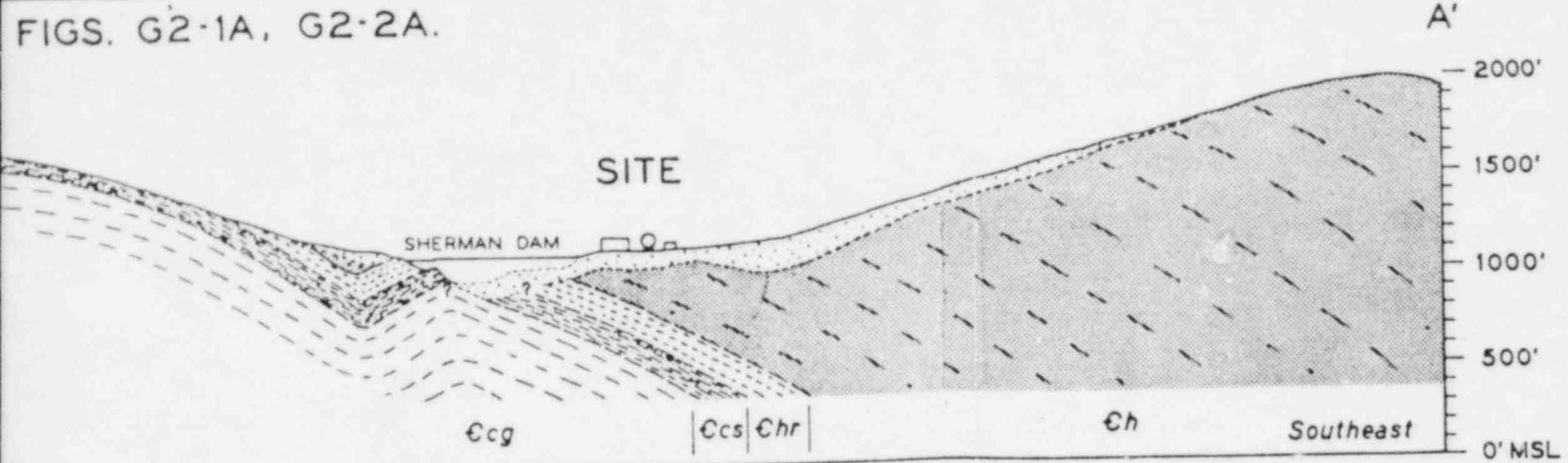
YANKEE NUCLEAR POWER STATION
Rowe, Massachusetts

SITE SOILS LAYERS
and
BEDROCK TOPOGRAPHY


REF. FIGS.
A
Northwest
GLACIAL SEDIMENTS
Ch
HOOSAC ALBITE GN
Chr
HOOSAC RUSTY GNE
Ccs
CAVENDISH GARNET
Ccg
CAVENDISH EVENLY-


REF. FIG. G2-3A
B
Northwest
PRE-CONSTRUCTION GROUND SURFACE
Ch
OUTWASH: BROWN SA
TILL: COMPACT, FINE
VARVED CLAY-SILT:
SAND: FINE TO COAR
Ch
BEDROCK: HOOSAC F

FIGS. G2-1A, G2-2A.



SEDIMENTS

ALBITE GNEISS

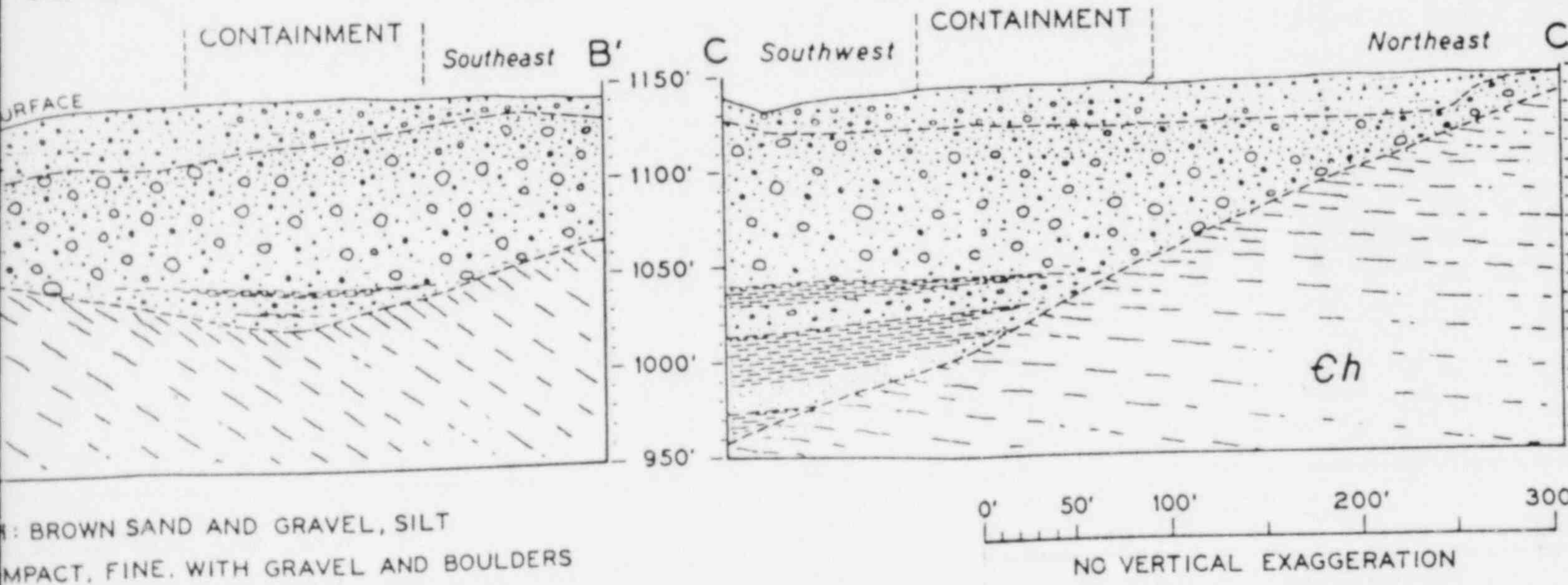
RUSTY GNEISS

H GARNET SCHIST

H EVENLY-LAYERED GNEISS

A. GEOLOGIC PROFILE - SITE LOCALE

G2-3A



: BROWN SAND AND GRAVEL, SILT

MPACT, FINE, WITH GRAVEL AND BOULDERS

CLAY-SILT: COMPACT, MINOR SAND

INE TO COARSE, VERY COMPACT

: HOOSAC FORMATION, ALBITE GNEISS

B. GEOLOGIC PROFILES - ON-SITE

FIGURE G2-4
SITE LOCALE & ON-SIT

GEOLOGIC PROFILES

YANKEE NUCLEAR POWER STATION
Rowe, Massachusetts

GEOLOGIC PROFILE

Geologic profiles for the Yankee site are depicted in Figure G2-4 of Weston Geophysicals' report titled, "Geology and Seismology", dated January 29, 1979.

Profile C-C' is presented here, at an expanded scale, showing the foundation footings for the vapor container. The location of profile C-C' is shown in Figure A and the geologic profile with containment foundation in Figure B. This profile was developed from information obtained in the 1956 seismic refraction profiles, eight borings put down at the site in 1956 and 1977, six borings put down in 1978, and Yankee Drawing 9699-FC-59A, Foundation Details, Vapor Container.

As shown in Figure B, the foundation for the vapor container is founded on very dense glacial till as are other plant structures.

