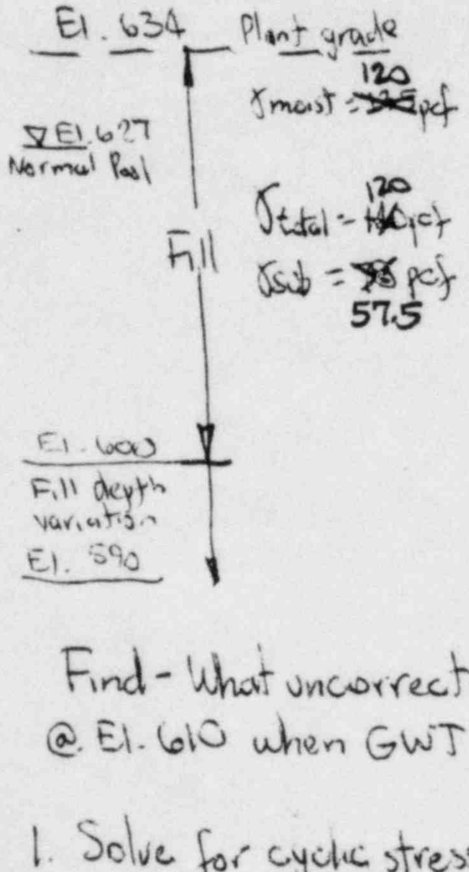
J. Kane
5/16/82

3/B8

Subject: Liquefaction Analysis - Midland

Method - Seed - Idriss Simplified Analysis

Assumptions
- GWT was at or BELOW elev. 610
- Peak ground surface acceleration = 0.19 g
- required factor of safety = 1.5 against liquefaction type failure
- Magnitude 6 earthquake



Find - What uncorrected blow count (N) is required to have a F.S. = 1.5 @ El. 610 when GWT is @ Elev. 610 when a_{max} = 0.19g.?

1. Solve for cyclic stress ratio causing liquefaction τ/σ_o' for use in Seed's curves

$$\frac{\tau}{\sigma_o'} = \text{cyclic stress ratio} = \frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{\overset{a_{max}}{0.19\,g}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

Where a_{max} = peak ground surface acceleration

r_d = stress reduction factor (in this case @ 20' depth)

σ_o = total overburden pressure (in this case @ El. 610)

σ_o' = effective overburden pressure @ El. 610

Method believed to be used by Paul Hadala

$$\frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

$$= 0.65 \times \frac{0.19\cancel{g}}{\cancel{g}} \times \frac{(120\,lb/ft^3 \times 24\,ft)}{(120\,lb/ft^3 \times 24\,ft)} \times 0.94$$

$$\frac{\tau_{avg}}{\sigma_o'} = 0.116$$

From Seed's curves w/ $M = 6$ & $\frac{\tau_{avg}}{\sigma_o'} = 0.116$

the corrected blow count $N_1 = 8$ blow/ft

Since $N_1 = C_N \cdot N$

$$N = \frac{N_1}{C_N}$$

where $C_N = 1 - 1.25 \log \frac{\sigma_o'}{\sigma_1}$

$\sigma_o' = 120\,lb/ft^3 \times 24\,ft$
$= 2880\,lb/ft^2$
$= 1.44$ tsf

$\sigma_1 = 1$ tsf

$$C_N = 1 - 1.25 \log \overset{0.158}{\frac{1.44}{1.0}}$$

$C_N = 1 - 0.198$
$C_N = 0.802$

$$N = \frac{N_1}{C_N} = \frac{8}{0.802} = 9.97 \text{ Say } 10$$

w/ F.S. = 1.5 $1.5 \times N = 1.5 \times 10 = 15$ blow/ft rd @ 24' depth = 0.94

$$\frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot rd$$

$$= 0.65 \times \frac{0.19g}{g} \times \frac{(120 lb/ft^3 \times 24 ft)}{(120 lb/ft^3 \times 24 ft)} \times 0.94$$

$$\frac{\tau_{avg}}{\sigma_o'} = 0.116$$

For F.S. = 1.5 — Here Factor of safety is being applied to cyclic stress ratio

$1.5 \times 0.116 = 0.174$

For M = 6 from Seed's curves w/ $\frac{\tau_{avg}}{\sigma_o} = 0.174$

the corrected blow count $N_1 = 12.5$ blows/ft

$N_1 = C_N \cdot N$

$N = \frac{N_1}{C_N}$

where $C_N = 1 - 1.25 \log \frac{\sigma_o'}{\sigma_1}$

$C_N = 1 - 1.25 \log \frac{1.44}{1.0}$

$\sigma_o' = 120 lb/ft^3 \times 24 ft = 2880$ psf

$\sigma_o' = 1.44$ tsf

$\sigma_1 = 1$ tsf

$C_N = 1 - 1.25(.158)$

$C_N = 1 - 0.198$

$C_N = 0.802$

$N = \frac{N_1}{C_N} = \frac{12.5}{0.802} = 15.6$

Adopt $N = 16$

Oct. 14, 1981
1 of 2
J. Kane

3/138

Subject: Summary of CPCo Responses on INPUT OF SOIL PARAMETERS TO SEISMIC DESIGN (Refer to NRC Questions 13, 25 & 48)

DIESEL GENERATOR BUILDING

pg. 13-1 Seismic reanalysis is being conducted to account for lack of compaction. Except for Vs (shear wave velocity) & ρ (soil density) the structural & soil properties are UNCHANGED

The reanalysis was to consider range in Vs from 500 fps to 1359 fps & ρ from 120 pcf to 135 pcf

pg. 13-2 The IMPACT of the new range in Vs & ρ on structural response is:

a. Acceleration, velocity & displacement is INCREASED which means HIGHER MOMENTS & SHEAR FORCES

b. Floor response acceleration spectra curves are WIDENED & INCREASED. This in turn more severe seismic loading on Cat. I equipment & piping

Results of REANALYSIS TO BE RESUBMITTED

pg. 25-2 FINITE ELEMENT approach in seismic analysis HAS NOT BEEN used @ MIDLAND

- HAVE USED HALF SPACE LUMPED SPRING & MASS APPROACH (BECHTEL TOPICAL REPORT 4A, Revision 3)

- Original Soil Input / Input in Reanalysis (pg 25-3)

Original Soil Input		Input in Reanalysis (pg 25-3)
Elastic modulus	= 22,000 KSF	= 6,598 KSF
Poisson's ratio	= 0.42	= 0.45
Unit weight	= 135 pcf	= 115.6 pcf
Shear wave velocity	= 1,359 ft/sec	= 796 ft/sec

pg. 25-3 - New seismic analysis (reanalysis) showed SHIFT in natural frequency

- The design forces & response spectra WERE DEVELOPED BY ENVELOPING the results of original & new analysis. CPCo concludes that this provides envelope of soil conditions from lower Vs = 500 fps to upper Vs = 1,359

Oct. 14, 1981
2 of 2
J. Kane

DIESEL GENERATOR BUILDING (cont.)

pg. 48-3 Seismic reanalysis has been performed in accordance w/ CPCo response to Q 13, 25 & 48. Results WILL BE SUBMITTED in the future IN revision to responce to Q 25.

pg. 48-6 Gives soil input used in the original and new seismic analysis of DGB

Horizontal	Original Analysis		New Analysis	
	East/West	North/South	East/West	North/South
K_x (K/ft)	1.5×10^6	1.5×10^6	7.0×10^5 same as Structural Audit	6.9×10^5
K_ψ (K-ft/radian)	8.1×10^9	3.3×10^9	3.6×10^9 (3.5×10^9 @ Struct Audit)	1.7×10^9
D_x (Damping ratio)	0.40	0.40	0.63	0.56
D_ψ (Damping ratio)	0.38	0.23	0.41	0.36

Vertical	Original Analysis	New Analysis
K_z (K/ft)	2.0×10^6	9.3×10^5
D_z (Damping ratio)	0.68	0.95

Jo Kane

3/138

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

AUG 4 1980

Docket Nos.: 50-329/330

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

SUBJECT: CORP OF ENGINEERS REPORT AND REQUEST FOR ADDITIONAL INFORMATION ON PLANT FILL

My letter of June 30, 1980 requested the results of additional explorations and laboratory testing needed to support certain geotechnical engineering studies on the Midland plant fill and associated remedial actions. That letter noted that details on the extent of these studies would be provided by separate correspondence. Enclosure 1 is a letter report of July 7, 1980 by our consultant, the U.S. Army Corps of Engineers, and is forwarded to this end.

Paragraph 4 of the Corps report identifies additional information needed to resolve specific problems identified in paragraph 3. For purposes of control, we have re-numbered the subparagraphs of paragraph 4 to be sequential with our prior requests on this matter. They have also been marked to reflect the results of NRR review. Your reply should reference the revised numbering system and should address the requests as marked to reflect our changes.

Subparagraph 4j of the Corps report entitled Liquefaction Potential, is not included in our re-numbering since it represents an evaluation rather than a request. We consider this evaluation to be tentative at this time since it is subject to the determination of suitable seismic design input for the site. We will address this matter shortly by separate correspondence.

~~8008270159~~ 6 pp.

We would appreciate your reply at your earliest opportunity. Should you need clarification of these requests for additional information, please contact us.

Sincerely,

A. Schwencer

A. Schwencer, Acting Chief
Licensing Branch No. 3
Division of Licensing

Enclosure:
COE Letter Report
dated 7/7/80

cc: See next page

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DEPARTMENT OF THE ARMY

DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

ENCLOSURE 1

REPLY TO ATTENTION OF

7 JUL 1980

NCEED-T

SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant Units 1 and 2, Subtask No. 1 - Letter Report

THRU: Division Engineer, North Central
ATTN: NCDED-G (James Simpson)

TO: U.S. Nuclear Regulatory Commission
ATTN: Dr. Robert E. Jackson
Division of Systems Safety
Mail Stop P-314
Washington, D. C. 20555

1. The Detroit District hereby submits this letter report with regard to completion of subtask No. 1 of the subject Interagency Agreement concerning the Midland Nuclear Plant, Units 1 and 2. The purpose of this report is to identify unresolved issues and make recommendations on a course of action and/or cite additional information necessary to settle these matters prior to preparation of the Safety Evaluation Report.

2. The Detroit District's team providing geotechnical engineering support to the NRC to date has made a review of furnished documents concerning foundations for structures, has jointly participated in briefing meetings with the NRC staff, Consumers Power Company (the applicant) and personnel from North Central Division of the Corps of Engineers and has made detailed site inspections. The data reviewed includes all documents received through Amendment 78 to the operating license request, Revision 28 of the FSAR, Revision 7 to the 10 CFR 50.54(f) requests and MCAR No. 24 through Interim Report No. 8. Generally, each structure within the complex was studied as a separate entity.

3. A listing of specific problems in review of Midland Units 1 and 2 follows for Category I structures. The issues are unresolved in many instances, because of inadequate or missing information. The structures to be addressed follow the description of the problem.

a. Inadequate presentation of subsurface information from completed borings on meaningful profiles and sectional views. All structures.

8008270160

b. Discrepancies between soil descriptions and classifications on boring logs with submitted laboratory test results summaries. Examples of such discrepancies are found in boring T-14 (Borated water tank) which shows stiff to very stiff clay where laboratory tests indicate soft clay with shear strength of only 500 p.s.f. The log of boring T-15 shows stiff, silty clay, while the lab tests show soft, clayey sand with shear strength of 120 p.s.f. All structures.

c. Lack of discussion about the criteria used to select soil samples for lab testing. Also, identification of the basis for selecting specific values for the various parameters used in foundation design from the lab test results. All structures.

d. The inability to completely identify the soil behavior from lab testing (prior to design and construction) of individual samples, because in general, only final test values in summary form have been provided. All structures.

(1) Lack of site specific information in estimating allowable bearing pressures. Only textbook type information has been provided. If necessary, bearing capacity should be revised based on latest soils data. All structures on, or partially on, fill.

(2) Additional information is needed to indicate the design methods used, design assumptions and computations in estimating settlement for safety related structures and systems. All structures except Diesel Generator Building where surcharging was performed.

e. A complete detailed presentation of foundation design regarding remedial measures for structures undergoing distress is required. Areas of remedial measures except Diesel Generator Building.

f. There are inconsistencies in presentation of seismic design information as affected by changes due to poor compaction of plant fill. Response to NRC question 35 (10 CFR 50.54f) indicates that the lower bound of shear wave velocity is 500 feet per second. We understand that the same velocity will be used to analyze the dynamic response of structures built on fill. However, from information provided by the applicant at the site meeting on 27 and 28 February 1980, it was stated that, except for the Diesel Generator Building, higher shear wave velocities are being used to re-evaluate the dynamic response of the structures on fill material. Structures on fill or partially on fill except Diesel Generator Building.

4. A listing of specific issues and information necessary to resolve them.

39. ~~a.~~ Reactor Building Foundation Response found in Rev. 10 (Nov. 1980)

(1) Settlement/Consolidation. Basis for settlement/consolidation of the reactor foundation as discussed in the FSAR assumes the plant site would

not be dewatered. Discuss and furnish computation for settlement of the Reactor Buildings in respect to the changed water table level as the result of site dewatering. Include the effects of bouyancy, which were used in previous calculations, and fluctuations in water table which could happen if the dewatering system became inoperable.

(2) Bearing Capacity. Bearing capacity computations should be provided and should include method used, foundation design, design assumptions, adopted soil properties, and basis for selecting ultimate bearing capacity and resulting factor of safety.

40. ~~b~~. Diesel Generator Building.

(1) Settlement/Consolidation. In the response to NRC Question 4 and 27, (10 CFR 50.54f), the applicant has furnished the results of his computed settlements due to various kinds of loading conditions. From his explanation of the results, it appears that compressibility parameters obtained by the preload tests have been used to compute the static settlements. Information pertaining to dynamic response including the amplitude of vibration of generator pedestals have also been furnished. The observed settlement pattern of the Diesel Generator Building indicates a direct correlation with soil types and properties within the backfill material. To verify the preload test settlement predictions, compute settlements based on test results on samples from new borings which we have requested in a separate memo and present the results. Reduced ground water levels resulting from dewatering and diesel plus seismic vibration should be considered in settlement and seismic analysis. Furnish the computation details for evaluating amplitude of vibration for diesel generator pedestals including magnitude of exciting forces, whether they are constant or frequency dependent.

(2) Bearing Capacity. Applicant's response to NRC Question 35 (10 CFR 50.54f) relative to bearing capacity of soil is not satisfactory. Figure 35-3, which has been the basis of selection of shear strength for computing bearing capacity does not reflect the characteristics of the soils under the Diesel Generator Building. A bearing capacity computation should be submitted based on the test results of samples from new borings which we have requested in a separate memo. This information should include method used, foundation design assumptions, adopted soil properties and basis for selection, ultimate bearing capacity and resulting factor of safety.

(3) Preload Effectiveness. The effectiveness of the preload should be studied with regard to the moisture content of the fill at the time of preloading. The height of the water table, its time duration at this level, and whether the plant fill was placed wet or dry of optimum would be all important considerations.

(a) Granular Soils.

When sufficient load is applied to granular soils it usually causes a reorientation of grains and movement of particles into more stable positions plus (at high stresses) fracturing of particles at their points of contact. Reorientation and breakage creates a chain reaction among these and adjacent particles resulting in settlement. Reorientation is resisted by friction between particles. Capillary tension would tend to increase this friction. A moisture increase causing saturation, such as a rise in the water table as occurred here, would decrease capillary tension resulting in more compaction. Present a discussion on the water table and capillary water effect on the granular portion of the plant fill both above and below the water table during and after the preload.

(b) Impervious and/or Clay Soils.

Clay fill placed dry of optimum would not compact and voids could exist between particles and/or chunks. In this situation SPT blow counts would give misleading information as to strength. Discuss the raising of the water table and determine if the time of saturation was long enough to saturate possible clay lumps so that the consolidation could take place that would preclude further settlement.

Discuss the preload effect on clay soils lying above the water table (7 feet $\pm$) that were possibly compacted dry of optimum. It would appear only limited consolidation from the preload could take place in this situation and the potential for further settlement would exist.

Discuss the effect of the preload on clays placed wet of optimum. It would appear consolidation along with a gain in strength would take place. Determine if the new soil strength is adequate for bearing capacity.

~~Conclusion: Since the reliability of existing fill and compaction information is uncertain, additional borings and tests to determine void ratio (granular soils) relative density, moisture content, density, consolidation properties and strength (triaxial tests) would appear to be desirable in order to satisfactorily answer the above questions. Borings should be continuous push with undisturbed cohesive soil samples taken.~~ Deleted: Covered by 6/30/80 Letter

(4) Miscellaneous. A contour map, showing the settlement configuration of the Diesel Generator Building, furnished by the applicant at the meeting of 27 and 28 February 1980 indicates that the base of the building has warped due to differential settlements. Additional stresses will be induced in the various components of the structure. The applicant should evaluate these stresses due to the differential settlement and furnish the computations and results for review.

41. ~~f.~~ Service Water Building Foundation.

(1) Bearing Capacity. A detailed pile design based upon pertinent soil data should be developed in order to more effectively evaluate the proposed pile support system prior to load testing of test piles. Provide adopted soil properties, reference to test data on which they are based, and method and assumptions used to estimate pile design capacity including computations. Provide estimated maximum static and dynamic loads to be imposed and individual contribution (DL, LL, OBE, SSE) on the maximum loaded pile. Provide factor of safety against soil failure due to maximum pile load.