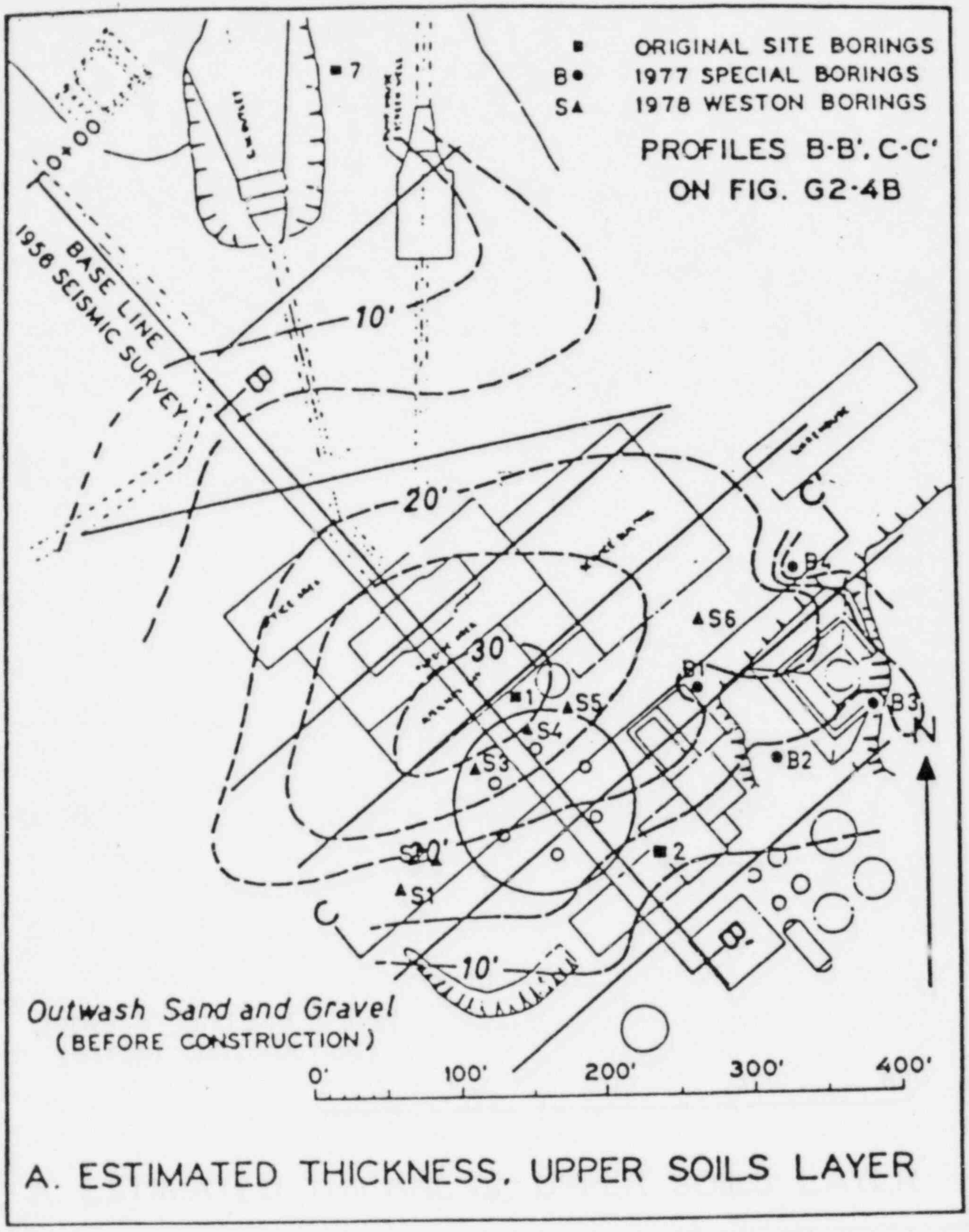


FIGURE A

YANKEE ATOMIC ELECTRIC COMPANY - PLAN VIEW

GEOLOGIC PROFILE LOCATION

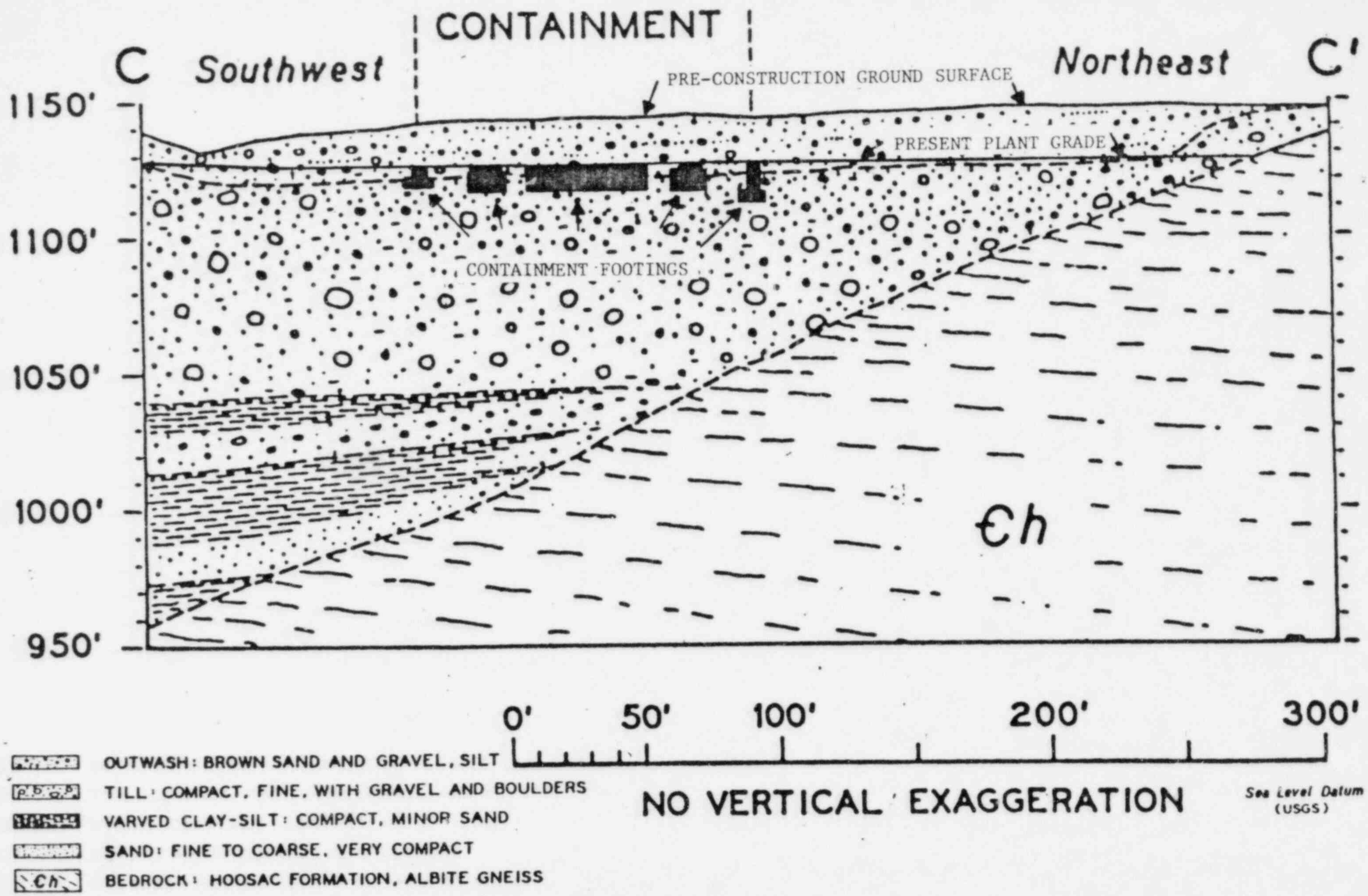


FIGURE B

YANKEE ATOMIC ELECTRIC COMPANY - GEOLOGIC PROFILE

DYNAMIC STABILITY OF NATURAL SLOPES

An approximate dynamic assessment of the natural slopes to the south and east of the plant was performed to further document our conclusion that natural slope stability is not a safety concern at the Yankee plant.

Liquefaction of the glacial till slope materials is not considered possible under dynamic loading conditions because the lodgement till exhibits dilatant behavior in shear. Therefore, the Newmark sliding block analysis [1,2] is an appropriate method of analysis since the till would not undergo any significant loss of strength or pore pressure development during cyclic loading.

The use of the Newmark method inherently implies that the criterion of performance of the slopes be based on the magnitude of the predicted permanent displacements as opposed to the concept of a factor of safety based on limit equilibrium principles.

The same slopes that were analyzed for static stability were again used for this dynamic assessment. Information concerning the static stability of the slopes was previously transmitted and discussed with the NRC.

The results from the Newmark sliding block analysis show that for both the Yankee composite and NRC recommended spectras the permanent displacements of the slopes would be negligible during an earthquake and will not have any adverse impact on the plant.

For completeness, the calculation package is included.

Based on the results from the Newmark sliding block analysis in conjunction with the previously transmitted static stability evaluation, it is concluded that the natural slopes around the plant are not susceptible to catastrophic failure and therefore pose no threat to the Yankee plant.

REFERENCES

1. "Effects of Earthquakes on Dams and Embankments", by N. M. Newmark, the Fifth Rankine Lecture, 1965.

2. "Earthquake Resistance of Earth and Rock-Fill Dams, Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis", Miscellaneous Paper S-71-17, Report No. 5, U.S. Army Corps of Engineers, by Franklin and Chang, 1977.

8/6/82
1/17

Approximate Dynamic Assessment of Slopes surrounding the Yankee plant

goal

An approximate dynamic assessment will be performed for SEP slope stability topic. The same slopes (i.e. A + B) that were used for the static analysis will be used.

To accomplish this task permanent displacements of the slopes will be estimated using the Newmark sliding block analysis.

References: "Effects of Earthquakes on Dams and Embankments" by N.M. Newmark, the Fifth Rankine Lecture, 1965.

"Misc. Paper S-71-17, Earthquake Resistance of Earth and Rock-Fill Dams, Report No. 5, Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis" 1977. US Corps of Engineers, WES, Vicksburg. by Franklin + Chang

"A Simplified Procedure for Estimating Earthquake Induced Deformations in Dams and Embankments" by Makdisi & Seed, EERC, Univ. of Calif. 1977.

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gale

According to Makdisi & Seed (1977)
"Newmark (1965) and Seed (1966)
proposed methods of analysis for
predicting the permanent displacements
of dams subjected to earthquake
shaking and suggested this as a
criterion of performance as opposed
to the concept of a factor of
safety based on limit equilibrium
principles."

Following the same reasoning as
GEI (attached) it is conservative
to use the strengths as used
for the static stability analysis.

Results of static analysis
for slopes A + B are also attached.

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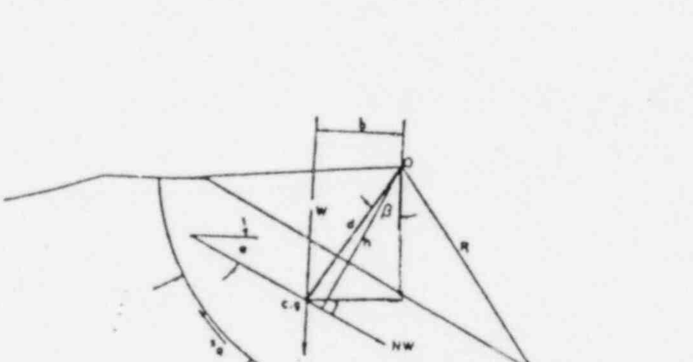
jah

Figures used to estimate the standardized maximum displacement from values of N/A are attached and were taken from the referenced documents.