(2) Settlements.

(a) Discuss and provide analysis evaluating possible differential settlement that could occur between the pile supported end and the portion placed on fill and glacial till.

(b) ~~Present~~ Discuss why the retaining wall adjacent to the intake structure is not required to be a Seismic Category I structure. Describe the impact of failure on safety related features (e.g., diesel fuel oil storage tanks) behind or near the wall. Evaluate the observed settlement of both the service water pumphouse retaining walls and the intake structure retaining wall and the significance of the settlement including future settlement prediction on the safe operation of the Midland Nuclear Plant. This evaluation should address actual stresses induced by the settlement against allowable stresses permitted by approved codes.

(3) Seismic Analysis. Provided the proposed 100 ton ultimate pile load capacities are achieved and reasonable margin of safety is available, the vertical pile support proposed for the overhang section of the Service Water Pump Structure will provide the support necessary for the structure under combined static and seismic inertial loadings even if the soil under the overhang portion of the structure should liquefy. There is no reason to think this won't be achieved at this time, and the applicant has committed to a load test to demonstrate the pile capacity. The dynamic response of the structure, including the inertial loads for which the structure itself is designed and the mechanical equipment contained therein, would change as a result of the introduction of the piles. Therefore:

(a) Please summarize or provide copies of reports on the dynamic analysis of the structure in its old and proposed configuration. For the latter, provide detailed information on the stiffness assigned to the piles and the way in which the stiffnesses were obtained and show the largest change in interior floor vertical response spectra resulting from the proposed modification. If the proposed configuration has not yet been analyzed, describe the analyses that are to be performed giving particular attention to the basis for calculation or selection, of and the range of numerical stiffness values assigned to the vertical piles.

(b) Provide after completion of the new pile foundation, in accordance with commitment No. 6, item 125, Consumers Power Company memorandum dated 13 March 1980, the results of measurements of vertical applied load and absolute pile head vertical deformation which will be made when the structural load is jacked on the piles so that the pile stiffness can be determined and compared to that used in the dynamic analysis.

42. ~~d.~~ Auxiliary Building Electrical Penetration Areas and Feedwater Isolation Valve Pits.

(1) Settlement. Provide the assumptions, method, computation and estimate of expected allowable lateral and vertical deflections under static and seismic loadings.

(2) Provide the construction plans, and specifications for underpinning operations beneath the Electrical Penetration Area and Feedwater Valve Pit. The requested information to be submitted should cover the following in sufficient details for evaluation:

(a) Details of the temporary dewatering system (locations, depth, size and capacity of wells) including the monitoring program to be required, (for example, measuring drawdown, flow, frequency of observations, etc.) to evaluate the performance and adequacy of the installed system.

(b) Location, sectional views and dimensions of access shaft and drift to and below auxiliary building wings.

(c) Details of temporary surface support system for the valve pits.

~~(d)~~ Dewatering before underpinning is recommended in order to preclude differential settlement between pile and soil supported elements and negative drag forces.

(~~e~~ d) Provide adopted soil properties, method and assumptions used to estimate caisson and/or pile design capacities, and computational results. Provide estimated maximum static and dynamic load (compression, uplift and lateral) to be imposed and the individual contribution (DL, LL, OBE, SSE) on maximum loaded caisson and/or pile. Provide factor of safety against soil failure due to maximum pile load.

(~~f~~ e) Discuss and furnish computations for settlement of the portion of the Auxiliary Building (valve pits, and electrical penetration area) in respect to changed water level as a result of the site dewatering. Include the effect of bouyancy, which was used in previous calculations, and fluctuations in water table which could happen, if dewatering system becomes inoperable.

(~~g~~ f) Discuss protection measures to be required against corrosion, if piling is selected.

(g) Identify specific information, data and method of presentation to be submitted for regulatory review at completion of underpinning operation. This report should summarize construction activities, field inspection records, results of field load tests on caissons and piles and an evaluation of the completed fix for assuring the stable foundation.

43. Borated Water Tanks.

(1) Settlement. The settlement estimate for the Borated Water Storage Tanks furnished by the applicant in response to NRC Question 31 (10 CFR 50.54f) is based upon the results of two plate load tests conducted at the foundation elevation (EL 627.00±) of the tanks. Since a plate load test is not effective in providing information regarding the soil beyond a depth more than twice the diameter of the bearing plate used in the test, the estimate of the settlement furnished by the applicant does not include the contribution of the soft clay layers located at depth more than 5' below the bottom of the tanks (see Boring No. T-14 and T-15, and T-22 thru T-26).

(a) Compute settlements which include contribution of all the soil layers influenced by the total load on the tanks. Discuss and provide for review the analysis evaluating differential settlement that could occur between the ring (foundations) and the center of the tanks.

(b) The bottom of the borated tanks being flexible could warp under differential settlement. Evaluate what additional stresses could be induced in the ring beams, tank walls, and tank bottoms, because of the settlement, and compare with allowable stresses. Furnish the computations on stresses including method, assumptions and adopted soil properties in the analysis.

(2) Bearing Capacity. Laboratory test results on samples from boring T-15 show a soft stratum of soil below the tank bottom. Consideration has not been given to using these test results to evaluate bearing capacity information furnished by the applicant in response to NRC Question 35 (10 CFR 50.54f). Provide bearing capacity computations based on the test results of the samples from relevant borings. This information should include method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and resulting factor of safety for the static and the seismic loads.

44. Underground Diesel Fuel Tank Foundation Design

(1) Bearing capacity. Provide bearing capacity computation based on the test results of samples from relevent borings, including method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and the resulting factor of safety.

(2) Provide tank settlement analysis due to static and dynamic loads including methods, assumptions made, etc.

(3) What will be effects of uplift pressure on the stability of the tanks and the associated piping system if the dewatering system becomes inoperable?

45. ~~6~~. Underground Utilities:

(1) Settlement

(a) Inspect the interior of water circulation piping with video cameras and sensing devices to show pipe cross section, possible areas of crackings and openings, and slopes of piping following consolidation of the plant fill beneath the imposed surcharge loading.

(b) The applicant has stated in his response to NRC Question 7 (10 CFR 50.54f) that if the duct banks remain intact after the preload program has been completed, they will be able to withstand all future operating loads. Provide the results of the observations made, during the preload test, to determine the stability of the duct banks, with your discussion regarding their reliability to perform their design functions.

(c) The response to Question 17 of "Responses to NRC Requests Regarding Plant Fill" states that "there is no reason to believe that the stresses in Seismic Category I piping systems will ever approach the Code allowable." We question the above statement based on the following:

Profile 26" - OHBC-54 on Fig. 19-1 shows a sudden drop of approx. 0.2 feet within a distance of only 20 feet. Using the procedure on p. 17-2,

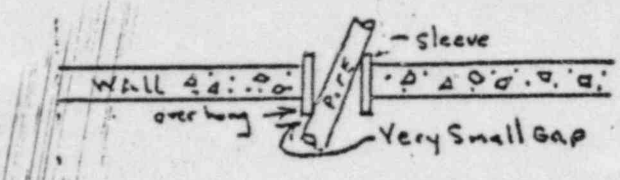
$$\sigma_b = E(e) = E\left(\frac{D}{2R}\right) = E\left(\frac{D}{2}\right)\left(\frac{8\delta}{L^2}\right)$$

$$\sigma_b = 30000 \left(\frac{26}{2}\right)\left[\frac{8(0.2)(12)}{(20x12)^2}\right] = 130.0 \text{ KSI}$$

~~Furthermore, the Eq. 10(a) of Article NC-3652.3, Sec. III, Division 1, of the ASME code requires that some Stress Intensification Factor "i" be assigned to all computed settlement stresses.~~ Yet, Table 17-2 lists only 52.5 KSI as allowable stress for this pipe. This matter requires further review. Please respond to this apparent discrepancy and also specify the location of each computed settlement stress at the pipeline stationing shown on the profiles. More than one critical stress location is possible along the same pipeline.

(d) During the site visit on 19 February 1980, we observed three instances of what appeared to be degradation of rattlespace at penetrations of Category I piping through concrete walls as follows:

West Borated Water Tank - in the valve pit attached to the base of the structure, a large diameter steel pipe extended through a steel sleeve placed in the wall. Because the sleeve was not cut flush with the wall, clearance between the sleeve and the pipe was very small.



Service Water Structure - Two of the service water pipes penetrating the northwest wall of the service water structure had settled differentially with respect to the structure and were resting on slightly squashed short pieces of 2 x 4 placed in the bottom of the penetration. From the inclination of the pipe, there is a suggestion that the portions of the pipe further back in the wall opening (which was not visible) were actually bearing on the invert of the opening. The bottom surface of one of the steel pipes had small surface irregularities around the edges of the area in contact with the 2 x 4. Whether these irregularities are normal manufacturing irregularities or the result of concentration of load on this temporary support caused by the settlement of the fill, was not known.

These instances are sufficient to warrant an examination of those penetrations where Category I pipe derives support from plant fill on one or both sides of a penetration. In view of the above facts, the following information is required.

(1) What is the minimum seismic rattlespace required between a Category I pipe and the sleeve through which it penetrates a wall?

(2) Identify all those locations where a Category I pipe deriving support from plant fill penetrates an exterior concrete wall. Determine and report the vertical and horizontal rattlespace presently available and the minimum required at each location and describe remedial actions planned as a result of conditions uncovered in the inspection. It is anticipated that the answer to Question (1) can be obtained without any significant additional excavation. If this is not the case, the decision regarding the necessity to obtain information at those locations requiring major excavation should be deferred until the data from the other locations have been examined.

(e) Provide details (thickness, type of material etc.) of bedding or cradle placed beneath safety related piping, conduits, and supporting structures. Provide profiles along piping, and conduits alignments showing the properties of all supporting materials to be adopted in the analysis of pipe stresses caused by settlement.

(f) The two reinforced concrete return pipes which exit the Service Water Pump Structure, run along either side of the emergency cooling water reservoir, and ultimately enter into the reservoir, are necessary for safe shutdown. These pipes are buried within or near the crest of Category I slopes that form the sides of the emergency cooling water reservoir. There is no report on, or analysis of, the seismic stability of post earthquake residual displacement for these slopes. While the limited data from this area do not raise the specter of any problem, for an important element of the plant such as this, the earthquake stability should be examined by state-of-the-art methods. Therefore, provide results of the seismic analysis of the slopes leading to an estimate of the permanent deformation of the pipes. Please provide the following: (1) a plan showing the pipe location with respect to other nearby structures, slopes of the reservoir and the coordinate system; (2) cross-sections showing the pipes, normal pool levels, slopes, subsurface conditions as interpreted from borings and/or logs of excavations at (a) a location parallel to and about 50 ft from the southeast outside wall of the service water pipe structure and (b) a location where the cross section will include both discharge structures. Actual boring logs should be shown on the profiles; their offset from the profile noted, and soils should be described using the Unified Soil Classification System; (3) discussion of available shear strength data and choice of strengths used in stability analysis; (4) determination of static factor or safety, critical earthquake acceleration, and location of critical circle; (5) calculation of residual movement by the method presented by Newmark (1965) or Makdisi and Seed (1978); and (6) a determination of whether or not the pipes can function properly after such movements.

46. ~~X~~. Cooling Pond.

(1) Emergency Cooling Pond. In recognition that the type of embankment fill and the compaction control used to construct the retention dikes for the cooling pond were the same as for the problem plant fill, we request reasonable assurance that the slopes of the Category I Emergency Cooling Pond (baffle dike and main dike) are stable under both static and dynamic loadings. We request a revised stability analysis for review, which will include identification of locations analyzed, adopted foundation and embankment conditions (stratification, seepage, etc.) and basis for selection, adopted soil properties, method of stability analysis used and resulting factor of safety with identification of sliding surfaces analyzed. Please address any potential impact on Category I pipes near the slopes, based on the results of this stability study. Recommendations for location of new exploration and testing have been provided in a separate letter.

(2) Operating Cooling Pond. A high level of safety should be required for the remaining slopes of the Operating Cooling Pond unless it can be assured that a failure will not: (a) endanger public health and properties, (b) result in an assault on environment, (c) impair needed emergency access. Recommendations for locations of new borings and laboratory tests have been submitted in a separate letter. These recommendations were made on the assumptions that the stability of the operating cooling pond dikes should be demonstrated.

47. ~~i~~. Site Dewatering Adequacy.

(1) In order to provide the necessary assurance of safety against liquefaction, it is necessary to demonstrate that the water will not rise above elevation 610 during normal operations or during a shutdown process. The applicant has decided to accomplish this by pumping from wells at the site. In the event of a failure, partial failure, or degradation of the dewatering system (and its backup system) caused by the earthquake or any other event such as equipment breakdown, the water levels will begin to rise. Depending on the answer to Question (a) below concerning the normal operating water levels in the immediate vicinity of Category I structures and pipelines founded on plant fill, different amounts of time are available to accomplish repair or shutdown. In response to Question 24 (10 CFR 50.54f) the applicant states "the operating groundwater level will be approximately el 595 ft" (page 24-1). On page 24-1 the applicant also states "Therefore el 610' is to be used in the designs of the dewatering system as the maximum permissible groundwater level elevation under SSE conditions." On page 24-15 it is stated that "The wells will fully penetrate the backfill sands and underlying natural sands in this area." The bottom of the natural sands is indicated to vary from elevation 605 to 580 within the plant fill area according to Figure 24-12. The applicant should discuss and furnish response to the following questions:

(a) Is the normal operating dewatering plan to (1) pump such that the water level in the wells being pumped is held at or below elevation 595 or (2) to pump as necessary to hold the water levels in all observation wells near Category I Structures and Category I Pipelines supported on plant fill at or below elevation 59[illegible], (3) to pump as necessary to hold water levels in the wells mentioned in (2) above at or below elevation 610, or (4) something else? If it is something else, what is it?

(b) In the event the water levels in observation wells near Category I Structures or Pipelines supported on plant fill exceed those for normal operating conditions as defined by your answeer to Question (a) what action will be taken? In the event that the water level in any of these observation wells exceeds elevation 610, what action will be taken?

(c) Where will the observation wells in the plant fill area be located that will be monitored during the plant lifetime? At what depths will the screened intervals be? Will the combination of (1) screened interval in cohesionless soil and (2) demonstration of timely response to changes in cooling pond level prior to drawdown be made a condition for selecting the observation wells? Under what conditions will the alarm mentioned on page 24-20 be triggered? What will be the response to the alarm? A worst case test of the completed permanent dewatering and groundwater level monitoring systems could be conducted to determine whether or not the time required to accomplish shutdown and cooling is available. This could be done by shutting off the entire dewatering system when the cooling pond is at elevation 627 and determining the water level versus time curve for each observation well. The test should be continued until the water level under Category I structure, whose foundations are potentially liquefiable, reaches elevation 610 (the normal water level) or the sum of the time intervals allotted for repair and the time interval needed to accomplish shutdown (should the repair prove unsuccessful) has been exceeded, whichever occurs first. In view of the heterogeneity of the fill, the likely variation of its permeability and the necessity of making several assumptions in the analysis which was presented in the applicant's response to Question 24a, a full-scale test should give more reliable information on the available time. In view of the above the applicant should furnish his response to the following:

If a dewatering system failure or degradation occurs, in order to assure that the plant is shutdown by the time water level reaches elevation 610, it is necessary to initiate shutdown earlier. In the event of a failure of the dewatering system, what is the water level or condition at which shutdown will be initiated? How is that condition determined? An acceptable method would be a full-scale worst-case test performed by shutting off the entire dewatering system with the cooling pond at elevation 627 to determine, at each Category I Structure deriving support from plant fill, the water level at which a sufficient time window still remains to accomplish shutdown before the water rises to elevation 610. In establishing the groundwater level or condition that will trigger shutdown, it is necessary to account for normal surface water inflow as well as groundwater recharge and to assume that any additional action taken to repair the dewatering system, beyond the point in time when the trigger condition is first reached, is unsuccessful.

(2) As per applicant response to NRC Question 24 (10 CFR 50.54f) the design of the permanent dewatering system is based upon two major findings: (1) the granular backfill materials are in hydraulic connection with an underlying discontinuous body of natural sand, and (2) seepage from the cooling pond is restricted to the intake and pump structure area, since the plant fill south of Diesel Generator Building is an effective barrier to the inflow of the cooling pond water. However, soil profiles (Figure 24-2 in the "Response to NRC Requests Regarding Plant Fill"), pumping test time-drawdown graphs (Figure 24-14), and plotted cones of influence (Figure 24-15) indicate that south of Diesel Generator Building, the plant fill material adjacent to

the cooling pond is not an effective barrier to inflow of cooling pond water. The estimated permeability for the fill material as reported by the applicant is 8 feet/day and the transmissivities range from 29 to 102 square feet/day. Evaluate and furnish for review the recharge rate of seepage through the fill materials from the south side of the Diesel Generator Building on the permanent dewatering system. This evaluation should especially consider the recovery data from PD-3 and complete data from PD-5.

(3) The interceptor wells have been positioned along the northern side of the Water Intake Structure and service water pump structures. The calculations estimating the total groundwater inflow indicate the structures serve as a positive cutoff. However, the isopachs of the sand (Figures 24-9 and 24-10) indicate 5 to 10 feet of remaining natural sands below these structures. The soil profile (Figure 24-2) neither agrees nor disagrees with the isopachs. The calculations for total flow, which assumed positive cutoff, reduced the length of the line source of inflow by 2/3. The calculations for the spacing and positioning of wells assumed this reduced total flow is applied along the entire length of the structures. Clarify the existence of seepage below the structures, present supporting data and calculations, and reposition wells accordingly. Include the supporting data such as drawdown at the interceptor wells, at midway location between any two consecutive wells, and the increase in the water elevations downstream of the interceptor wells. The presence of structures near the cooling pond appears to have created a situation of artesian flow through the sand layer. Discuss why artesian flow was not considered in the design of the dewatering system.

(4) Provide construction plans and specification of permanent dewatering system (location, depths, size and capacity of wells, filterpack design) including required monitoring program. The information furnished in response of NRC Question 24 (10 CFR 50.54f) is not adequate to evaluate the adequacy of the system.

(5) Discuss the ramifications of plugging or leaving open the weep holes in the retaining wall at the Service Water Building.

(6) Discuss in detail the maintenance plan for the dewatering system.

(7) What are your plans for monitoring water table in the control tower area of the Auxiliary Building?

(8) What measures will be required to prevent incrustation of the pipings of the dewatering system. Identify the controls to be required during plant operation (measure of dissolved solids, chemical controls). Provide basis for established criteria in view of the results shown on Table 1, page 23 of tab 147.

(9) Upon reaching a steady state in dewatering, a groundwater survey should be made to confirm the position of the water table and to insure that no perched water tables exist.

Dewatering of the site should be scheduled with a sufficient lead time before plant start up so that the additional settlement and its effects (especially on piping) can be studied. Settlement should be closely monitored during this period.

Provide your plans for conducting this groundwater survey.

N/A

j. Liquefaction Potential.

An independent Seed-Idriss Simplified Analysis was performed for the fill area under the assumption that the groundwater table was at or below elevation 610. For 0.19 g peak ground surface accceleration, it was found that blow counts as follows were required for a factor of safety of 1.5:

Elevation ft	Minimum SPT Blow Count*1 For F.S. = 1.5
610	14
605	16
600	17
595	19

The analysis was considered conservative for the following reasons (a) no account was taken of the weight of any structure, (b) liquefaction criteria for a magnitude 6 earthquake were used whereas an NRC memorandum of 17 Mar 80 considered nothing larger than 5.5 for an earthquake with the peak acceleration level of 0.19 g's, (c) unit weights were varied over a range broad enough to cover any uncertainty and the tabulation above is based on the most conservative set of assumptions. Out of over 250 standard penetration tests on cohesionless plant fill or natural foundation material below elevation 610, the criteria given above are not satisfied in four tests in natural materials located below the plant fill and in 23 tests located in the plant fill. These tests involve the following borings:

SW3, SW2, DG-18, AX 13, AX 4, AX 15, AX 7, AX 5, AX 11, DG 19, DG 13, DG 7, DG 5, D 21, GT 1, 2.

Some of the tests on natural material were conducted at depths of at less than 10 ft before approximately 35 ft of fill was placed over the location. Prior to comparison with the criteria these tests should be multiplied by a factor of about 2.3 to account for the increase in effective overburden pressure that results from the placement and future dewatering of the fill.

1*For M = 7.5, blow counts would increase by 30%.

Of the 23 tests on plant fill which fail to satisfy the criteria, most are near or under structures where remedial measures alleviating necessity for support from the fill are planned. Only 4 of the tests are under the Diesel Generator Building (which will still derive its support from the fill) and 3 others are near it. Because these locations where low blow counts were recorded are well separated from one another and are not one continuous stratum but are localized pockets of loose material, no failure mechanism is present.

In view of the large number of borings in the plant fill area and the conservatism adopted in analysis, these few isolated pockets are no threat to plant safety. The fill area is safe against liquefaction in a Magnitude 6.0 earthquake or smaller which produces a peak ground surface acceleration of 0.19 g or less provided the groundwater elevation in the fill is kept at or below elevation 610.

48. Seismic analysis of structures on plant fill material.

(1) Category I Structures. From Section 3.7.2.4 of the FSAR it can be calculated that an average V_s of about 1350 ft/sec was used in the original dynamic soil structure interaction analysis of the Category I structures. This is confirmed by one of the viewgraphs used in the 28 February Bechtel presentation. Plant fill V_s is clearly much lower than this value. It is understood from the response to Question 13 (10 CFR 50.54f) concerning plant fill that the analysis of several Category I structures are underway using a lower bound average V_s = 500 ft/sec for sections supported on plant fill and that floor response spectra and design forces will be taken as the most severe of those from the new and old analysis. The questions which follow are intended to make certain if this is the case and gain an understanding of the impact of this parametric variation in foundation conditions.

(a) Discuss which Category I structures have been and/or will be reanalyzed for changes in seismic soil structure interaction due to the change in plant fill stiffness from that envisioned in the original design. Have any Category I structures deriving support from plant fill been excluded from reanalysis? On what basis?

(b) Tabulate for each old analysis and each reanalysis, the foundation parameters (v_s, ν and ρ) used and the equivalent spring and damping constants derived therefrom so the reviewer can gain an appreciation of the extent of parametric variation performed.

(c) Is it the intent to analyze the adequacy of the structures and their contents based upon the envelope of the results of the old and new analyses? For each structure analyzed, please show on the same plot the old, new, and revised enveloping floor response spectra so the effect of the changed backfill on interior response spectra predicted by the various models can be readily seen.

(2) Category I retaining wall near the southeast corner of the Service Water Structure. This wall is experiencing some differential settlement. Boring information in Figure 24-2 (Question 24, Volume 1 Responses to NRC Requests Regarding Plant Fill) suggests the wall is founded on natural soils and backfilled with plant fill on the land side. Please furnish details clarifying the following:

(a) Is there any plant fill underneath the wall? What additional data beyond that shown in Figure 24-2 support your answer?

(b) Have or should the design seismic loads (FSAR Figure 2.5-45) be changed as a result of the changed backfill conditions?

(c) Have or should dynamic water loadings in the reservoir be considered in the seismic design of this wall? Please explain the basis of your answer.

5. In your response for the comments and questions in paragraph 4 above, if you feel that sufficiently detailed information already exists on the Midland docket that may have been overlooked, please make reference to that information. Resolution of issues and concerns will depend on the expeditious receipt of data mentioned above. Contact Mr. Neal Gehring at FTS 226-6793 regarding questions.

FOR THE DISTRICT ENGINEER:

D. McCallister

P. McCALLISTER
Chief, Engineering Division

3/138

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO:
WESGA

10 Aug. 81

SUBJECT: Midland Nuclear Power Plant Ground Motion Study

Commander
US Army Engineer District, Detroit
Attn: NCEED-T/Mr. Neil Gehrig
PO Box 1027
Detroit, MI 48231

DRAFT

1. Reference memorandum for record dated 3 August 1981, subject: Effect of Plant Fill on Seismic Ground Motion Environment at the Midland Michigan Nuclear Power Plant by Dr. Paul F. Hadala (Incl 1). This memorandum is an interim report under your IAO Number CE-1A-80-047.

2. If you have any questions, please feel free to contact Dr. Hadala at FTS 542-3475.

FOR THE COMMANDER AND DIRECTOR:

1 Incl
as

CF w/incl:
Mr. Jim Simpson (NCDED-G)
Dr. Lyman Heller (NRC)
Mr. Joe Kane, (NRC) (4 copies)

H. B. SIMMONS
Engineer
Acting Technical Director

DEPARTMENT OF THE ARMY
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IN REPLY REFER TO: WESGA

DRAFT

3 August 1981

MEMORANDUM FOR RECORD

SUBJECT: Effect of Plant Fill on Seismic Ground Motion Environment at the Midland Michigan Nuclear Power Plant

1. INTRODUCTION. Under IAO Number CE-IA-80-047 from the Detroit District CE, (who are in turn supporting the Site Analysis Branch of the Nuclear Regulatory Commission), the undersigned has participated in a continuing review of the plant fill at Midland Nuclear Power Plant. My participation has been limited to seismic considerations. References 1 and 2 addressed a number of questions but were primarily concerned with evaluation of liquefaction potential. This memorandum, which consists of two parts, addresses the effect of the plant fill on the earthquake induced ground motion environment. Part I is a review of Appendix B of Reference 3 requested by NRC. Part II consists of a series of SHAKE Code (Reference 4) one-dimensional wave propagation calculations performed by WES to study the effects of changing some of the parameters used in similar calculations in Reference 3.

2. PART I - REVIEW OF WESTON'S REPORT. In the main body of Reference 3, the only portion this writer is competent to evaluate is Section 2.2. The P- and S-wave velocity profile given in Figure 1 of Reference 3 and plotted in Figure 4 of that reference are considered reasonable. Inclosure 6 of Reference 1 shows the S-wave velocity (V_s) data for the plant fill to be consistent with the 440-1060 ft/sec range adopted by Weston. A closer look at Incl 6 of Reference 1 indicates the vast majority of the data lies between 575 and 900 ft/sec and that there is a slight trend of increase in V_s with depth. In the upper part of the fill, 700 ft/sec is an upper bound to nearly all the data. The P- and S-wave velocity profiles used for that portion of the profile below original ground surface come from the Weston preconstruction geophysical tests (see FSAR section 2.5.4.7.2) and the effect of the addition of the fill should be only a very slight increase in V_s. The amount of the increase is judged to be so small that it could not be resolved because it is below the sensitivity of the seismic test methods (see Reference 7). This reviewer is satisfied that the seismic profile used in the selection of records for use in the development of a site specific response spectra for the plant fill is physically reasonable for the site and consistent with the available data.

3. Appendix B of Reference 3 contains the results of a series of SHAKE Code one-dimensional wave propagation analyses performed to study "possible local amplification effects on earthquake ground motion at the Midland Plant Site."

Incl 2

DRAFT

WESGA 3 August 1981

SUBJECT: Effect of Plant Fill on Seismic Ground Motion Environment at the Midland Michigan Nuclear Power Plant

In Section 2.0 of Appendix B and Figures B-1, B-3, B-4, and B-5, soil profiles and properties used in the analyses are presented. This reviewer has no disagreement with the range of layered systems investigated, the densities, damping, and shear-wave velocities used. However, by specifying modulus factors, the authors of Appendix B effectively negated their choice of shear-wave velocities and substituted much lower values for the plant fill as shown in Figure 1. This point was first pointed out to the authors and the applicant's representatives at a meeting at NRC in Bethesda, MD on 30 June 1981.

4. SHAKE, when given modulus factors, uses them in preference to V_s to compute the initial shear modulus G_o. Since the shear modulus-shear strain curve for each layer which is used in the code is normalized to G_o, shear moduli for all strain levels are controlled by its specification. Figure 1 also shows that the upper 50 ft of the till was represented as being five times stiffer than it actually was. This reviewer verified that the solid curve in Figure 1 was what was actually used for Case A by performing a duplicate of one of the authors' calculations and reproducing the results in Figure B-8 of Reference 3.

5. The normalized shear modulus and damping versus strain curves used in the Appendix B calculations were not given in the report. However, they were supplied to the reviewer by letter (Reference 6) and were determined to be those developed in Reference 8. The curves are shown in Figures 2 and 3. The use of these curves is considered an acceptable state-of-the-art practice.

6. In Reference 6, the authors provided the reviewers with a set of new results from SHAKE Code calculations for the Case A profile in which the modulus factors were adjusted to produce initial shear moduli consistent with the shear-wave velocities. This revised set of properties is referred to as Case A, variation 1. Figure 4 shows the effect of changing modulus factors on the amplification factor versus frequency curve generated by the SHAKE code calculation for Case A soil profile. The El Centro 5/18/1940 acceleration record for the Imperial Valley station scaled to 0.12 g was used as an outcrop of the Saginaw bedrock (which underlies the profile at a depth of 371 ft) in this calculation. As shown in Figure 4, there are substantial percentage increases in amplification factors at frequencies between 1/2 and 10 hz when more realistic modulus factors are used.

7. In References 3 and 6, a substantial number of computer code parameter studies were performed. A wide variation in soil profiles was considered along with four different earthquake records. In all cases the records were scaled to 0.12 g and input as outcrop motions for an outcrop of the Saginaw Formation. In the case of the El Centro record, which was recorded on deep alluvium, a question arises as to the appropriateness of using this record as a bedrock outcrop. The other three records used are all from the Lytle Creek Earthquake (Richter magnitude 5.4). Two of the three records have been classified as being recorded at "intermediate" sites and one, the Cedar Springs, Allen Ranch record is from a rock site. The use of these records as rock outcrop motions is more realistic than is the use of the El Centro record. The effect of the choice of the layer selected for the outcrop will be examined in Part II.

DRAFT

WESGA 3 August 1981

SUBJECT: Effect of Plant Fill on Seismic Ground Motion Environment at the Midland Michigan Nuclear Power Plant

8. Also, the El Centro record, which was used in most of the Reference 3 SHAKE calculations is from the near field of an earthquake of magnitude 6.7 and is rich in low frequencies. The site specific response spectra developed in the report was based on a 5.3 m_b earthquake adjacent to the Midland site. Why the record for a substantially larger event was used as the baseline or pivot point of the parameter study is unknown. The other three records are from an earthquake of the appropriate magnitude.

9. In the final analysis, all of the work in Appendix B of Reference 3 is summarized in a plot of the ratio of response spectra for the top fill to that for the original ground surface. See Figure 5, which is a copy of Figure 9 of Reference 3. The fact that the analytically developed curve (a) lies below the curve for the comparable ratio of empirically developed site specific response spectra (SSRS) and, (b) has a similar shape been used as an argument that the empirically based SSRS are physically reasonable and are more conservative than the top of fill spectrum that would have been obtained via a SHAKE code calculation. The additional ratio of response spectra calculations furnished in Reference 6 are superimposed on Figure 5 and show that the effect of revising the site properties was to increase the calculated ratios of response spectra (RRS) in the 1/2 to 10Hz range. However, the analytically developed RRS still (with minor exception) lie below the RRS for the empirically developed site specific response spectra.

10. Figure 6 shows the 84th percentile, 5 percent damped site specific response spectra developed via empirical methods in References 3 and 5 for the top of plant fill and the original ground surface, respectively. Figure 6 indicates that the empirical approach produces substantial amplification of response at periods greater than 0.25 sec (frequencies less than 4 hz), which is supported by the additional calculations reported in Referece 6 and displayed in Figure 5.

11. PART II - SHAKE CODE PARAMETER STUDY. One of the objectives of the parameter study was to determine the differences between the ground motion environment at the top of plant fill and the original ground surface as calculated under the assumption of one-dimensional vertical shear wave propogation and all of the other assumptions implicit in the use of the SHAKE Computer code (see Reference 4). Another was to study effects of variations in the soil properties assumed for the plant fill. A third objective was to study the effect of varying the input accelerogram on ground motion. The second and third objectives have already been addressed in Reference 3 and the work reported herein merely extends the range of that parameter study. The fourth objective was to examine the effect of choice of outcropping layer. In Reference 3, all calculations were made assuming that the accelerogram represented the motion at a bedrock (Saginaw Formation) outcrop near the site. An equally (or perhaps more) reasonable assumption for some of the accelerograms is that they represent original ground surface (top of till) motions. Table 1 is a list of the thirteen SHAKE code calculations performed that shows the variation of parameters performed. Table 2 describes the earthquake accelerograms used as input.

DRAFT

12. The reference point for the parameter study reported herein is the Case A soil profile given in Reference 3 (see also Table 3), the El Centro earthquake accelerogram described in Table 2, which was scaled to 0.12 g in this study as well as in Reference 3. Each SHAKE run used an earth pressure at rest coefficient of 0.45 and the following options (see Page 15 of Reference 4): 1, 2, 3, 4, 5, 8, 9, and 15. The first calculation performed (Run 1) was performed to duplicate one of the calculations in Reference 3. It used the Case A soil profile (Table 3 and dotted line in Figure 1) even though it was considered much too soft in the fill layers and too stiff in the upper part of the glacial till. It was used because it was the one used in Reference 3. The modulus and damping versus strain relations used in Option 8 are those given in the sample data set on Page 40 of Reference 4 and are essentially those shown in Figures 2 and 3. The accelerogram (El Centro, SOOE, 5/18/40) scaled to 0.12 g was used only because of its choice by the authors of Reference 3. As shown in Table 2, it is a long record and was recorded in the near field region of an earthquake substantially larger than the safe shutdown earthquake for the Midland site. The outcrop location in the rock of the Saginaw Formation (which underlies the Case A profile at a depth of 371 ft) was chosen again because it was used in Reference 3. The actual accelerogram was recorded at the surface of a deep soft alluvial deposit and its frequency content is probably not appropriate for a rock outcrop. This calculation (the solid line curve in Figure 7) duplicated the amplification curve shown in Figure B-8 of Reference 3.

13. Figure 7 shows the effect of some variation in site profile on the amplification factor curve for the motion at the top of fill with respect to outcrop motion. Figure 8 shows the effects of the same variations in site profile on 5 percent damped shock spectra at the ground surface. Profile F, whose properties are given in Table 4 and whose low strain level shear modulus vs depth relationship is given by the solid line in Figure 1 represents this writer's judgment of the lower limit of fill stiffness and the best estimate of the stiffness of the natural ground. Profile G, whose properties are given in Table 5, represents this writer's judgment on the upper limit of the fill stiffness. Below the plant fill it is the same as F. Profile H (see Table 6) is the same site profile as F and G, but with the fill removed. The top of the profile in this case is the original ground surface. Case I is the same as A but with the fill removed.