N from Newmark (1965) for a circular cylindrical sliding surface for dilatant soils may be approximated as:

$$N = (FS - 1) \sin \beta$$



(a) Circular Sliding Surface

where: β is defined in above figure
FS = conventional static safety factor.

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With pts. O and c.g. already Known from the static analysis and applying trigonometry β can be readily determined.

Slope A

O is located @ coord. 1780, 4420

c.g. is located @ 805, 1360

$$\therefore \beta = \arctan \frac{\Delta X}{\Delta Y}$$

$$= \arctan \left(\frac{1780 - 805}{4420 - 1360} \right)$$

$$= 17.67^\circ$$

Slope B

O is located @ 1260, 1510

c.g. is located @ 1155, 1160

$$\therefore \beta = \arctan \left(\frac{1260 - 1155}{1510 - 1160} \right)$$

$$= 16.70^\circ$$

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gah

Remembering that FS comes from the static stability analysis

$\therefore \quad FS_{SLOPE\ A} = 1.46$

$FS_{SLOPE\ B} = 1.62$

Now N can be determined from

$$N = (FS-1) \sin \beta$$

$\therefore$

$$N_A = 0.14$$

$$N_B = 0.18$$

Franklin & Chang (1977) define N as the ground acceleration, as a fraction of g, required to make the factor of safety unity. This is what YAEC did (see GAH comps) for the pseudo-static analysis. Those N values for slopes A + B were 0.15 and 0.19, respectively. Therefore Newmarks approximation agrees well with Franklin + Chang (1977) approach.

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gah

For this permanent displacement analysis N values as calculated from the Newmark approach will be used since they are slightly more conservative for this analysis.

The YAEC composite spectra anchored at 0.1 g will be used to evaluate slope performance during an earthquake. The NRC spectra anchored at 0.2 g will also be checked.

	0.1g	0.2g
$(N/A)_{SLOPE\ A}$	1.4	0.7
$(N/A)_{SLOPE\ B}$	1.8	0.9

With these N/A values and using Fig 22 from Newmark (1965) and Fig. 16 from Franklin and Chang (1977) permanent displacements can be estimated. (Figures attached)

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gah

N/A	Standardized Max. Displacement, inches Fig. 22	Fig. 16
1.4	< 1.0	< 1.0
0.7	≈ 1.5	< 1.0
1.8	< 1.0	< 1.0
0.9	< 1.0	< 1.0

Therefore for both the YAEC (0.1g) and NRC (0.2g) earthquakes the Newmark sliding block analysis predicts negligible permanent displacements.

Slopes surrounding the Yankee plant will not catastrophically fail for earthquakes up to and greatly exceeding even the very conservative NRC recommended 0.2g spectra.

Jacobson

Rated 1965

Note: Use this in conjunction with CCE Misc. Paper S-71-17 "Earthquake Resistance of Earth and Rock-Fill Dams."

Fifth Rankine Lecture

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EFFECTS OF EARTHQUAKES ON DAMS AND EMBANKMENTS

by

N. M. NEWMARK, D.Sc., Ph.D., M.S., M.I.C.E.*

I wish to thank the British Geotechnical Society for the opportunity of visiting London again and for the honour of appearing before you in the home of the Institution of Civil Engineers, of which I am so proud to be a member.

Several years ago I transmitted some preliminary notes on the topic of earthquake effects on dams to the late Karl Terzaghi, whose invaluable advice and suggestions regarding those notes were freely used in the preparation of this Paper. I wish also to acknowledge the comments and suggestions I have had from time to time concerning the subject from my colleague at the University of Illinois, Dr Ralph B. Peck; from my associate in several consulting assignments, Dr Laurits Bjerrum; and from my colleague for several months, while he was visiting the University of Illinois, Dr N. N. Ambraseys.

Finally, I should like to acknowledge the assistance on some of the calculations for this lecture that were made by two of my associates at the University of Illinois, Dr John W. Melin, and Mr Mohammad Amin.

INTRODUCTION

General description of earthquake motions

In an earthquake, the earth moves in a nearly random fashion in all directions, both orizontally and vertically. Measurements have been made of earthquake motions in a umber of instances. In general, those measurements which are of greatest interest are the ecords of 'strong motion' earthquake accelerations, measured by the U.S. Coast and Geodetic urvey for a number of earthquakes in California in the past three decades. These accelera-ons, as a function of time, are available for motion in two horizontal directions as well as the vertical direction, at a number of locations for several earthquakes. From the time-cord of the acceleration, the velocities and displacements can be computed by integration.

One of the most intense strong motion records available is that for the El Centro, Cali-rnia earthquake of 18 May, 1940. The record for the north–south component of acceleration this earthquake is shown in Fig. 1, which also shows the values computed for velocity and splacement in the same direction. From the figure it can be observed that the maximum ound acceleration in the direction of this measurement is about 0·32 g, the maximum ground locity 13·7 in/sec, and the maximum ground displacement 8·3 in.

The general nature of earthquake motions is indicated by this figure. It can be noted at the highest intensity peaks of acceleration have a relatively short period or a relatively gh frequency; the most important peaks in the velocity, however, have a longer period ich corresponds to a lower frequency; and the important peaks in the ground displacement ve a much longer period still. For the ground conditions at El Centro the length of single ps of the highest intensities, in the various records, have durations of the order of the follow-: for acceleration, about 0·1 to 0·5 sec; for velocity, about 0·3 to 2 sec; and for displacement

* Professor of Civil Engineering, University of Illinois, Urbana, Illinois.

with Newmark's 5th Rankine Lecture

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MISCELLANEOUS PAPER S-71-17

EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS

Report 5

PERMANENT DISPLACEMENTS OF EARTH EMBANKMENTS BY NEWMARK SLIDING BLOCK ANALYSIS

by

Arley G. Franklin, Frank K. Chang

Soils and Pavements Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

November 1977
Report 5 of a Series

Prepared for Office, Chief of Engineers, U. S. Army
Washington, D. C. 20314

Under CWIS No. 31144

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U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

PB-276 820

A SIMPLIFIED PROCEDURE FOR ESTIMATING EARTHQUAKE-INDUCED DEFORMATIONS IN DAMS AND EMBANKMENTS

California University, Richmond, Earthquake Engineering Research Center

Prepared for

National Science Foundation, Washington, D. C.

August 1977

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Report on

SEISMIC STABILITY EVALUATION

HARRIMAN DAM

Submitted to

New England Power Company
Westborough, Massachusetts

Submitted by

Geotechnical Engineers Inc.
1017 Main Street
Winchester, Massachusetts 01890

June 23, 1982
Project 81858

Francis D. Leathers
Francis D. Leathers
Assistant Project Manager

Steve J. Poulos
Steve J. Poulos
Principal

defined as a constant rate of deformation at constant volume with constant shear stress and pore pressure (Poulos, 1981). Since the steady-state strength is sensitive to void ratio changes, the measured strengths from the $\bar{R}$ tests were corrected for the effect of void ratio changes during consolidation in the laboratory to estimate the in-situ steady-state strengths.