14. Figures 7 and 8 show that changing profiles make substantial differences in the results of the calculations. The effects are best shown on the shock spectra in Figure 8. When profiles A and I are compared (wrong site properties with and without fill) it is seen that the effect is also deamplification at frequencies above 1 Hz but to a lesser degree. Figure 7 shows that in the lower frequency range (below 1 Hz) the effect of the presence of the fill (either Case A or F) is substantial amplification over Case I.

15. Figure 7 also shows for Profile F what happens when the outcrop location is changed to the top of till (i.e. the original ground surface), a location considered more realistic for a ground motion record recorded on the surface of

DRAFT

deep alluvium. The nature of the amplification curve changes radically, the amplification factor remains close to unity at low frequencies and drops toward zero at the higher frequencies. This is in accord with common sense, as the top of fill is only 30 ft above the outcrop's elevation in the profile and radically different low frequency motions at the two points are a physical impossibility.

16. While this writer considers the spectra in Figure 8 for the A and I profiles inappropriate because of the bedrock outcrop location used in the calculation and the size of earthquake in which the record was obtained, the spectra in Figure 8 can be compared with the 84th percentile empirically developed spectra shown in Figure 6 by overlaying the transparency of Figure 6 given at the end of this report on Figure 8. In fact, the same comparison can be made with any of the spectra which follow.

17. Figure 9 shows the spectra for the top of the F, G, and H profiles calculated if the El Centro record is used as a till outcrop which is physically more realistic than using it as a rock outcrop record. In the 1 to 5 Hz frequency range, the Profile G spectra is almost the same as that for the original ground surface while the spectra for Profile F shows modest amplification. While the El Centro record is from an earthquake of larger magnitude than the safe shutdown earthquake, it is still of interest to compare Figure 9 with Figure 6. All three spectra exceed the empirical one for the top of fill at frequencies below 2 Hz.

18. The NRC staff requested a series of SHAKE calculations be performed with an accelerogram from the Fogoria-Cornino station and the 9/11/76 earthquake at Friuli, Italy. This record was among those used in the development of the empirical site specific response spectra. The card deck for this record was furnished to WES by NRC. Since the record was recorded in the surface and the site profile bears some general similarity in layering and stiffness to the Midland site, the original ground surface is the appropriate outcrop layer and was used as such in Runs 4, 5, and 11 (Table 1). Results from these three calculations are shown in Figures 10 and 11. Because of the physical proximity of the outcropping layer to the top of the fill, only limited amplification can occur. As expected, Profile F, the softer profile shows more amplification than Profile G. The peaks as shown in Figure 10 are at frequencies which are bounded from above by those obtained from the formula:

$$f = \frac{V_s}{4H}$$

Where H = layer thickness = 31 ft

f = frequency

V_s = average small strain level shear-wave velocity of fill (670 ft/sec for Profile F and 770 ft/sec for Profile G)

Since Profile F is softer and the response of the fill is, on a relative scale, much further into the nonlinear regime, the greater overprediction by the formula for Case F appears reasonable.

DRAFT

19. Figure 11 shows the spectra for the same 3 cases. The top of fill spectra for Profile G is essentially the same as that for the original ground surface (Profile H) while that for Profile F shows some amplification in the 2-5 Hz frequency range. Since the acceleration record, soil profiles and outcrop are all physically reasonable with respect to the assumptions made in the development of the empirical site specific response spectra, a comparison with transparency of Figure 6 is appropriate. Such a comparison shows that except for two very slight excursions, the spectra in Figure 11 are all below the 84th percentile empirical spectra for the top of fill.

20. The third acceleration record the NRC staff requested be used in SHAKE calculations was the Temblor record from the Parkfield earthquake (see Table 2). While the record was obtained in an earthquake of a magnitude close to that of the safe shutdown earthquake, this record was recorded at a rock site in fairly close proximity to the fault rupture. At the 30 June meeting at NRC, there was much discussion over the appropriateness (and lack thereof) of the Parkfield records for the Midland site. There is additional discussion on the subject in Reference 9. This issue falls within the expertise of the seismologist and the writer is not a seismologist. However, what is clear is that if the Temblor record is to be used in SHAKE calculations, it should be used as a rock outcrop record.

21. Figures 12 and 13 show the amplification factors and shock spectra computed using the Parkfield-Temblor record. The figure shows substantial amplification due to the fill at frequencies up to 4 Hz and the spectra showed in Figure 12 exceed those in Figure 6 substantially in the range below 5 Hz. It should be expected that, if the seismologists decide the Parkfield-Temblor record and others like it should make up a substantial part of the data set for an empirical analysis, then the empirically developed spectra for the plant fill would exceed that in Reference 3. Reference 9 showed that such was also the case for the spectra for the original ground surface.

22. Figure 14 shows the largest effect of any single variation made in this study. Changing from a rock outcrop (which is appropriate for the Temblor record) to a till outcrop substantially decreased the shock spectra amplitudes in the entire region of interest.

23. Figures 15 and 16 show ratios of response spectra calculated by the same equation used in Reference 3. These ratios are the ratio of the spectra for the top of fill to that for the outcrop motion. With only one exception, those cases where the writer judged the outcrop layer was reasonably matched to the actual conditions at the accelerograph secondary station are shown. The exception is the case of Runs 1 and 13 on Figure 16. This is shown because it was one of the cases examined in Reference 3. While the new calculation is close to the results in Figure B-14 of Reference 3, it is not an exact match. This is probably because the calculations which resulted in Figures 15 and 16 were performed with an amplification factor data set which had less resolution in the frequency domain ($\Delta f = 0.1$ Hz) and no smoothing of the input. The key point is that except for the cases involving the Parkfield-Temblor record, all lie

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well below the curve for the ratios of site specific response spectra given in Figure 9 of Reference 3 (also Figure 5 of this memorandum). It is also of interest to note that the ratios are generally closer to unity over the entire frequency range for the till outcrop cases than for those with the Saginaw outcrop.

24. SUMMARY. The analytical parameter study has shown that the effect of the addition of the fill on the ground motion environment can be different depending on the stiffness of the fill, the acceleration record chosen, or which layer is chosen for the outcrop. The largest variations observed in the top of fill ground shock environment occurred due to the change from Profiles A to F (or I to H). This represents a major change, not just in the fill but in the top 50 ft of original ground. The combined effects of making the site properties more realistic was to raise the shock environment (Figure 8). The second major variation was the result of changing outcrops. In general, changing from Saginaw to till as the outcropping layer decreased the shock environment substantially (compare Figures 8 and 9; see also Figure 14). The effect of stiffening the fill only was to decrease the shock environment at the top of the fill (Figure 11) by a modest amount.

25. The 5 percent damped shock spectra calculated for the top of the fill for all cases involving the Fogaria record were below the empirically developed 84th percentile site specific response spectra (SSRS). The use of the El Centro record as a till outcrop and realistic site properties produced spectra which exceeded the SSRS to a modest degree in a limited frequency range. The spectra calculated using the Parkfield-Temblor record as a rock outcrop significantly exceeded the SSRS. Whether the El Centro and Parkfield records are reasonable ones to use in the first place has been questioned. This writer cannot answer that question; it should be posed to the seismologists.

26. All ratios of response spectra (RRS) calculated except those for the Parkfield-Temblor record fell within the envelope of the RRS from the Reference 3 study.

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Assistant Chief
Geotechnical Laboratory

3 Incl:
1. Tables
2. Figures
3. References

Incl 1 to Incl 2

DRAFT

Table 1

SHAKE CODE PARAMETER STUDY

Run Number		Soil Profile	Record	Outcrop Assumed	Remarks
1		A	El Centro	Saginaw	Duplicate of Run From Reference 3,
2	.76/.93	F	El Centro	Till	More Realistic Outcrop, Vary Soil Profile
3		G	El Centro	Till	More Realistic Outcrop, Vary Soil Profile
4	.76/.85	F	Fogaria	Till	Vary Soil Profile
5	.76/.86	G	Fogaria	Till	
6		F	Parkfield-Temblor	Saginaw	Vary Soil Profile
7		G	Parkfield-Temblor	Saginaw	
8		F	El Centro	Saginaw	Compare with 1 (Soil Profile), Compare with 2 (Outcrop)
9	.76/.91	F	Parkfield-Temblor	Till	Compare with 6 (Outcrop)
10		H	El Centro	Till	Compare with 2 and 3 (Natural Ground vs Top of Fill)
11		H	Fogaria	Till	Compare with 4 and 5 (Natural Ground vs Top of Fill)
12		H	Parkfield-Temblor	Saginaw	Compare with 6 and 7 (Natural Ground vs Top of Fill)
13		I	El Centro	Saginaw	Compare with 1 (Natural Ground vs Top of Fill)

Table 2

GROUND MOTION RECORDS USED

Source Of Record *	Earthquake Mag. Date	Station	Component	a_{max} Before Scaling g.	Record Length sec.	Time Interval sec.	a_{max} After Scaling g.
CIT Vol II	El Centro 6.7_R 5/18/40	Imperial Valley #117	SOOE	0.35	53.8	0.02	0.12
Card Deck Provided by NRC	Friule After-shock 5.0_b 9/11/76	Fogaria-Cornino	EW	0.11	15.0	0.005	0.12
CIT Vol II	Parkfield 5.6 6/27/66	Temblor	525E	0.35	30.4	0.25	0.12

* All records were run without any further attempt at baseline connection.

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Table 3

****** OPTION 3 *** READ SOIL PROFILE

NEW SOIL PROFILE NO. 0 IDENTIFICATION CASE A

NUMBER OF LAYERS 8 DEPTH TO BEDROCK 371.00
NUMBER OF FIRST SUBMERGED LAYER 3 DEPTH TO WATER LEVEL 13.33

LAYER	TYPE	FACTOR MOD.	FACTOR DAMP.	THICKNESS FT	DEPTH FT	EFF. PRESS.	MODULUS KSF	DAMPING	UNIT WEIGTH KIPS/FT³	SHEAR VEL FT/SEC
1	1	0.10	1.00	6.67	3.33	0.40	1232.	0.050	0.1200	575.
2	1	0.10	1.00	6.67	10.00	1.20	1232.	0.050	0.1200	575.
3	1	0.10	1.00	6.67	16.67	1.79	1232.	0.050	0.1200	575.
4	1	0.50	1.00	5.50	22.75	2.14	2096.	0.050	0.1200	750.
5	1	0.50	1.00	5.50	28.25	2.46	2096.	0.050	0.1200	750.
6	1	4.00	1.00	10.00	36.00	2.91	2693.	0.050	0.1200	850.
7	1	4.00	1.00	10.00	46.00	3.48	2693.	0.050	0.1200	850.
8	1	6.00	1.00	10.00	56.00	4.06	2693.	0.070	0.1200	850.
9	1	6.00	1.00	10.00	66.00	4.63	2693.	0.070	0.1200	850.
10	1	6.00	1.00	10.00	76.00	5.21	2693.	0.070	0.1200	850.
11	1	8.00	1.00	45.00	103.50	7.13	22179.	0.070	0.1450	2300.
12	1	8.00	1.00	45.00	148.50	10.40	22179.	0.070	0.1450	2300.
13	1	8.00	1.00	50.00	196.00	13.85	37733.	0.070	0.1450	3000.
14	1	8.00	1.00	50.00	246.00	17.48	37733.	0.070	0.1450	3000.
15	2	1.13	1.00	50.00	296.00	21.11	37733.	0.070	0.1450	3000.
16	2	1.13	1.00	50.00	346.00	24.74	37733.	0.070	0.1450	3000.
17	BASE	5.00					112578.	0.	0.1450	5000.

****** OPTION 4 *** OBTAIN STRAIN COMPATIBLE SOIL PROPERTIES

MAXIMUM NUMBER OF ITERATIONS = 10
MAXIMUM ERROR IN PERCENT = 5.00
FACTOR FOR EFFECTIVE STRAIN IN TIME DOMAIN = 0.65

LAYER	TYPE	DEPTH	MAX SHEAR MOD	MAX SHEAR VEL
1	1	3.333	230.00000	248.42839
2	1	10.000	230.00000	248.42839
3	1	16.667	230.00000	248.42839
4	1	22.750	1150.00000	555.50278
5	1	28.250	1150.00000	555.50278
6	1	36.000	9200.00000	1571.19911
7	1	46.000	9200.00000	1571.19911
8	1	56.000	13800.00000	1924.31805
9	1	66.000	13800.00000	1924.31805
10	1	76.000	13800.00000	1924.31805
11	1	103.500	18400.00000	2094.93216
12	1	148.500	18400.00000	2094.93216
13	1	196.000	18400.00000	2094.93216
14	1	246.000	18400.00000	2094.93216
15	2	296.000	7967.54014	1478.72562
16	2	346.000	8627.68848	1434.52553

Table 4

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****** OPTION 2 *** READ SOIL PROFILE

NEW SOIL PROFILE NO. 0 IDENTIFICATION CASE F

NUMBER OF LAYERS 9 DEPTH TO BEDROCK 471.00
NUMBER OF FIRST SUBMERGED LAYER 3 DEPTH TO WATER LEVEL 13.33

LAYER	TYPE	FACTOR MOD.	FACTOR DAMP.	THICKNESS	DEPTH	EFF. PRESS.	MODULUS KIPS/SQ FT	DAMPING	UNIT WEIGTH KIPS/FT³	SHEAR VEL FT/SEC
1	1	0.54	1.00	6.67	3.33	0.40	1232.	0.050	0.1200	575.
2	1	0.54	1.00	6.67	10.00	1.20	1232.	0.050	0.1200	575.
3	1	0.54	1.00	6.67	16.67	1.79	1232.	0.050	0.1200	575.
4	1	0.91	1.00	5.50	22.75	2.14	2096.	0.050	0.1200	750.
5	1	0.91	1.00	5.50	28.25	2.46	2096.	0.050	0.1200	750.
6	1	1.17	1.00	10.00	36.00	2.91	2693.	0.050	0.1200	850.
7	1	1.17	1.00	10.00	46.00	3.48	2693.	0.050	0.1200	850.
8	1	1.17	1.00	10.00	56.00	4.06	2693.	0.070	0.1200	850.
9	1	1.17	1.00	10.00	66.00	4.63	2693.	0.070	0.1200	850.
10	1	1.17	1.00	10.00	76.00	5.21	2693.	0.070	0.1200	850.
11	1	9.64	1.00	45.00	103.50	7.13	22179.	0.070	0.1450	2300.
12	1	9.64	1.00	45.00	148.50	10.40	22179.	0.070	0.1450	2300.
13	1	16.41	1.00	50.00	196.00	13.45	37733.	0.070	0.1450	3000.
14	1	16.41	1.00	50.00	246.00	17.48	37733.	0.070	0.1450	3000.
15	2	5.13	1.00	100.00	321.00	22.92	37733.	0.070	0.1450	3000.
16	2	4.47	1.00	100.00	421.00	30.18	37733.	0.070	0.1450	3000.
17	BASE	5.00					112578.	0.	0.1450	5000.

****** OPTION 4 *** OBTAIN STRAIN COMPATIBLE SOIL PROPERTIES

MAXIMUM NUMBER OF ITERATIONS = 10
MAXIMUM ERROR IN PERCENT = 5.00
FACTOR FOR EFFECTIVE STRAIN IN TIME DOMAIN = 0.65

LAYER	TYPE	DEPTH	MAX SHEAR MOD	MAX SHEAR VEL
1	1	3.333	1232.80000	575.15331
2	1	10.000	1232.80000	575.15331
3	1	16.667	1232.80000	575.15331
4	1	22.750	2095.29999	749.82587
5	1	28.250	2095.29999	749.82587
6	1	36.000	2693.29999	850.11890
7	1	46.000	2693.29999	850.11890
8	1	56.000	2693.29999	850.11890
9	1	66.000	2693.29999	850.11890
10	1	76.000	2693.29999	850.11890
11	1	103.500	22178.89990	2300.01703
12	1	148.500	22178.89990	2300.01703
13	1	196.000	37733.79980	3000.03500
14	1	246.000	37733.79980	3000.03500
15	2	321.000	37733.26074	3000.01355
16	2	421.000	37734.28564	2999.97482

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Table 5

****** OPTION 2 *** READ SOIL PROFILE

NEW SOIL PROFILE NO. 0 IDENTIFICATION CASE 0

NUMBER OF LAYERS	9	DEPTH TO BEDROCK	421.00
NUMBER OF FIRST SUBMERGED LAYER	3	DEPTH TO WATER LEVEL	13.33

LAYER	TYPE	FACTOR MOD.	FACTOR DAMP.	THICKNESS FT.	DEPTH FT	EFF. PRESS.	MODULUS KIPS/SQ FT	DAMPING	UNIT WEIGTH KIPS/FT³	SHEAR VEL FT/SEC
1	1	0.79	1.00	6.67	3.33	0.40	1826.	0.050	0.1200	700.
2	1	0.79	1.00	6.67	10.00	1.20	1826.	0.050	0.1200	700.
3	1	0.79	1.00	6.67	16.67	1.79	1826.	0.050	0.1200	700.
4	1	1.31	1.00	5.50	22.75	2.14	3019.	0.050	0.1200	900.
5	1	1.31	1.00	5.50	28.25	2.46	3019.	0.050	0.1200	900.
6	1	1.17	1.00	10.00	36.00	2.91	2693.	0.050	0.1200	850.
7	1	1.17	1.00	10.00	46.00	3.48	2693.	0.050	0.1200	850.
8	1	1.17	1.00	10.00	56.00	4.06	2693.	0.070	0.1200	850.
9	1	1.17	1.00	10.00	66.00	4.63	2693.	0.070	0.1200	850.
10	1	1.17	1.00	10.00	76.00	5.21	2693.	0.070	0.1200	850.
11	1	9.64	1.00	45.00	103.50	7.13	22179.	0.070	0.1450	2300.
12	1	9.64	1.00	45.00	148.50	10.40	22179.	0.070	0.1450	2300.
13	1	16.41	1.00	50.00	196.00	13.85	37733.	0.070	0.1450	3000.
14	1	16.41	1.00	50.00	246.00	17.48	37733.	0.070	0.1450	3000.
15	2	5.13	1.00	100.00	321.00	22.92	37733.	0.070	0.1450	3000.
16	2	4.47	1.00	100.00	421.00	30.18	37733.	0.070	0.1450	3000.
17	BASE	5.00					112578.	0.	0.1450	5000.