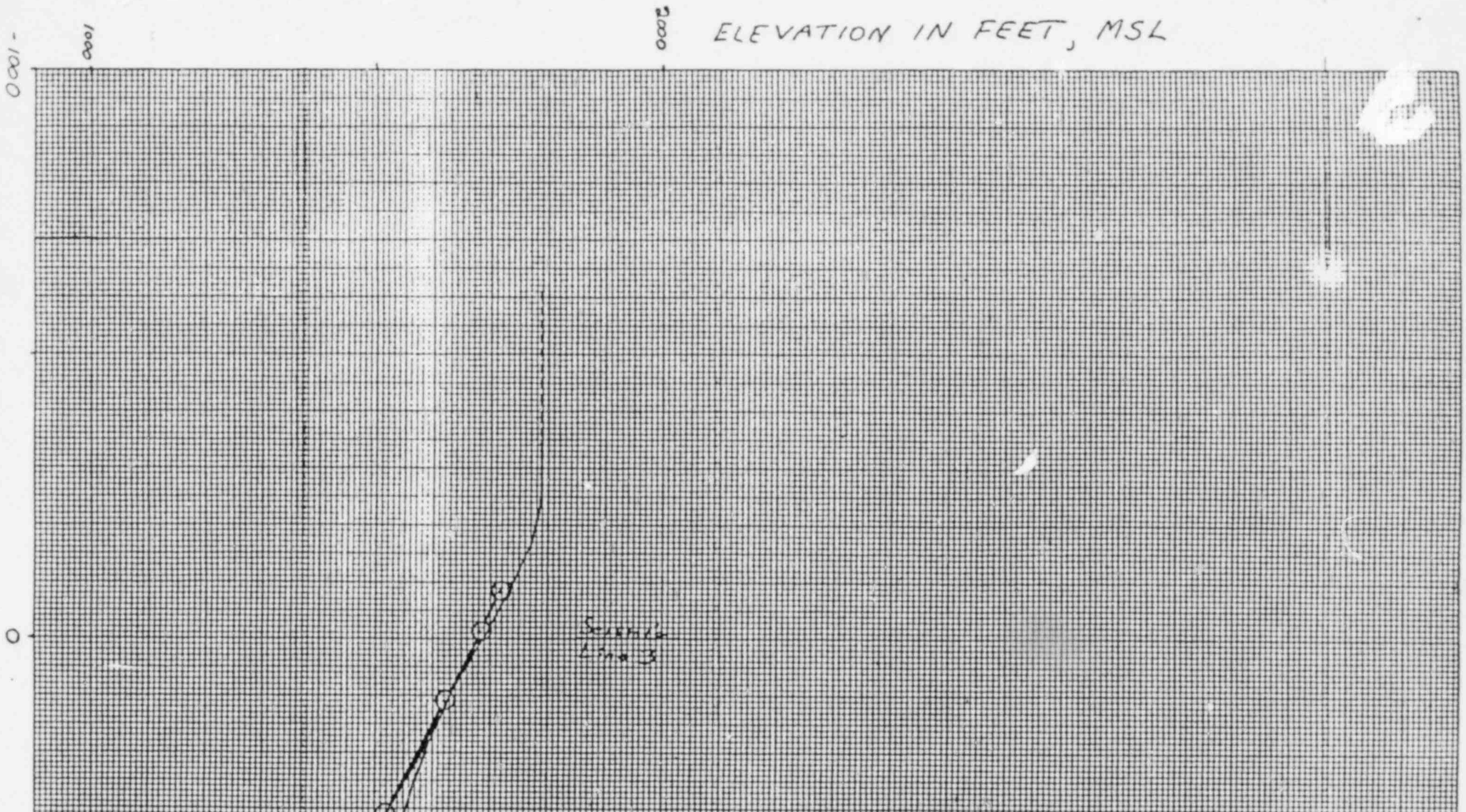
The shape of the steady-state line for the dumped shell was determined from undrained triaxial ($\bar{R}$) tests on compacted specimens, as shown in Fig. 1. The steady-state line from the compacted specimens and the uncorrected and corrected undrained steady-state strength from the five undisturbed shell specimens are shown in Fig. 2. The average in-situ undrained steady-state shear strength for the dumped shell is about 2000 psf after correction for consolidation. The in-situ undrained steady-state shear strengths are, on average, about 80% of the drained steady-state strengths.

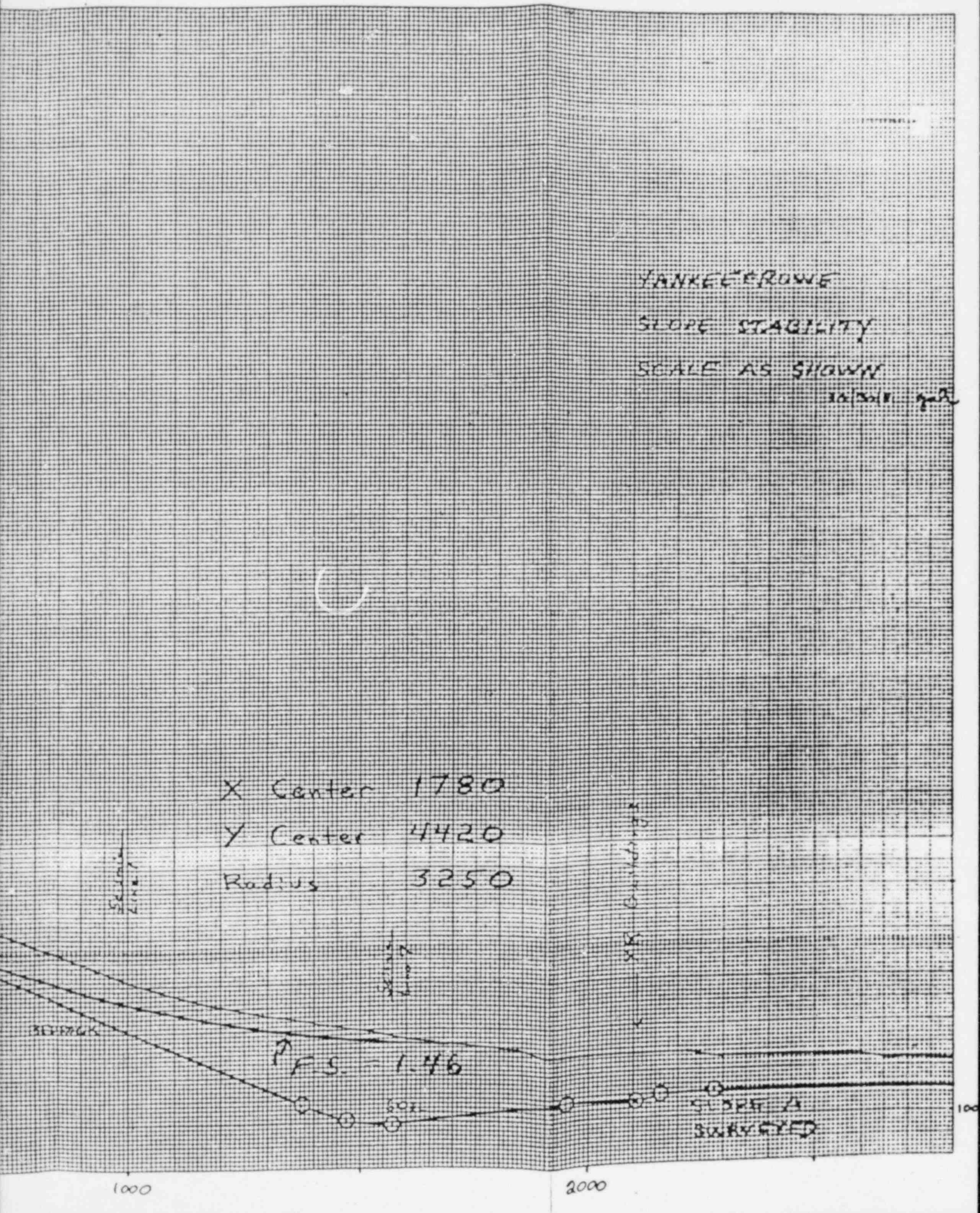
For the dumped shell below a depth of 60 ft, the higher corrected blowcounts indicate that the soils are dilative, which means that the undrained strength would be greater than the drained strengths below a depth of 60 ft. However, the negative pore pressures required to mobilize undrained strength greater than drained strength should not be relied upon for stability analyses. Therefore, an undrained strength equal to the drained strength was used for the deeper zones of the dumped shell. Drained steady-state strengths were calculated using the steady-state friction angle, $\phi_s = 30°$, determined from the $\bar{R}$ tests on compacted specimens (GEI, 1981a).

3.2.2 Hydraulic and Washed Core

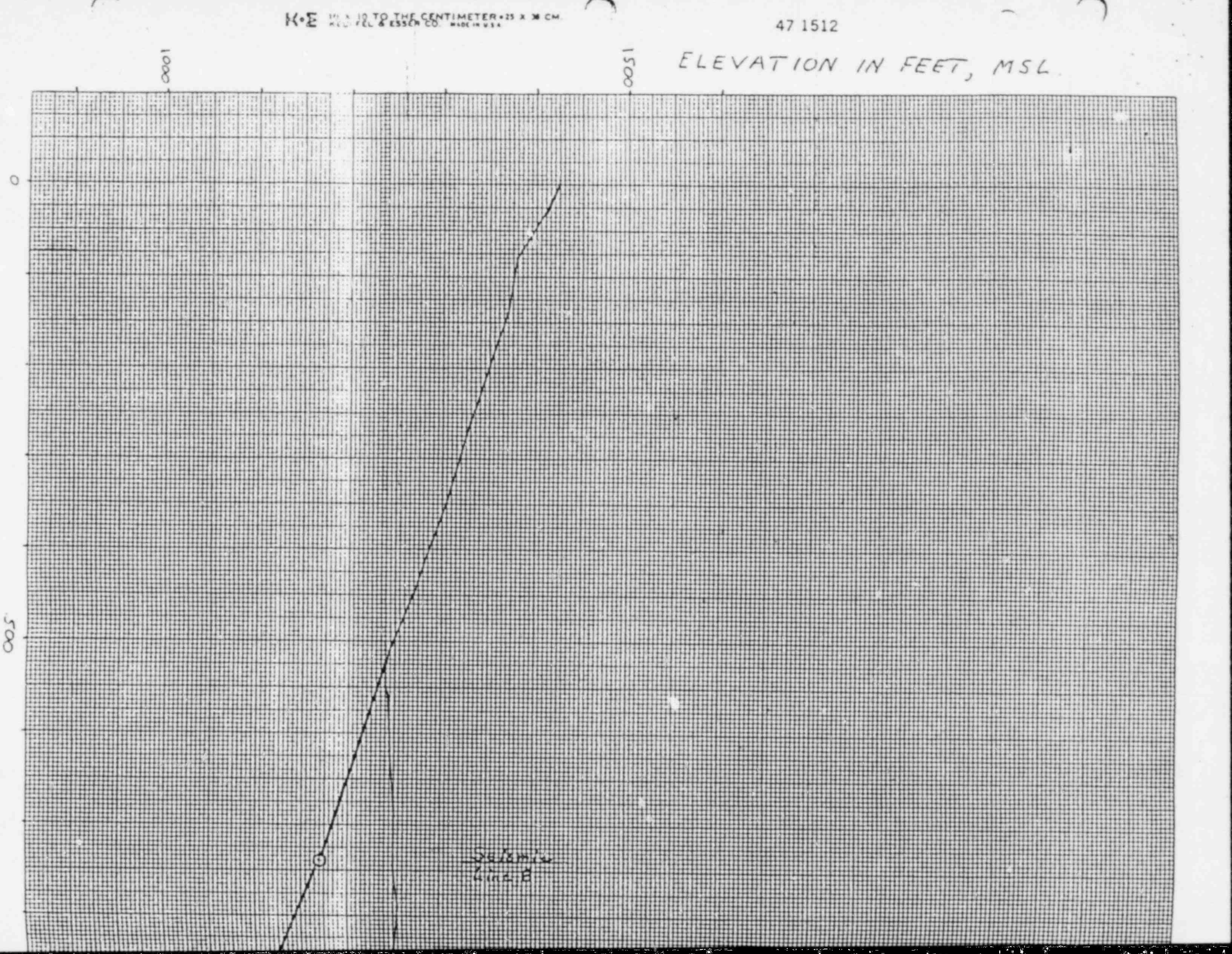
In-situ undrained steady-state shear strengths for the hydraulic core and washed zone of core were determined using the same approach as described above for the shallow dumped shell. Steady-state strengths were measured on three undisturbed specimens of washed core and two undisturbed specimens of hydraulic core. Steady-state lines were not measured for either core material at Harriman Dam. However, a band of steady-state lines was determined for the hydraulic core at Sherman Dam (GEI, 1982). Since Sherman Dam was constructed using procedures and materials similar to Harriman, it was assumed that the steady-state lines of the Harriman core materials were parallel to the lines from Sherman Dam.

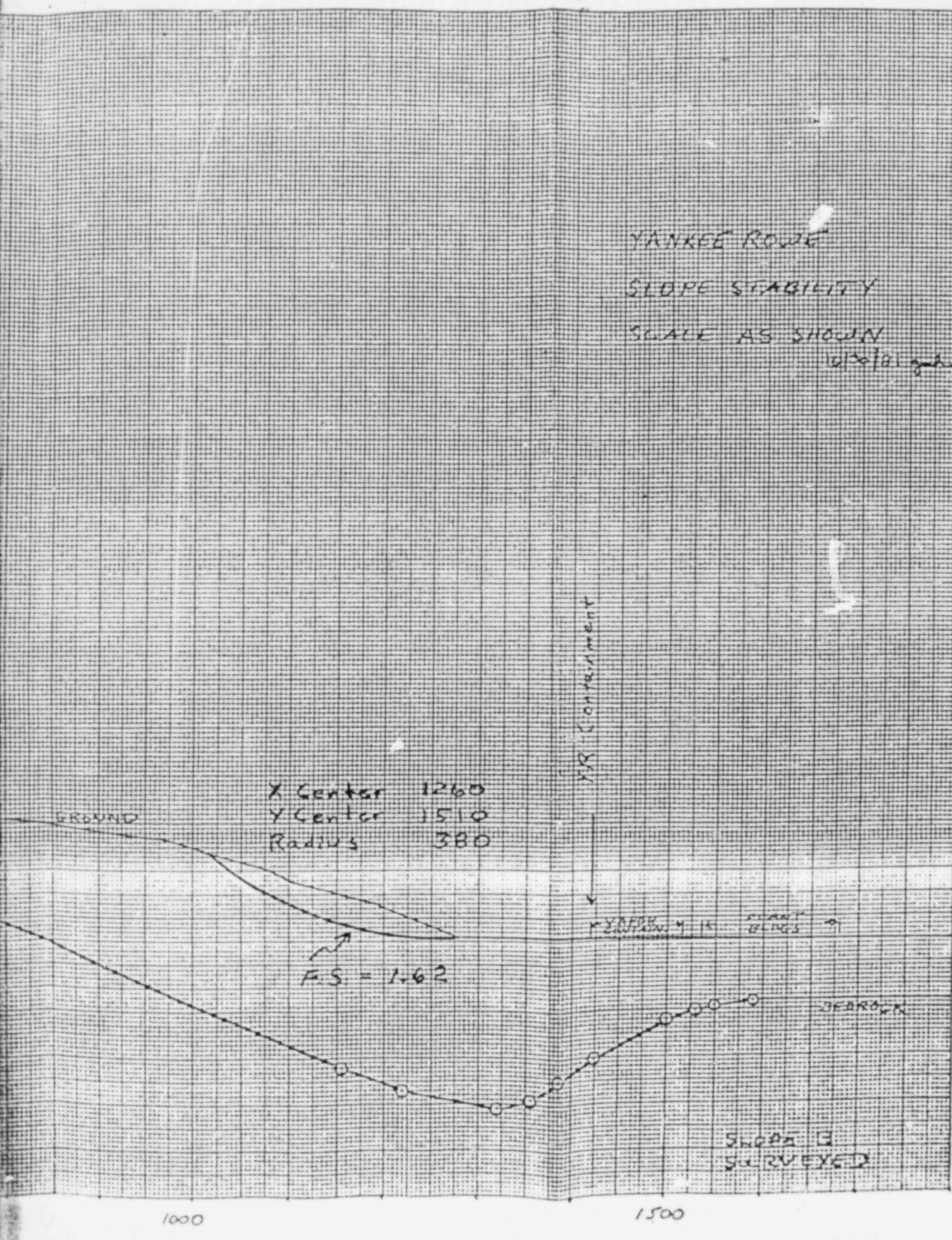
The steady-state lines and the uncorrected and corrected undrained steady-state strengths for the Harriman hydraulic core and washed core are shown in Fig. 3. The average in-situ steady-state strength for the three specimens of washed


ELEVATION IN FEET, MSL
1000
2000
-1000
0
GROUND



YANKEE ROWE
SLOPE STABILITY
SCALE AS SHOWN
X Center 1780
Y Center 4420
Radius 3250
BEDROCK
F.S. = 1.46
SLOPE A
SURVEYED
100
1000
2000


K·E 10 X 10 TO THE CENTIMETER 25 X 38 CM. KEUFFEL & ESSER CO. MADE IN U.S.A.
47 1512
ELEVATION IN FEET, MSL
1000
1500
0
500
Seismic
Line B



YANKEE ROWE
SLOPE STABILITY
SCALE AS SHOWN
10/30/81
X Center 1260
Y Center 1510
Radius 380
GROUND
YR Containment
F.S. = 1.62
BEDROCK
SHOWN SURVEYED
1000
1500

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Comparisons with model tests

The theoretical procedures described herein have been applied to tests of a model of a rockfill dam, described by Davis *et al.* (1960). The scale of the model was 1/300 of the prototype. The dynamic tests of the model were made by striking a shaking table with a heavy pendulum. A rebound of the pendulum caused a second input at a lower acceleration. Hence, data could be obtained both for the initial strike and for the first rebound.

On the whole, the model tests indicated a fair agreement with the calculations, for comparable conditions. Within the accuracy of the records obtained in the tests, the measured motions were in fairly good agreement with the results computed by means of equation (23) and Fig. 21.

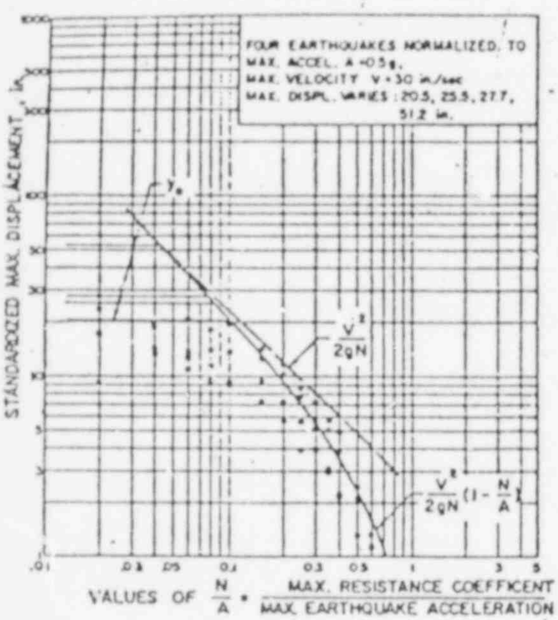


Fig. 21. Standardized displacement for normalized earthquakes (symmetrical resistance)

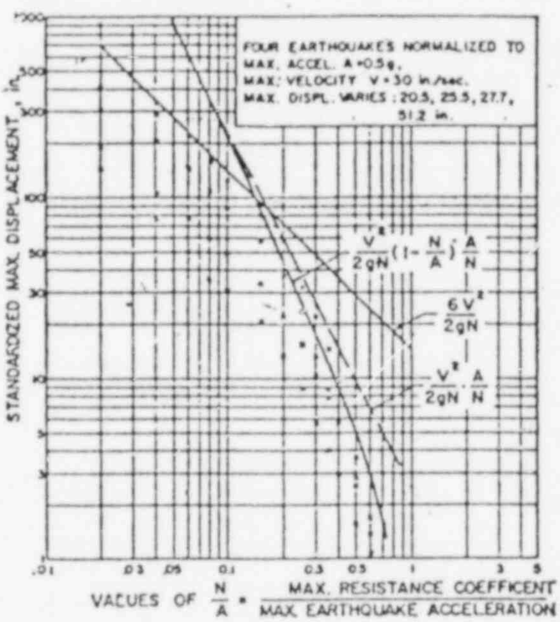


Fig. 22. Standardized displacement for normalized earthquakes (unsymmetrical resistance)