****** OPTION 4 *** OBTAIN STRAIN COMPATIBLE SOIL PROPERTIES

MAXIMUM NUMBER OF ITERATIONS	=	10
MAXIMUM ERROR IN PERCENT	=	5.00
FACTOR FOR EFFECTIVE STRAIN IN TIME DOMAIN	=	0.65

LAYER	TYPE	DEPTH	MAX SHEAR MOD	MAX SHEAR VEL
1	1	3.333	1819.30000	698.69795
2	1	10.000	1819.30000	698.69795
3	1	16.667	1819.30000	698.69795
4	1	22.750	3019.89999	900.18877
5	1	28.250	3019.89999	900.18877
6	1	36.000	2893.29999	850.11890
7	1	46.000	2893.29999	850.11890
8	1	56.000	2893.29999	850.11890
9	1	66.000	2893.29999	850.11890
10	1	76.000	2893.29999	850.11890
11	1	103.500	22178.89990	2300.01703
12	1	148.500	22178.89990	2300.01703
13	1	196.000	37733.79980	3000.03500
14	1	246.000	37733.79980	3000.03500
15	2	321.000	37733.26074	3000.01355
16	2	421.000	37732.28564	2999.97482

Table 6

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****** OPTION 2 *** READ SOIL PROFILE

NEW SOIL PROFILE NO. 0 IDENTIFICATION CASE H

NUMBER OF LAYERS 7 DEPTH TO BEDROCK 440.00
NUMBER OF FIRST SUBMERGED LAYER 1 DEPTH TO WATER LEVEL 0.

LAYER	TYPE	FACTOR MOD.	FACTOR DAMP.	THICKNESS FT	DEPTH FT	EFF. PRESS.	MODULUS KIB/SQ FT	DAMPING	UNIT WEIGTH KIB/FT³	SHEAR VEL FT/SEC
1	1	1.17	1.00	10.00	5.00	0.29	2693.	0.050	0.1200	850.
2	1	1.17	1.00	10.00	15.00	0.86	2693.	0.050	0.1200	850.
3	1	1.17	1.00	10.00	25.00	1.44	2693.	0.070	0.1200	850.
4	1	1.17	1.00	10.00	35.00	2.02	2693.	0.070	0.1200	850.
5	1	1.17	1.00	10.00	45.00	2.59	2693.	0.070	0.1200	850.
6	1	9.64	1.00	45.00	72.50	4.51	22179.	0.070	0.1450	2300.
7	1	9.64	1.00	45.00	117.50	7.78	22179.	0.070	0.1450	2300.
8	1	16.41	1.00	50.00	165.00	11.23	37733.	0.070	0.1450	3000.
9	1	16.41	1.00	50.00	215.00	14.46	37733.	0.070	0.1450	3000.
10	2	5.46	1.00	100.00	290.00	20.30	37733.	0.070	0.1450	3000.
11	2	4.68	1.00	100.00	390.00	27.56	37733.	0.070	0.1450	3000.
12	BASE	5.00					112578.	0.	0.1450	5000.

****** OPTION 4 *** OBTAIN STRAIN COMPATIBLE SOIL PROPERTIES

MAXIMUM NUMBER OF ITERATIONS = 10
MAXIMUM ERROR IN PERCENT = 5.00
FACTOR FOR EFFECTIVE STRAIN IN TIME DOMAIN = 0.65

LAYER	TYPE	DEPTH	MAX SHEAR MOD	MAX SHEAR VEL
1	1	5.000	2693.29999	850.11890
2	1	15.000	2693.29999	850.11890
3	1	25.000	2693.29999	850.11890
4	1	35.000	2693.29999	850.11890
5	1	45.000	2693.29999	850.11890
6	1	72.500	22178.89990	2300.01703
7	1	117.500	22178.89990	2300.01703
8	1	165.000	37733.79980	3000.03500
9	1	215.000	37733.79980	3000.03500
10	2	290.000	37733.88867	3000.03854
11	2	390.000	37733.36328	3000.09714

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to
Incl 1

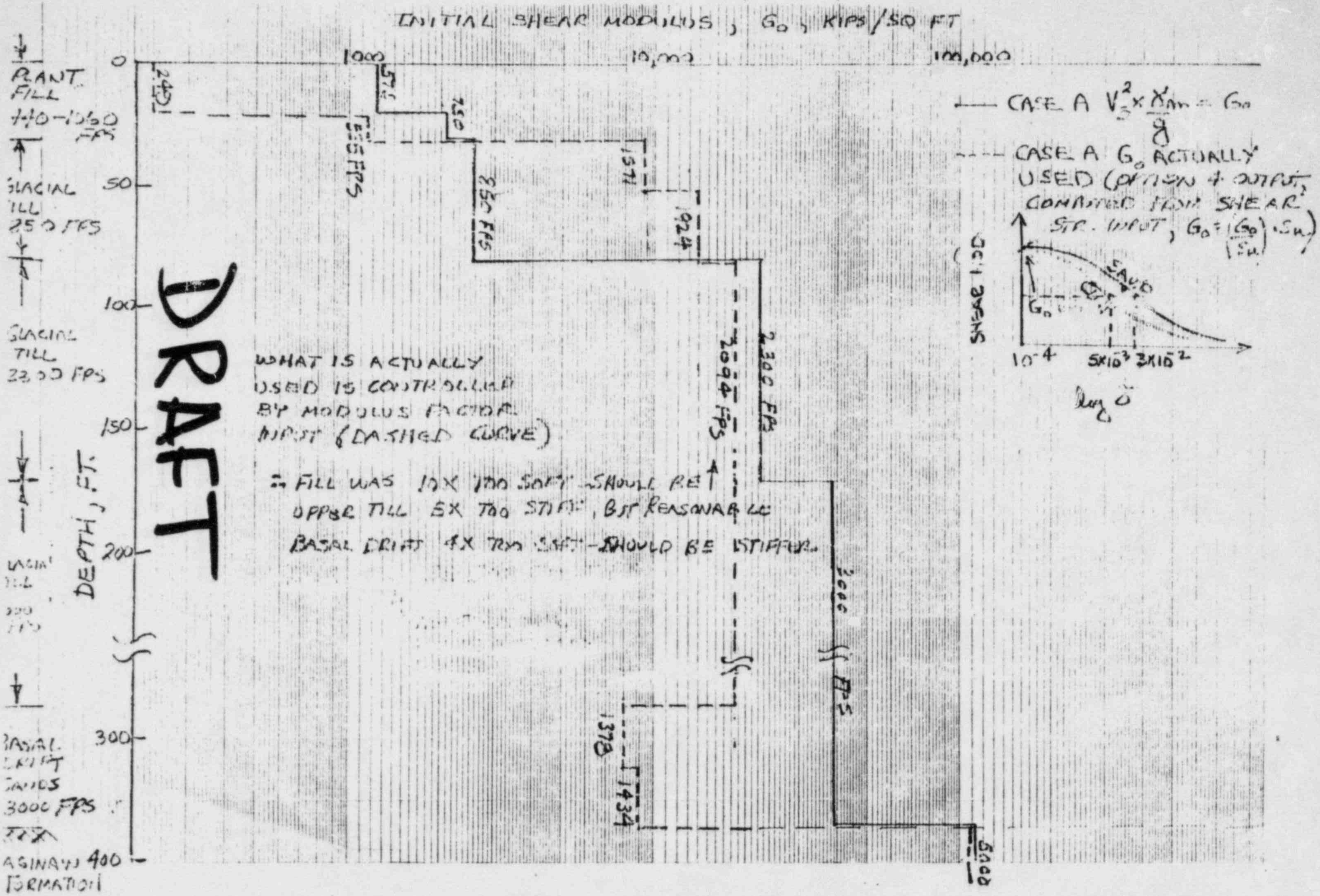

Figure 1. Initial Shear Modulus Vs Depth for Case A

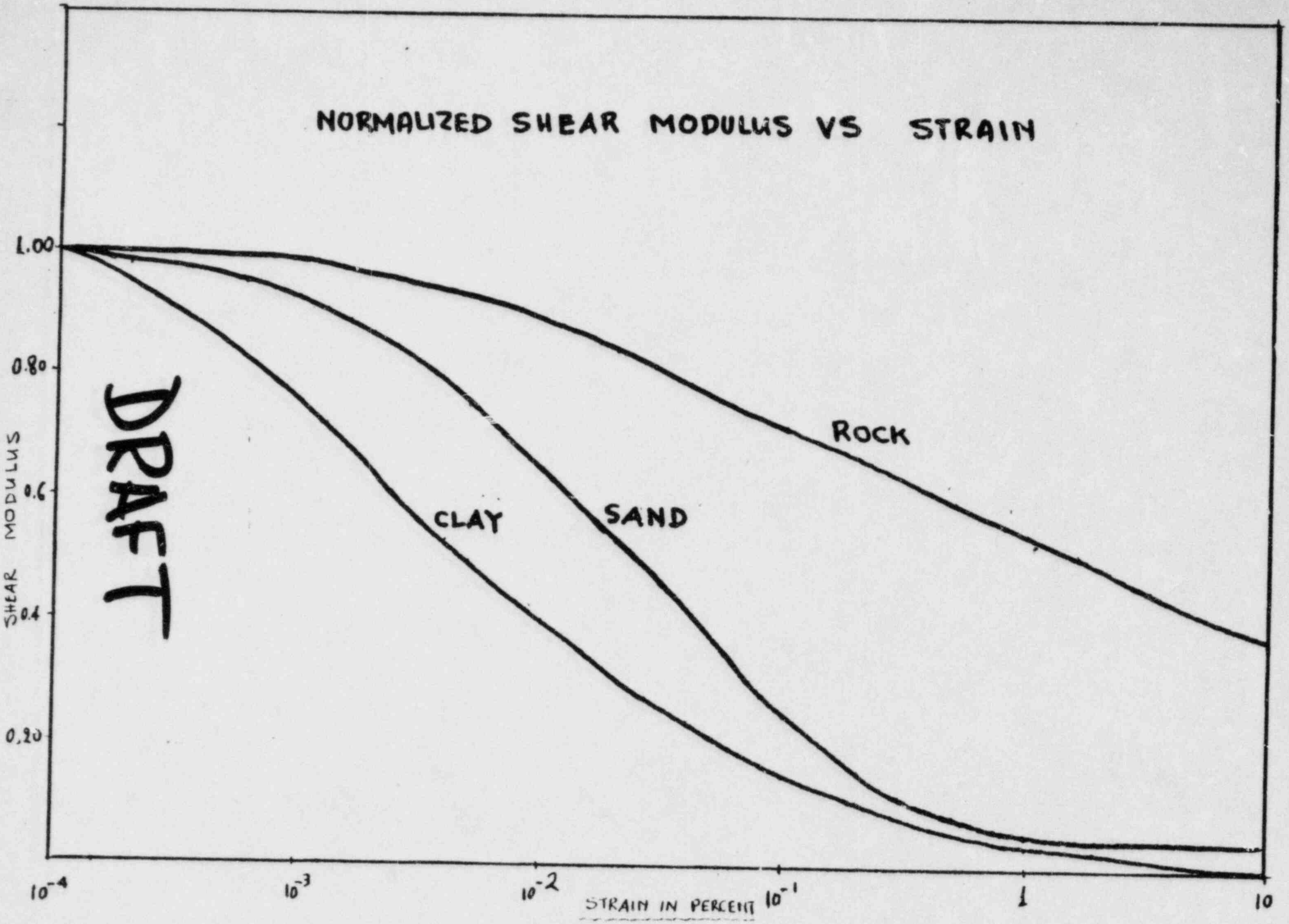

Figure 2. Normalized Shear Modulus Vs Shear Strain

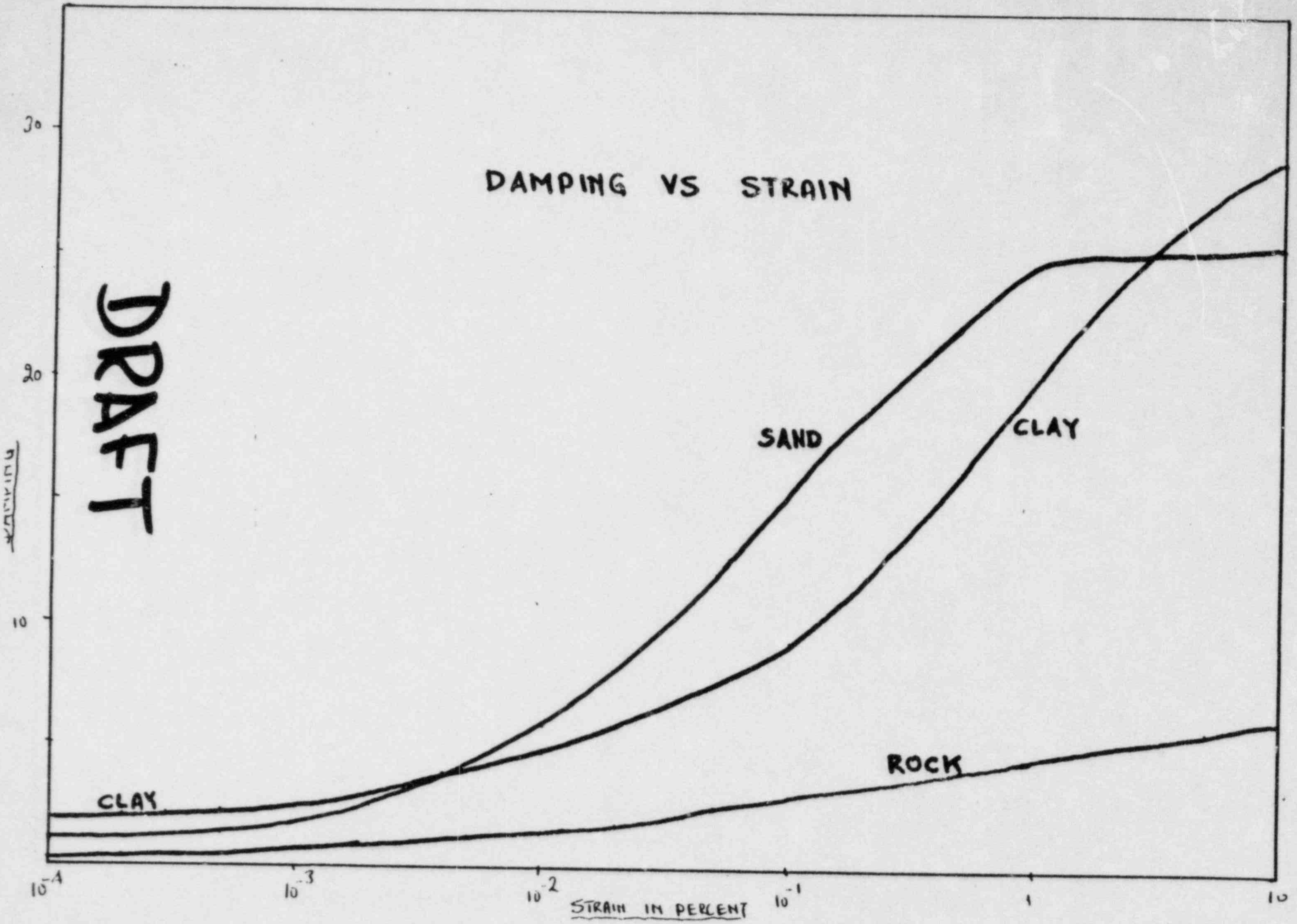

Figure 3. Normalized Damping Vs Shear Strain

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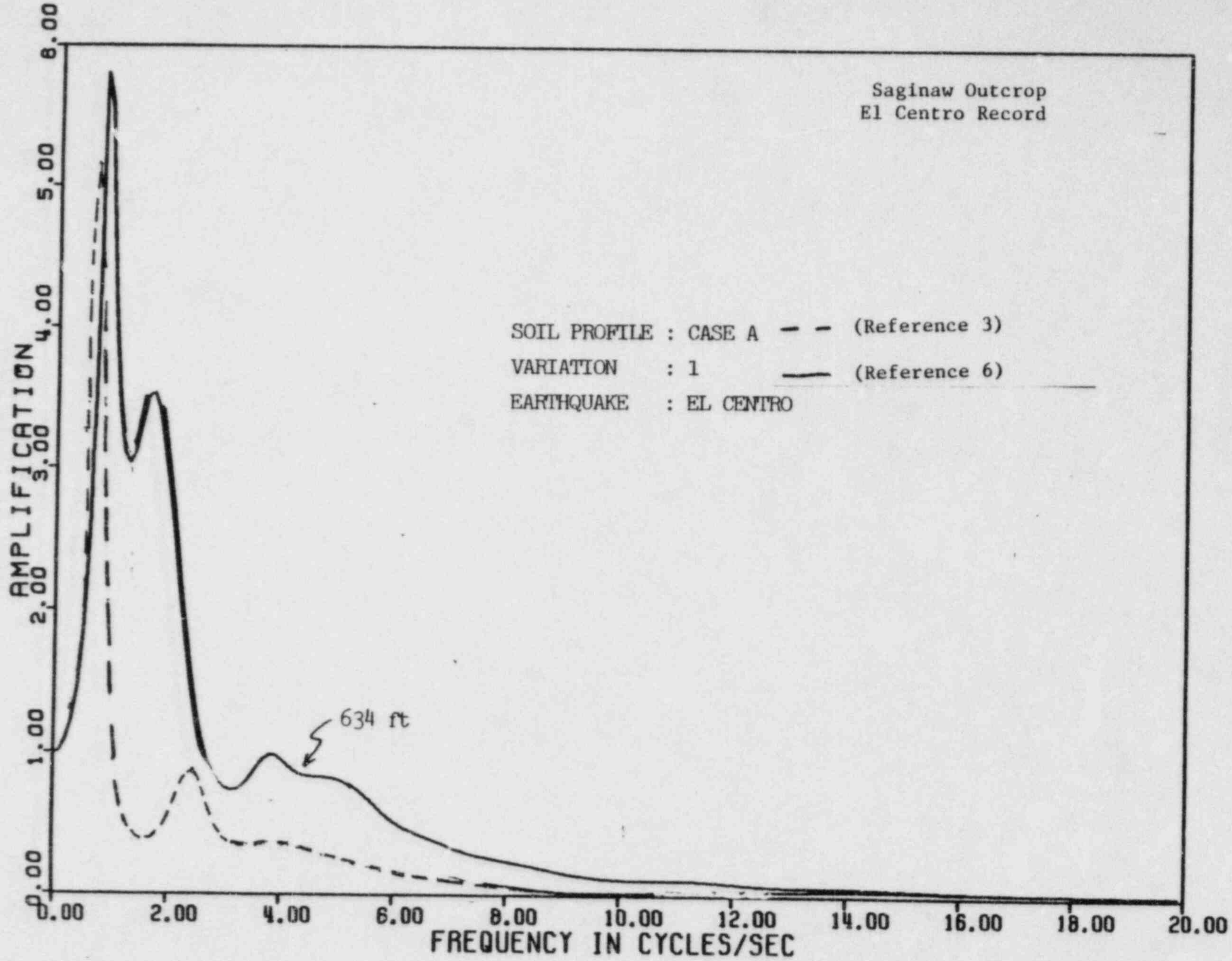

Figure 4. Effect of Changing Modulus Factors, Case A

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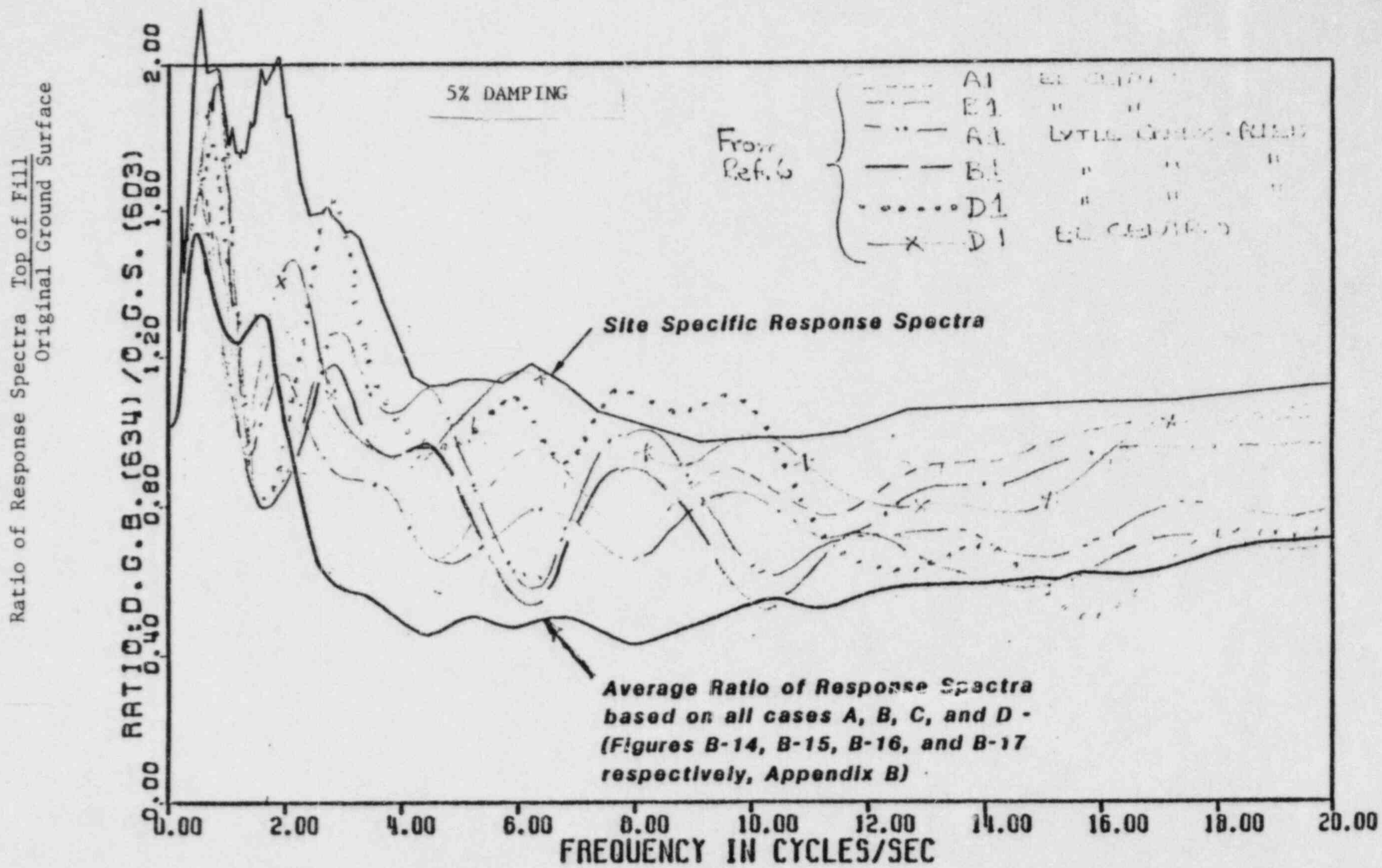

Figure 5. Ratio of response spectra for the soil at and near the Diesel Generator Building area (Figure B-25, Appendix B) and site specific response spectra.

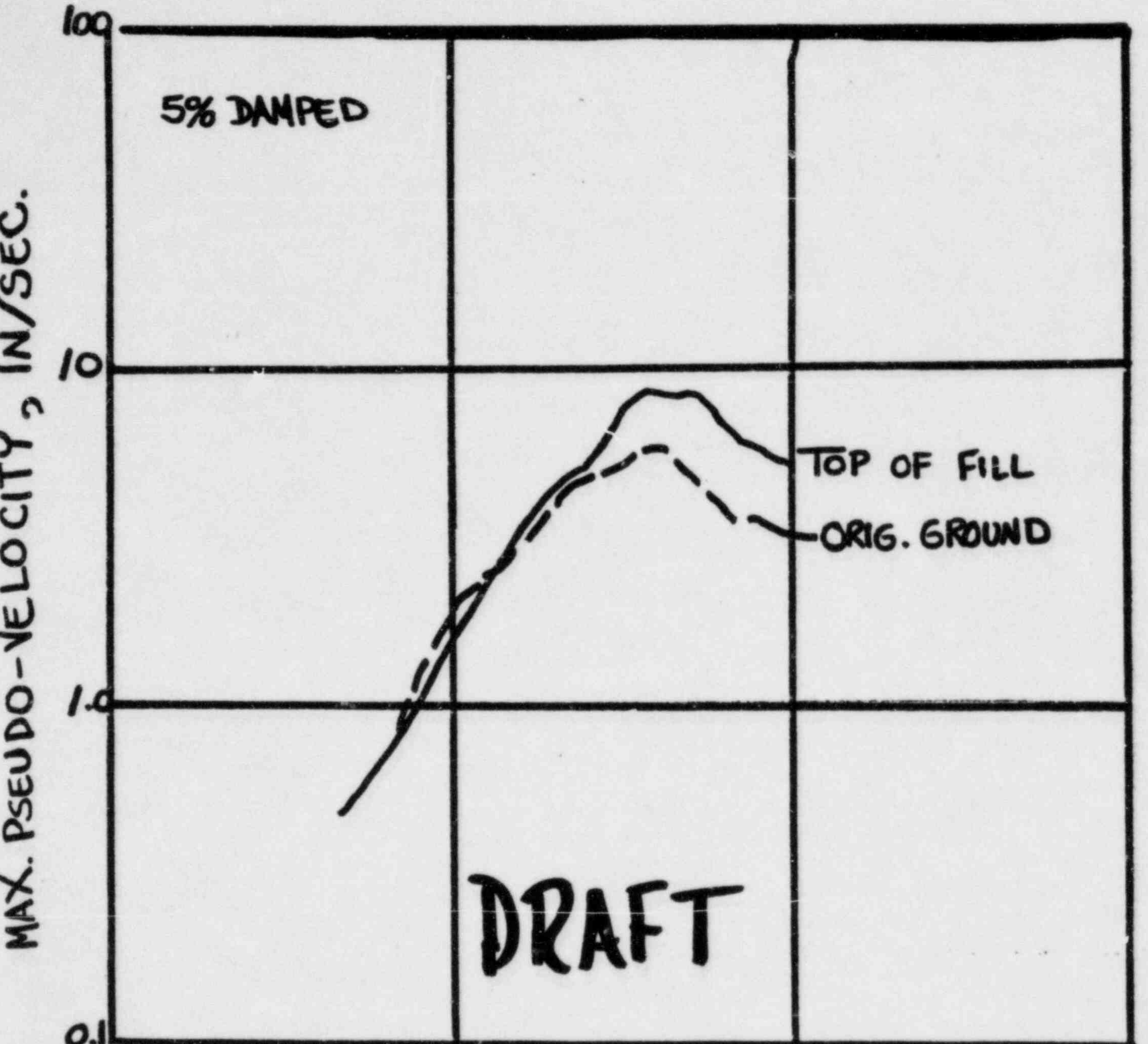

FIG. 6 84TH PERCENTILE SPECTRA FROM REFERENCES 3 AND 5

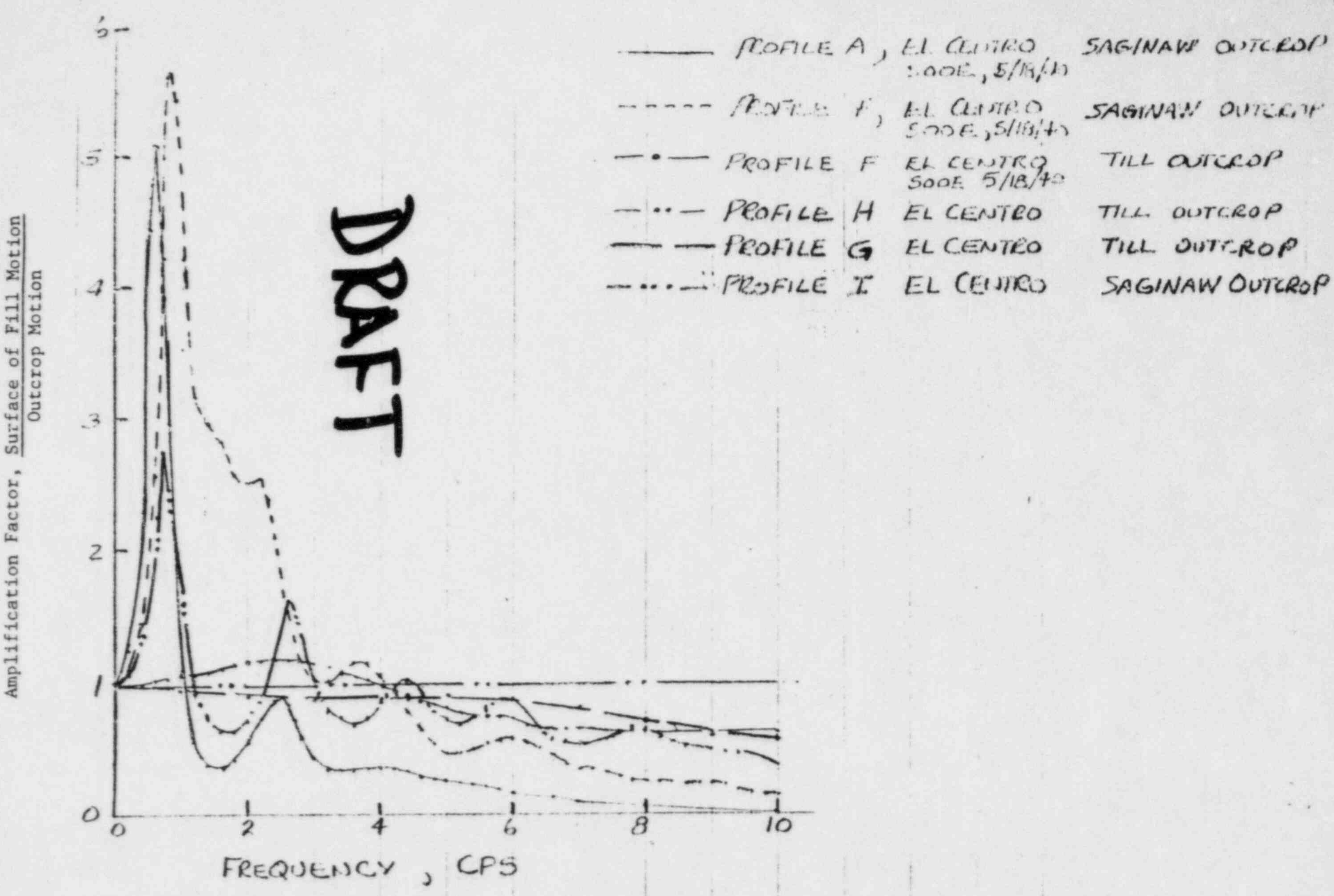

Figure 7. Comparison of Amplification Curves Obtained Using the El Centro Record

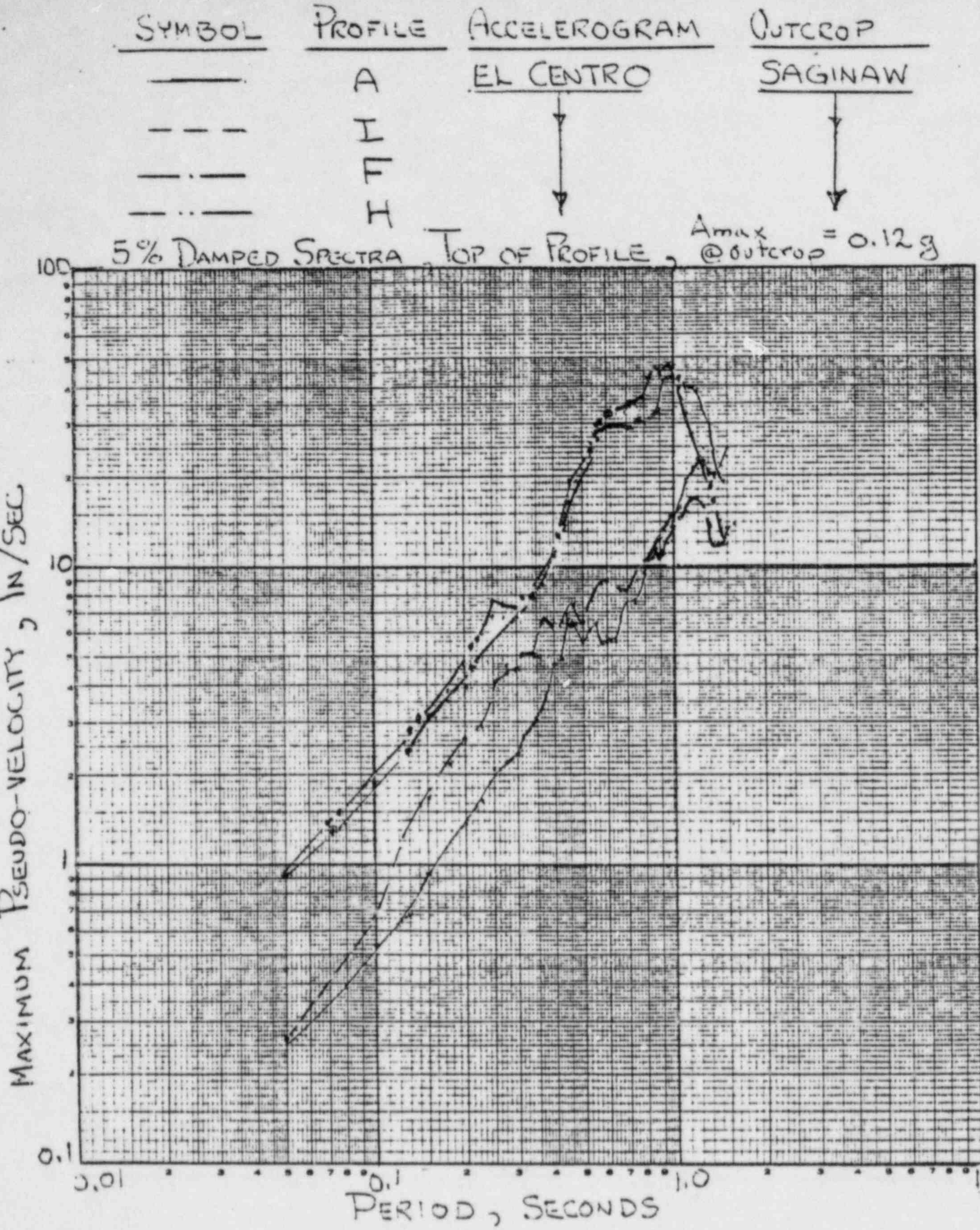

Figure 8. Shock Spectra for Top of the Site Profile, Fill Vs No Fill and for Variations in Site Profile