Comments and conclusions

For the maximum probable earthquake in California, which is a reasonable maximum earthquake for many other areas of the world, Fig. 22 may be used directly to obtain a measure of the maximum displacement for unsymmetrical sliding. If the maximum resistance coefficient is about 0·16, or about one-third the maximum earthquake acceleration, the net displacement will be about 1 ft. If the maximum resistance coefficient N is about 0·20 times the maximum earthquake acceleration, or N equals 0·1, the maximum displacement is about 5 ft. The maximum displacement increases rapidly as N decreases. Values of N in the range of 0·1 to 0·15 are not uncommon for earth dams designed for earthquake resistance. Of course, a design with a somewhat smaller value of N would have a smaller displacement if the earthquake were less intense. For an earthquake with a maximum acceleration of 0·25 g, and a maximum velocity of 15 in/sec, the displacements computed would be one-fourth those quoted, if the value of the ratio of N to A were the same. In other words, for the same relative value of resistance coefficient, the displacement varies as the square of the ground velocity. This displacement lowers the crest of the dam.

from: Newmark (1965)

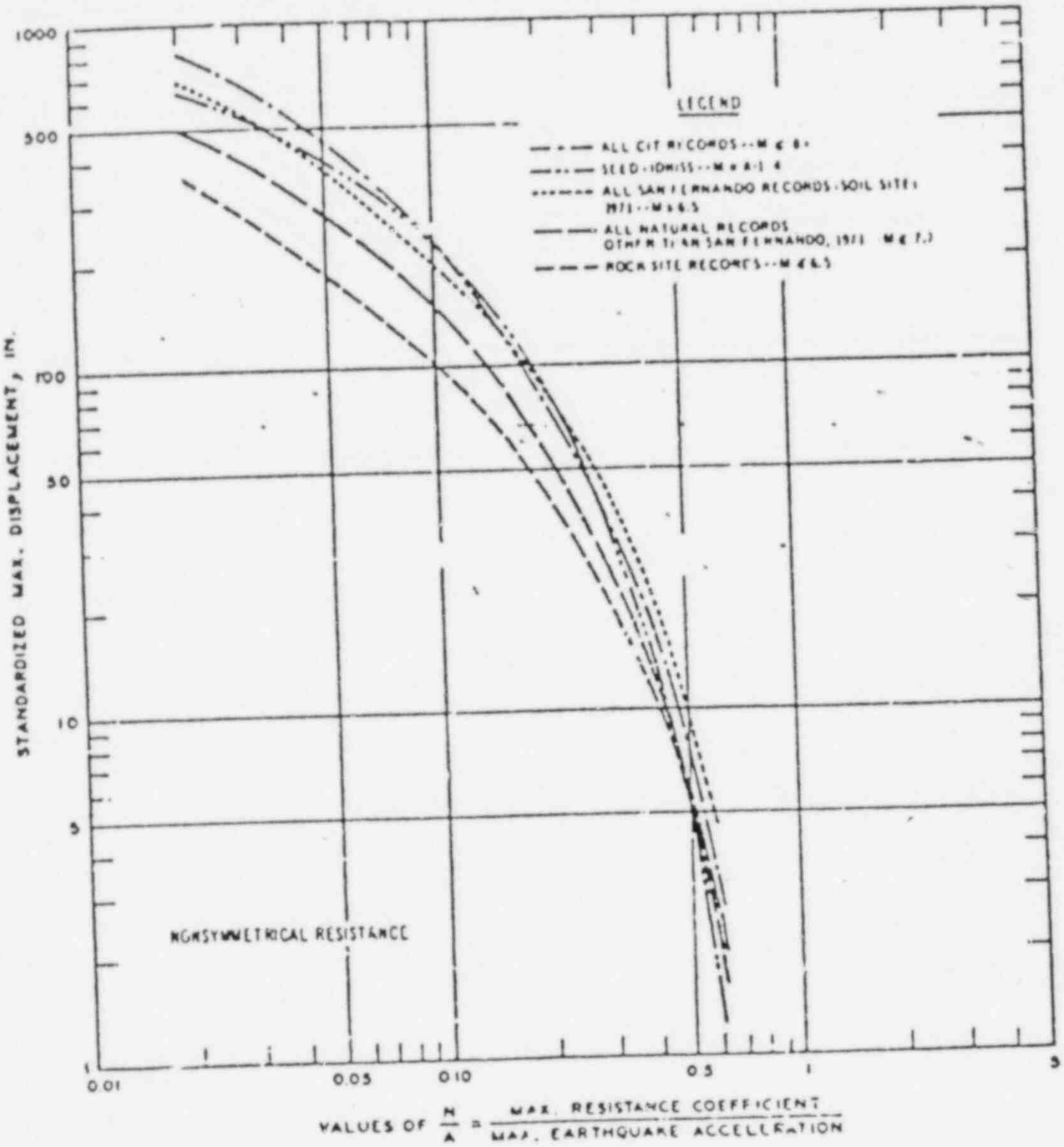
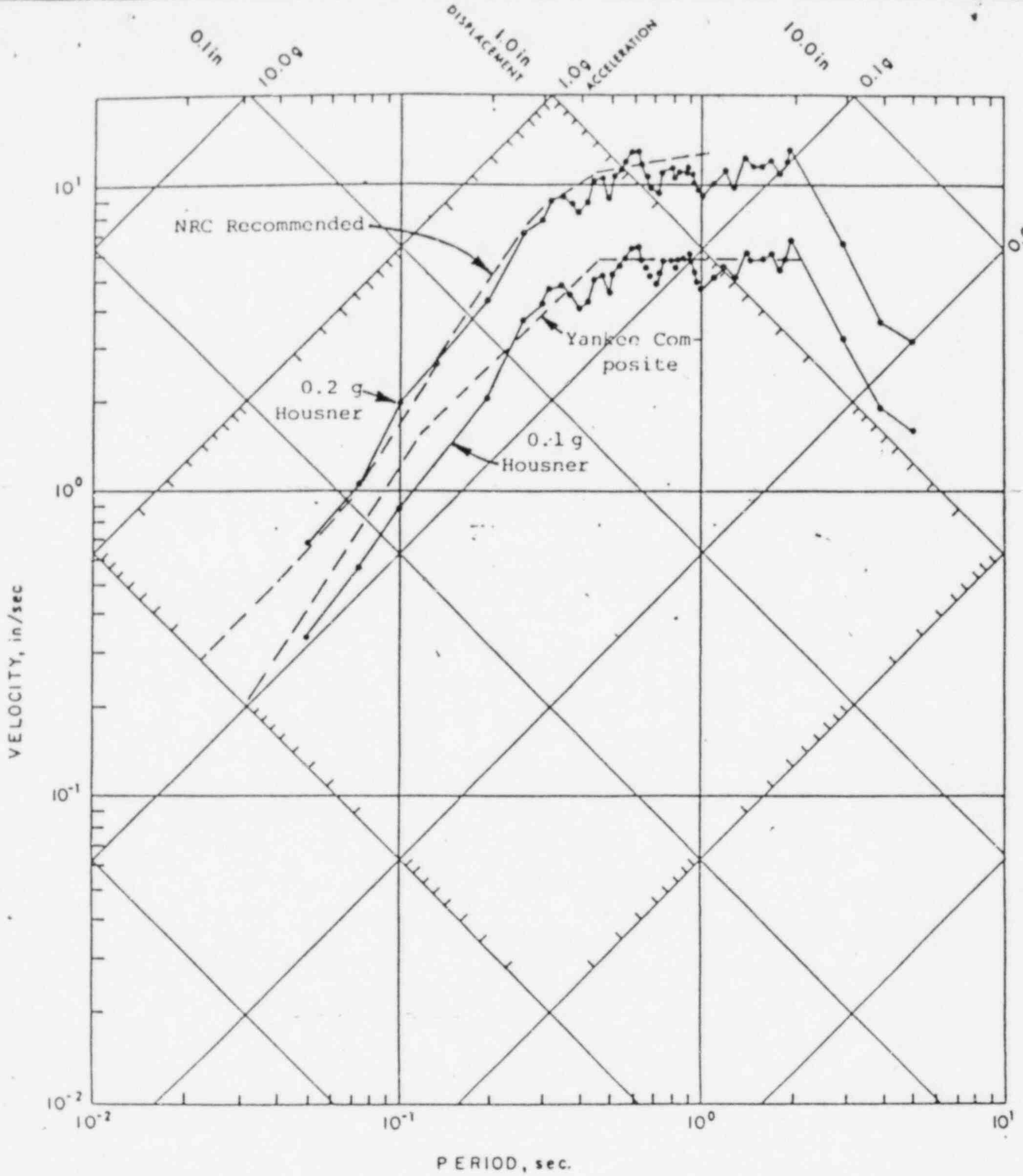


Figure 16. Upper bound envelope curves of permanent displacements for all natural and synthetic records analyzed

from: Franklin & Chang (1977)



New England Power Company Westborough, Massachusetts	Harriman Dam	EARTHQUAKE RESPONSE SPECTRA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	Dec. 15, 1981 Fig. 5

SLOPE FAILURE INTO SHERMAN RESERVOIR

The only slope identified around the Sherman Reservoir which, if it were to fail, would produce a significant wave in the reservoir is shown in Figure 1. From geologic and seismic surveys, it has been estimated that this slope has a surface area of about 9×10^5 ft^2 and a volume of about 2.2×10^7 ft^3. Seismic lines 5 and 6 on Figure 2 show the material thickness for this hypothetical maximum slide area.