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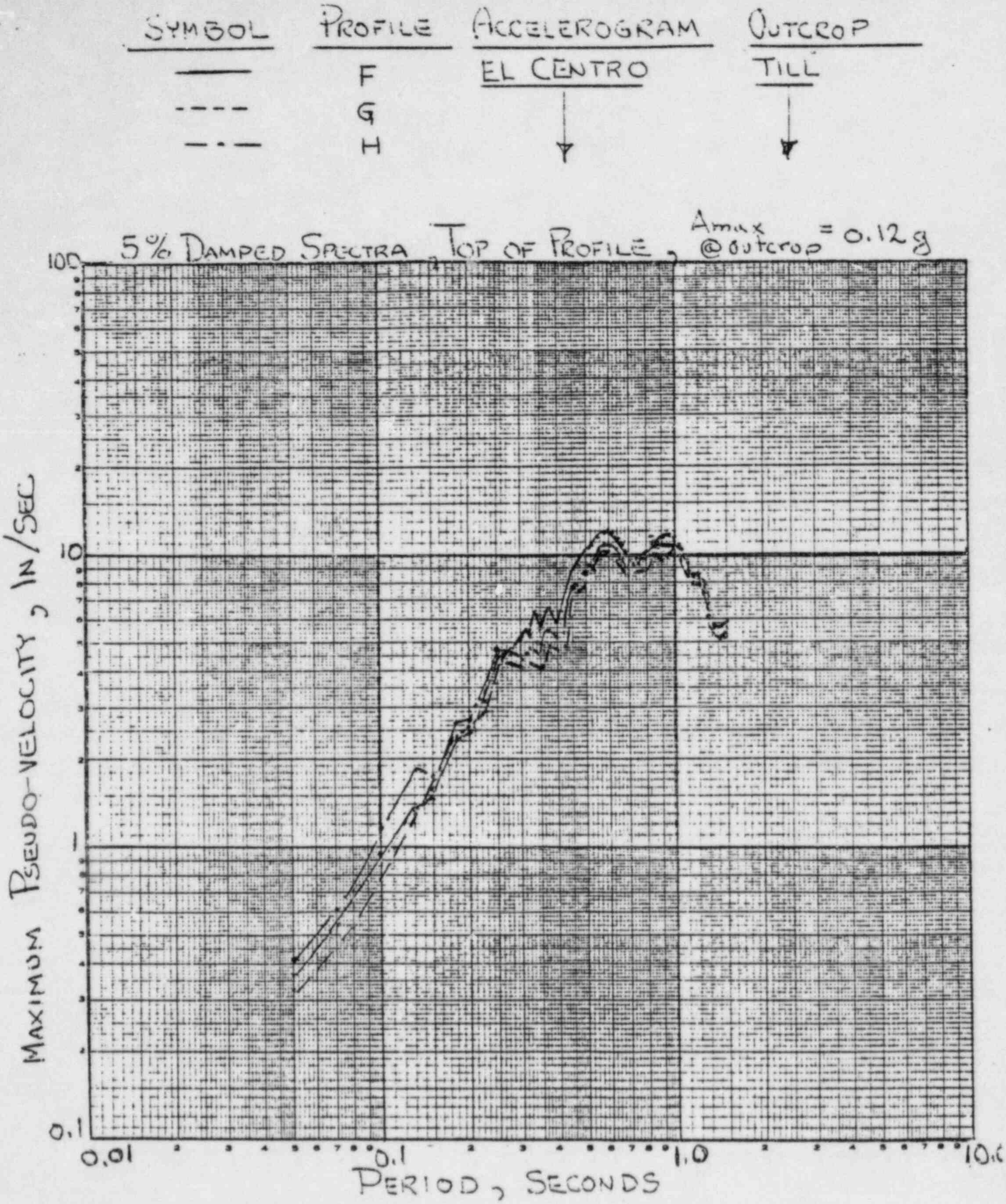

Figure 9. Shock Spectra for Profiles F, G, and H Using the El Centro Record as a Till Outcrop

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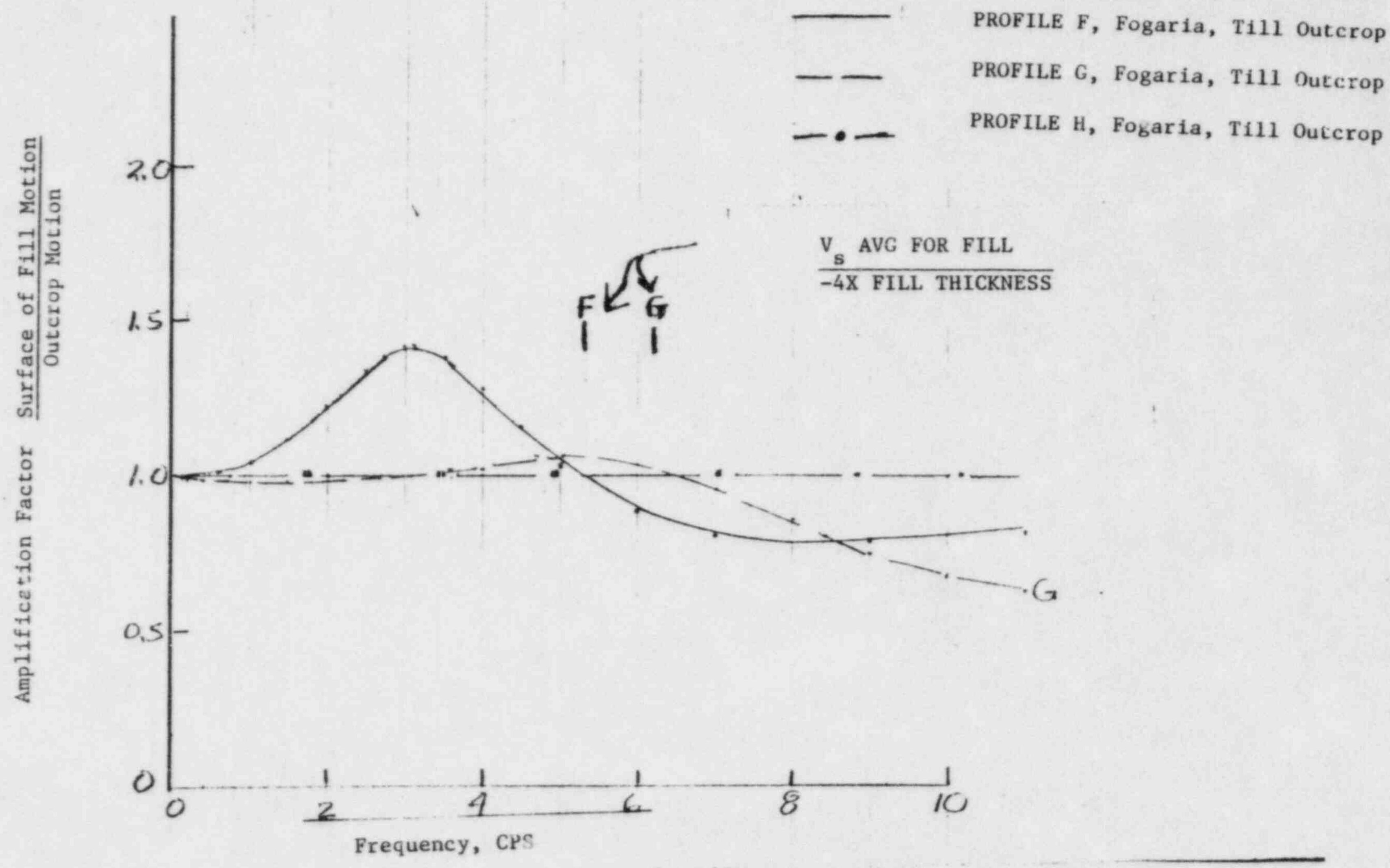

Figure 10. Amplification Factors Calculated Using Fogaria Record

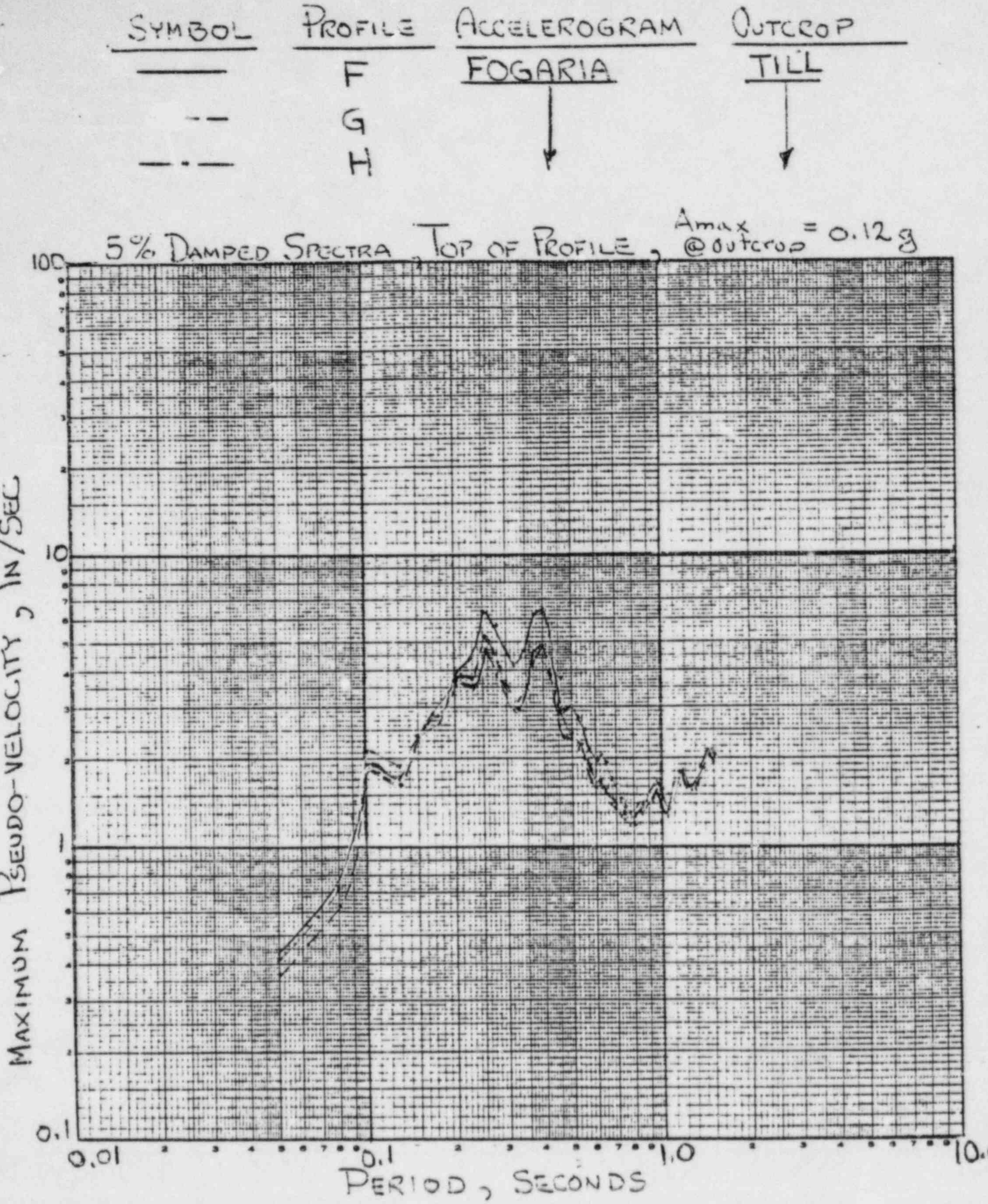

Figure 11. Shock Spectra Calculated for the Surface of Three Different Soil Profiles Using the Fogaria Record

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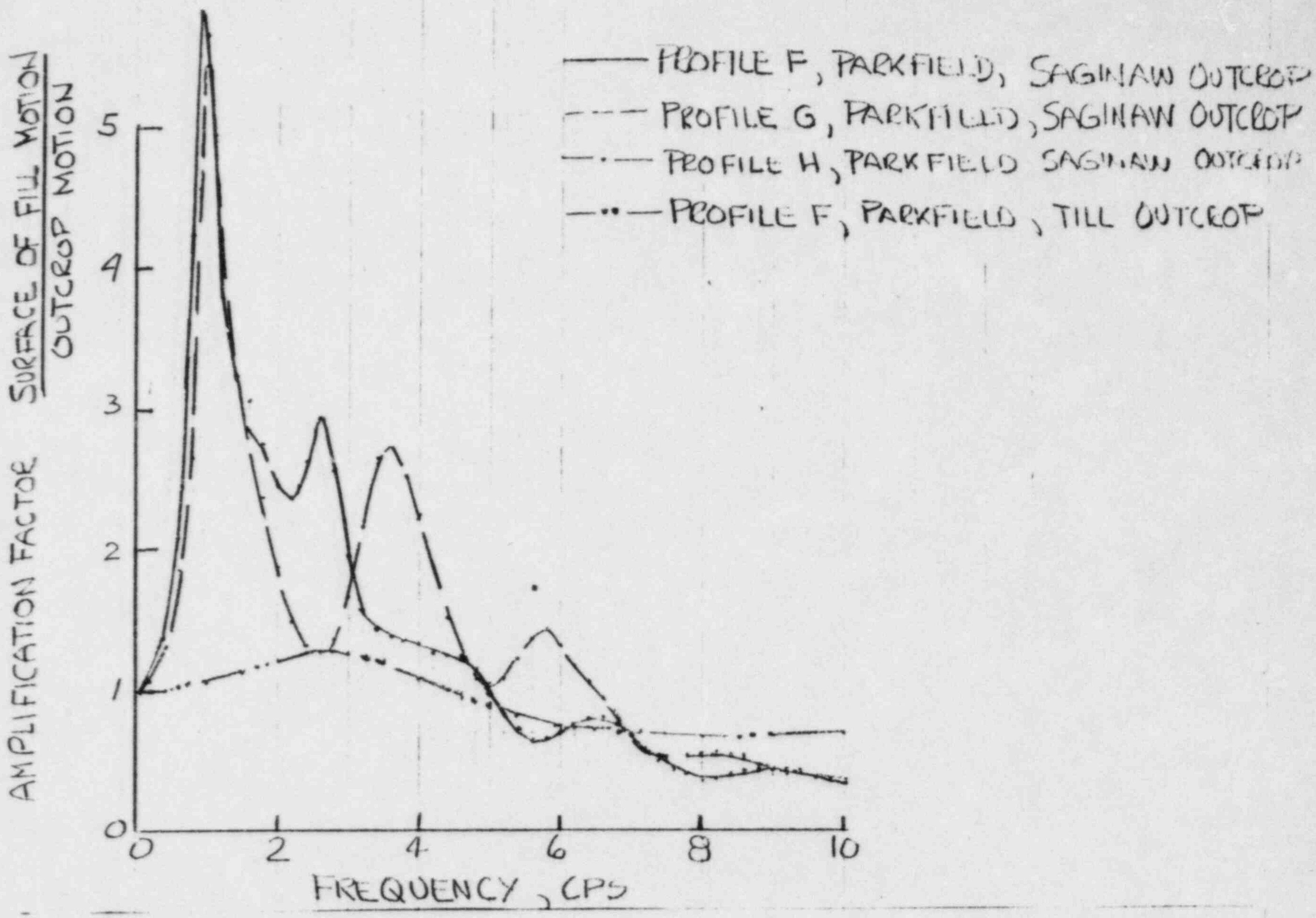