Model studies and a few actual observations have shown that the wave energy imparted by slope failures into water bodies does not exceed about 2% of the net potential energy of the sliding body. The wave generated by a landslide can be approximated by using solitary wave theory. Assuming that 2% of the net potential energy of the soil mass, or about 5.64×10^9 ft-lb, is transformed into a solitary wave with an 800-foot wave front, the wave height generated in 100 feet of water would be about 12.4 feet. Run up on a 1 on 2 riprapped slope, such as at Sherman Dam, would be 13.6 feet above still water level. Thus, the dam would be overtopped only if the reservoir level were greater than 1116 feet (spillway elevation is 1103.66 feet). A washout and failure of the dam would probably not occur, however, unless the pool level were much closer to the top of the dam. The reservoir elevation would probably have to be higher than 1116 feet before the wave would impact plant structures, all of which, except the pumphouse, are more than 100 feet from the reservoir. Since these calculations are generally conservative, the maximum probable wave height generated by a deep-seated slide of the soil mass shown in Figure 1 would be less than that used here.

Because the slope failure postulated is highly improbable (there has been no sign of movement since the last ice age, 12,000 years ago) and because the reservoir would need to be more than 10 feet above spillway crest just to have the wave overtop the dam or impact plant structures, but not necessarily fail them, this postulated event poses no real safety hazard to the Yankee plant at Rowe, Massachusetts.

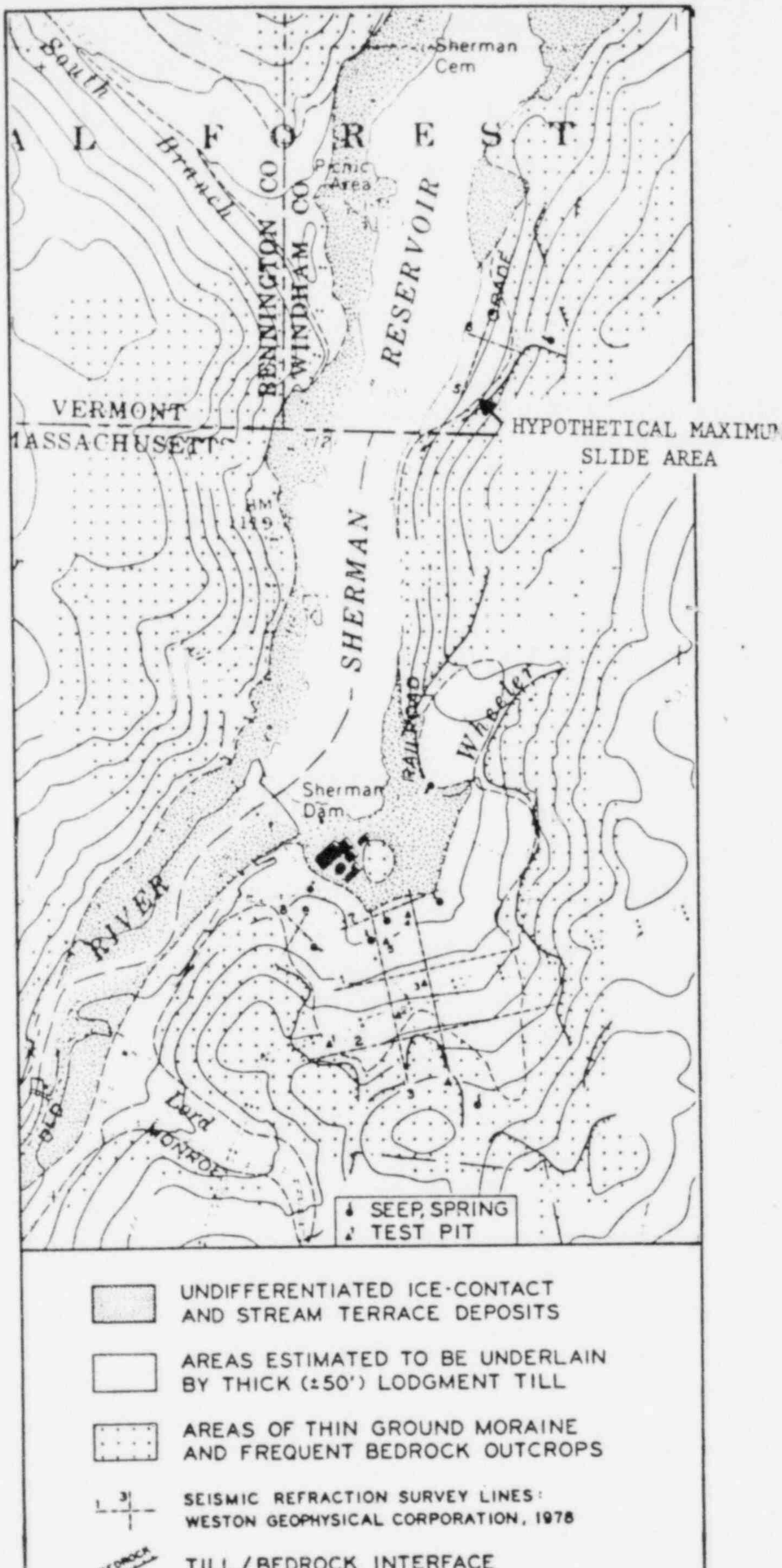
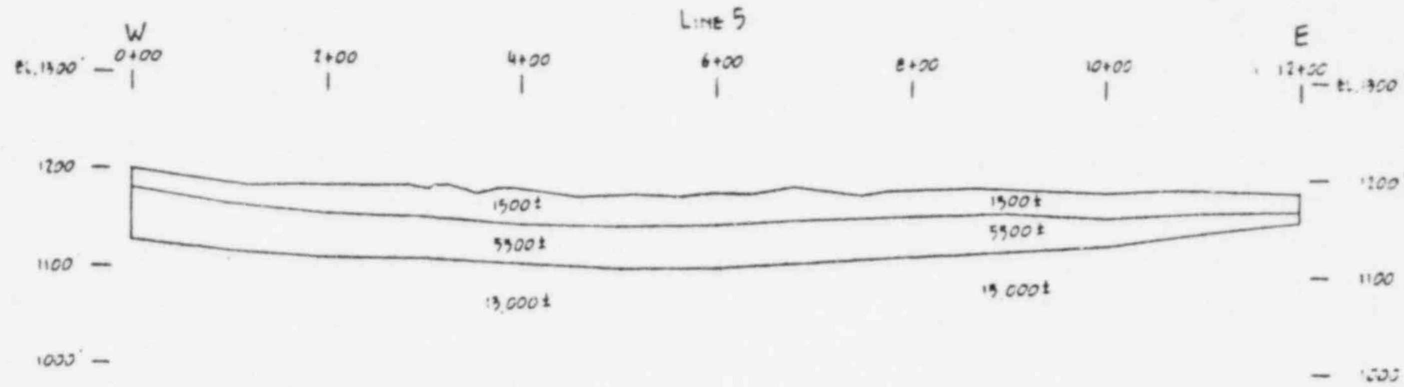
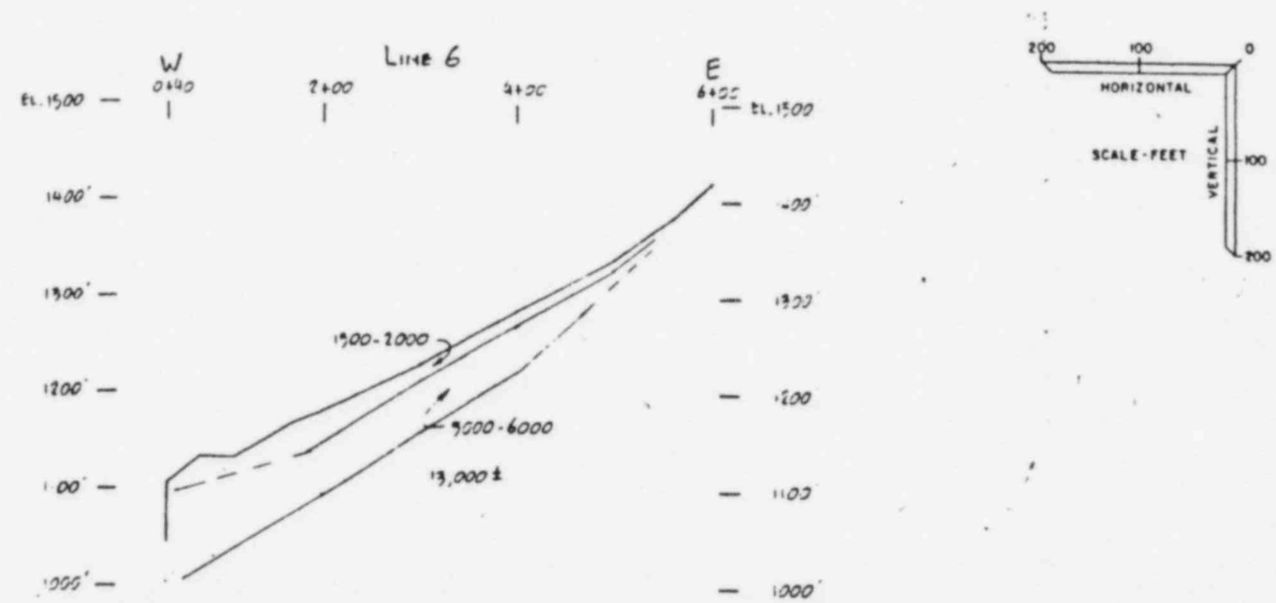


FIGURE 1

YANKEE ATOMIC ELECTRIC COMPANY - SITE AREA





NOTES: 1. VELOCITIES SHOWN ARE IN FEET/SECOND.

2. CONFER TEXT FOR DISCUSSION OF VELOCITY VALUES.

3. GROUND SURFACE PROVIDED BY YANKEE ATOMIC.

4. LOCATION OF SEISMIC LINES SHOWN ON FIGURE G2-1.

FIGURE 2

J. O. NO.

Revised Sept

SPECIFICATION

FOR

SITE CLEARING AND ROUGH GRADIN

FOR

YANKEE ATOMIC ELECTRIC PLANT

YANKEE ATOMIC ELECTRIC COMPANY

ROWE, MASSACHUSETTS

-:-

STONE & WEBSTER ENGINEERING CORPORATION

BOSTON, MASSACHUSETTS

S P E C I F I C A T I O N

FOR

SITE CLEARING AND ROUGH GRADING

FOR

YANKEE ATOMIC ELECTRIC PLANT

YANKEE ATOMIC ELECTRIC COMPANY

ROWE, MASSACHUSETTS

-:-

Stone & Webster Eng. Corp., Engrs. Boston, Mass., September 3, 1957
Revised September 13, 1957

GENERAL

This specification covers all materials, labor and equipment required to complete the clearing and rough grading of the site for the Yankee Atomic Electric Plant of the Yankee Atomic Electric Company at Rowe, Massachusetts, all on a unit price basis.

The following unit prices shall be established in the contract for this work:

1. Clearing and burning or disposal of all stumps and brush, per acre

2. General excavation of sand, gravel and boulders weighing less than 1 ton each per cubic yard

   a. Including disposal by this Contractor within 1,000 ft from place of excavation

   Including loading by this Contractor direct from excavation equipment on to railroad cars on siding of H.T.&W. Railroad where it crosses site, for disposal by others

3. Excavation of drainage ditches by equipment, as directed by the Engineers, per cubic yard. Excavated material to be piled alongside ditch

4. Additional cost per cubic yard for hauling and disposal by this Contractor in truck load lots at distances greater than 1,000 ft from place of excavation, in 1/4 mile increments of distance in excess of 1,000 ft

5. Placing compacted fill, using excavated material, per cubic yard in place

6. Excavation and disposal within 1,000 ft from place of excavation of boulders weighing 1 ton or more each, per ton

7. Excavation of solid rock ledge, per cubic yard

GENERAL CONDITIONS

All work performed at the site under this contract shall be governed by the "General Conditions - Material Delivered and Erected" in so far as these are applicable. These General Conditions are bound with and made a part of this specification.

ENGINEERS' DRAWINGS

The location and scope of the work is shown on the following Engineers' drawings:

9699-FY-6A - Issue No. 2 - Plot Plan
9699-FY-1C - Issue No. 2 - Site Clearing
9699-FY-5A - Issue No. 2 - Exclusion Area Plan

The Contractor shall be guided by such other drawings as may be furnished or approved by the Engineers to further explain the work.

LOCATION OF SITE, AND ACCESS THERETO

The site of the Yankee Atomic Electric Plant is on the east bank of the Deerfield River near the Sherman Dam of the New England Power Company. It is accessible by the Hoosac Tunnel and Wilmington Railroad which provides single track freight service from the Boston & Maine Railroad main line at the east portal of the Hoosac Tunnel to Readsboro, Vermont, several miles above the site. The railroad passes between the Sherman Dam and the site at the 6.5 mile mark.

Highway access is available from:

a. Greenfield, Massachusetts, on Route 2, and an improved but secondary highway from Charlemont to Monroe Bridge

b. Greenfield, Massachusetts, on Route 112 or Route 8A to Readsboro, Vermont, and then to Monroe Bridge

c. North Adams, Massachusetts, via Route 8 over the Berkshire Divide to Monroe Bridge

Highway access from Monroe Bridge to the plant site is either over Sherman Dam or via a road on the east side of the Deerfield River from both Rowe and Monroe Bridge. The bridge at the dam has a rated capacity of 50 tons.

GENERAL DESCRIPTION OF WORK

The existing elevations of the surface of the site are shown on drawing No. 9699-FY-1C. The plant areas to be cleared and graded are outlined on that drawing and on drawing No. 9699-FY-6A. The latter drawing shows the finished grades of the yard and roadways of the completed project.

The existing ground slopes from elevation approximately 1010 at the south end of Sherman Dam to elevation of approximately 1050 at the south side of the project area. The single track of the H.T. & W. Railroad is between the Deerfield River and the plant. The track is at approximately El. 1015 at the east side of the site and El. 1002 at the west side, running in a cut. The railroad alignment and grade are not to be changed. The finished grade of the northern half of the project area south of the railroad is to be at El. 1020. This is, in general, the plant operating area. The southern half of the project area is to slope from El. 1030 to El. 1040. This is in general the waste disposal area.

When the project is completed the division between these two principal areas will consist in part of a sloping bank, and in part will occur where the primary auxiliary building and fuel storage pit are placed in such manner that yard will be at El. 1020 on the north side of these structures and at El. 1030 or higher on the south side. Sloping banks are to be left by this Contractor as directed by the Engineers at all points of division between the two areas.

The above finished grades of El. 1020 and El. 1030 are tentative, and subject to possible revision upwards to not higher than El. 1025 and El. 1035 respectively, over all or part of the site, as may be directed by the Engineers.

Plant roadways are to be provided from the entrance gate at the west side of the plant as shown on drawing No. 9699-FY-5A, extending from there to the various parts of the plant area as shown on drawing No. 9699-FY-6A, with a ramp leading from the plant operating area to the waste disposal area. The finished grades of the plant roadways are shown on drawing No. 9699-FY-6A.

Two spur tracks are to extend into the plant operating area, connecting to the H. T. & W. Railroad, at the east side of the site as shown on drawing No. 9699-FY-6A. These spurs will connect to the adjacent existing railroad at approximately El. 1015 and will slope up to El. 1020 in the plant operating area.

Between the H. T. & W. Railroad, and the Deerfield River, an area just east of the dam, where the existing ground varies from approximately El. 1010 to El. 1028, is to be graded level at approximately El. 1014, to be used as a laydown area for construction purposes.

This Contractor shall do the clearing, excavation and fill necessary for the complete clearing and rough grading of the plant operating area, waste disposal area, construction laydown area, plant roadway system and spur track installation, all as shown on the drawings herein listed. Included in the work at the unit price established for same in the contract shall be the excavation of all drainage ditches required at this stage of operations for general project purposes, as directed by the Engineers.

SURFACE AND SUBSOIL CONDITIONS

Bound in this specification is a "Boring and Seismic Survey Plan" and a "Log of Borings". These may be referred to by the Contractor in planning the work but his interpretation of them shall be his own responsibility. In general the subsoil consists of sand, gravel and various sizes of boulders, and it is not anticipated that any large quantity of solid rock ledge will be encountered.

LINES, GRADES AND STAKING OUT OF WORK AREAS

The Engineers will stake out the work for this Contractor, indicating the extent and final elevations of the areas to be cleared and rough-graded. They will mark out the alignments, grades and widths of roadbed for the plant roadways and spur tracks, and all drainage ditches required at this stage of operations for general project purposes. In general the plant operating area and waste disposal area shall be rough-graded at levels 4 in. below, roadways 8 in. below and spur tracks 18 in. below their finished grades. The construction laydown area shall be rough graded and compacted at its finished grade. Sloping banks around the perimeters of excavated and filled areas shall be sloped at 1 1/2 to 1 unless otherwise directed by the Engineers. Subject to the approval of the Engineers, this Contractor may at his own expense construct temporary roadways as required for his own convenience in doing the work.

ALLOWABLE TOLERANCES

Rough graded surfaces shall be graded with a tolerance of plus or minus 2 in. and shall be free of ruts or irregularities. If this Contractor inadvertently excavates deeper than the allowable tolerance or does so in order to remove a boulder or boulders, he shall backfill to the required elevation and tolerance. Where backfill is required to correct an error of this Contractor, it shall be placed at this Contractor's expense. Areas where the existing ground is below the desired levels for rough grading shall be cleared and stripped of all topsoil and filled with compacted fill to the required elevations and tolerance.

CLEARING

In all areas where work is to be done by him under this contract, this Contractor shall clear away, pile and burn or, if too wet to burn, dump within 1,000 ft where directed by Engineers, all stumps and underbrush. All burning shall be subject to approval by the Engineers as to time and place, but this approval shall not relieve the Contractor of responsibility for the fire hazard. Stump removal by this Contractor shall be limited to areas excavated or filled by him.

LARGE BOULDERS

Excavated boulders estimated to weigh 1 ton or more, based on an assumed unit weight of 170 lb per cu ft shall be paid for at the unit price established in the contract for this purpose. Excavation of boulders weighing less than 1 ton each shall be paid for at the unit price for excavation of sand and gravel.

FILL AND BACKFILL

All fill and backfill within the area of the project shall be done with suitable excavated material selected or approved for the purpose by the Engineers. This shall be placed in uniform layers not more than 12 in. thickness. Each layer shall be thoroughly compacted by running heavy construction equipment over it for at least ten passes in a manner approved by the Engineers.