Figure 12. Amplification Factors Calculated Using Parkfield Record

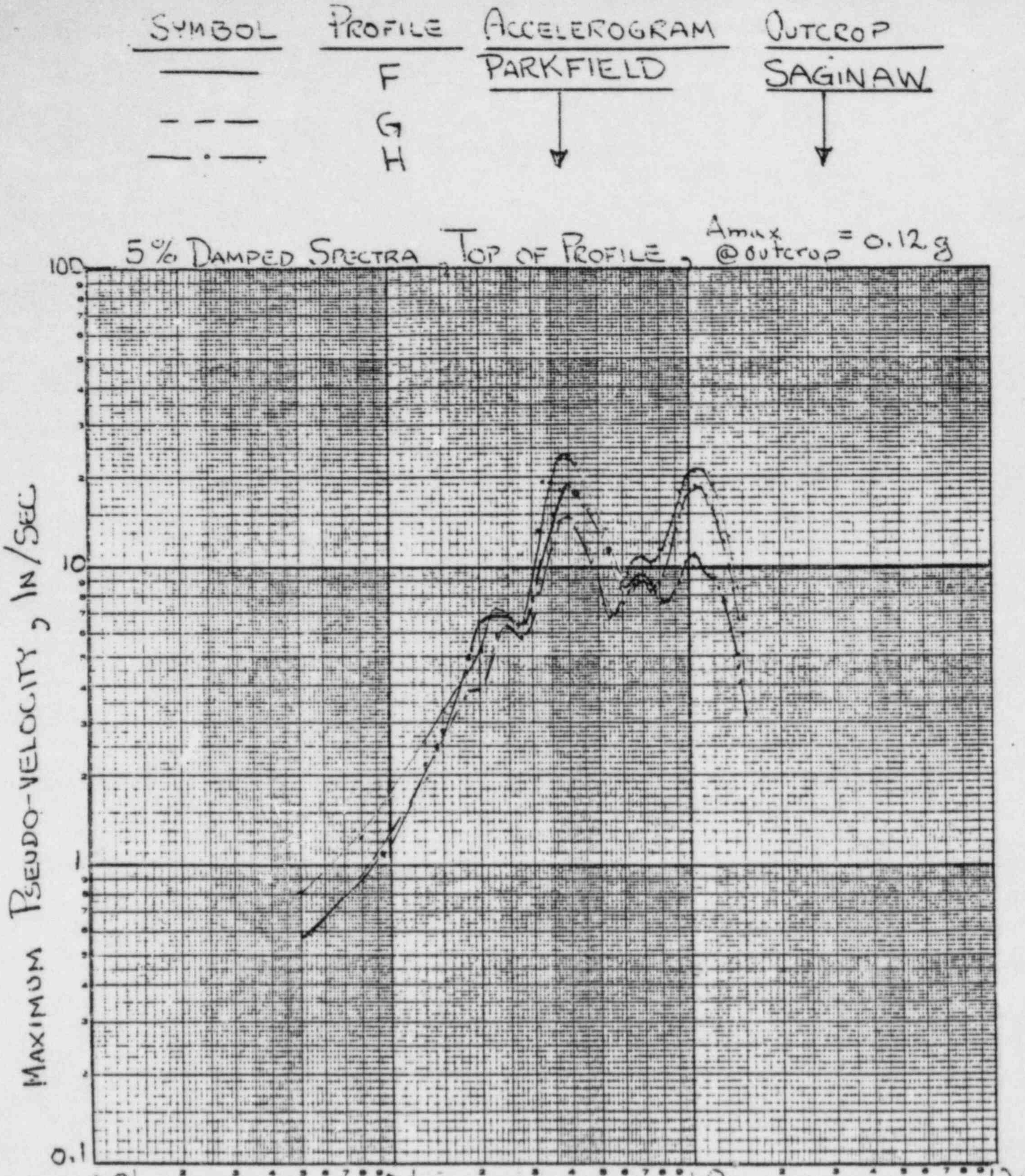

Figure 13. Shock Spectra for the Top of Profiles F, G, and H Calculated Using the Parkfield-Temblor Record as Input at an Outcrop of the Saginaw Formation

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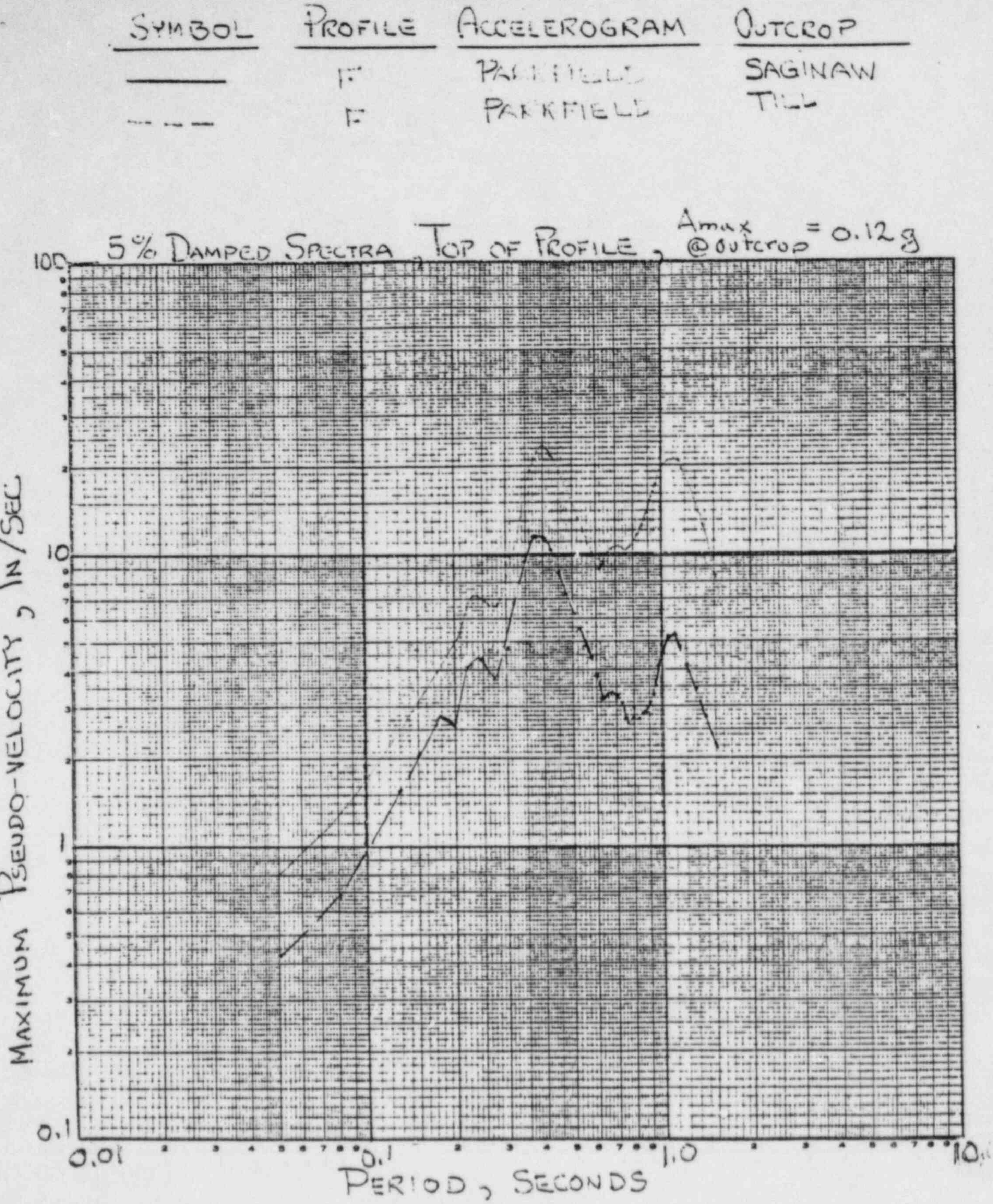


Figure 14. Effect of Selection of Outcropping Layer on Spectra for the Parkfield-Temblor Record

DRAFT

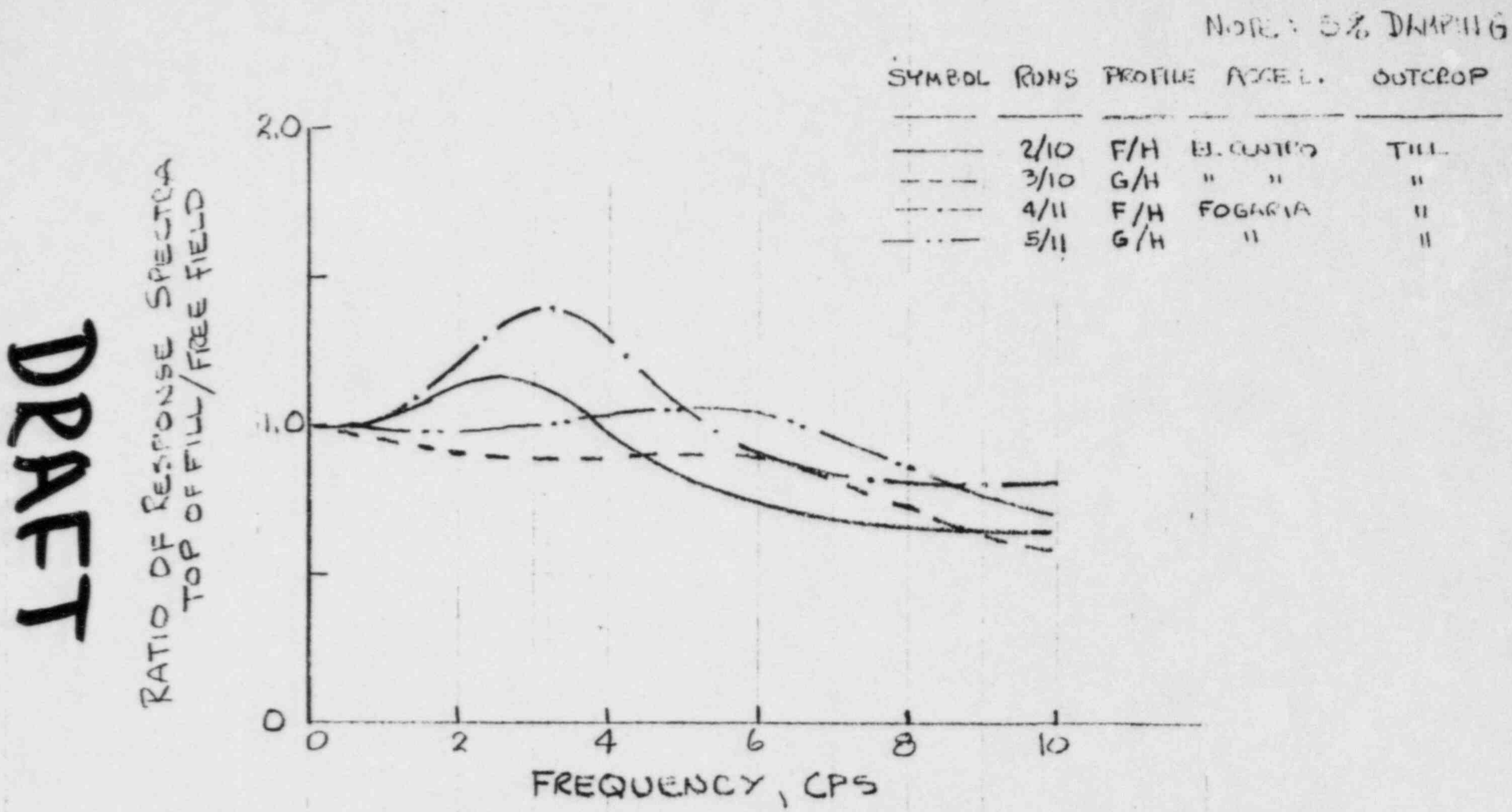


Figure 15. Ratios of Response Spectra Calculated for Till Outcrop Cases

DRAFT

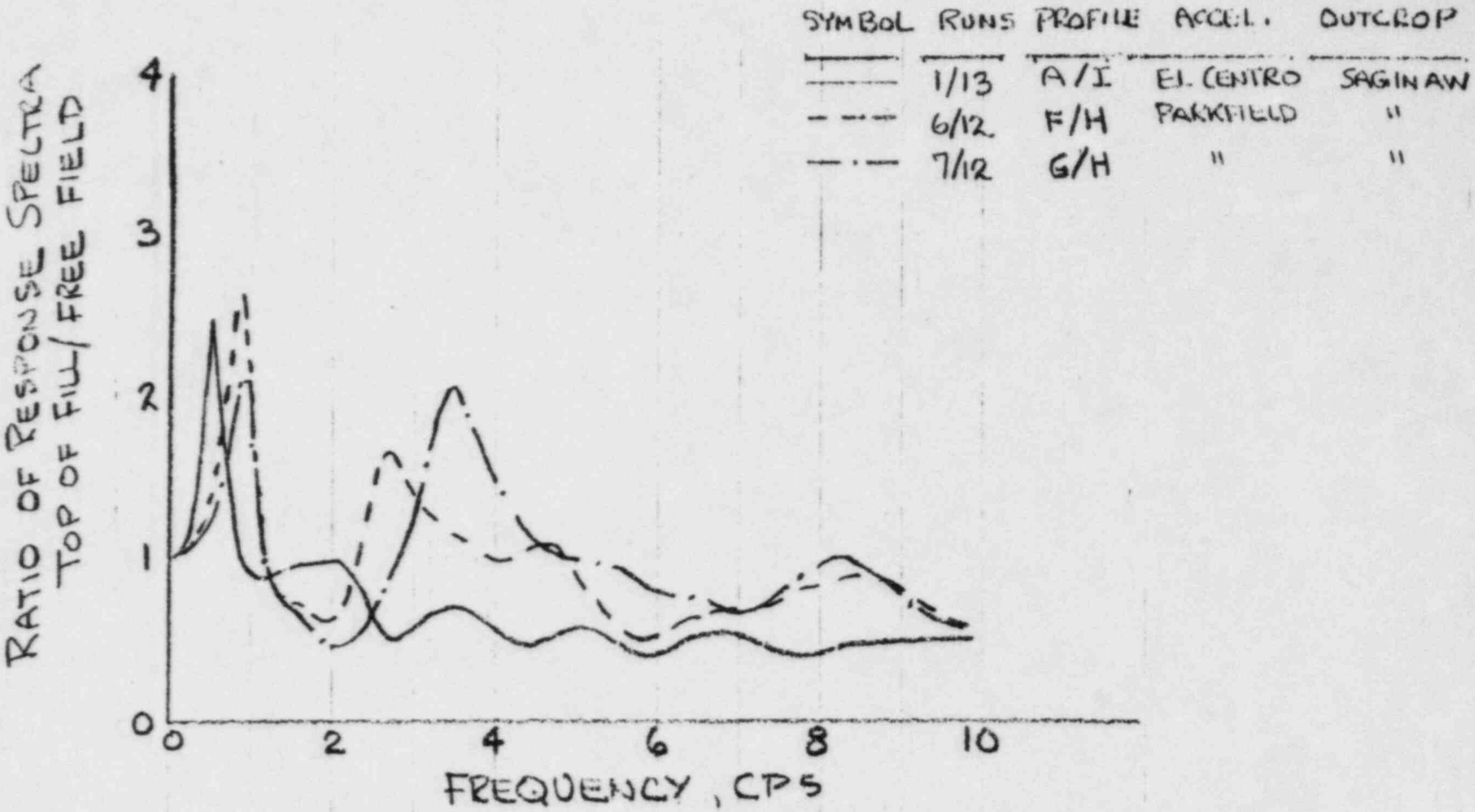


Figure 16. Ratios of Response Spectra Calculated for Saginaw Outcrop Cases

References

1. Hadala, P. F., "Visit to Midland, Michigan Nuclear Power Plant on 27-28 February 1980, A Review of the Midland Plant Units 1 and 2 FSAR (Including Revisions 1-27) 30 May 1980," US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

2. Hadala, P. F., "Letter dated 16 June 1981 to District Engineer, USAE District, Detroit (Attn: Mr. Neil Gehrig) Subject: Review of Amendment 85 - Midland Nuclear Power Plant," USAE Waterways Experiment Station, Vicksburg, Mississippi.

3. , "Site Specific Response Spectra, Midland Plant - Units 1 and 2, Part II, Response Spectra Applicable for the Top of Fill Material at the Plant Site," April 1981, Weston Geophysical Cooperation, Westboro, Massachusetts.

4. Schnabel, P. B., Lysmer, J., and Seed, H. B., "SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," EERC Report 72-12, Dec 1972, University of California at Berkeley, Berkeley, California.

5. , "Site Specific Response Spectra, Midland Plant - Units 1 and 2, Part I, Response Spectra - Safe Shutdown Earthquake, Original Ground Surface," Feb 1981 (Also Addendum to Part I dated Jun 1981), Weston Geophysical Corporation, Westboro, Massachusetts.

6. Vanmarke, E., "Letter to P. F. Hadala, dated 9 Jul 1981, Subject: Midland Plant Site Specific Response Spectra - Amplification Analysis."

7. , Engineer Manual EM 1110-1-1802, "Geophysical Exploration," Feb 1979, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, Washington, DC.

8. Seed, H. B., and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. 70-10, Dec 1970, Earthquake Engineering Research Center, University of California, Berkeley, California.

9. , "Site Specific Response Spectra, Midland Plant - Units 1 and 2, Addendum to Part I, Response Spectra - Original Ground Surface," June 1981, Weston Geophysical Corporation, Westboro, Massachusetts.

DRAFT

Incl 3
to
Incl 1

Telecopied 3/25/82

3/88

WESGA

11 March 1982

Commander
USAE District, Detroit
ATTN: NCEE D-T/Mr. Neil Gehring
477 Michigan Avenue
Detroit, MI 48226

Inclosed are two Memorandums for Record which describe the results of my evaluation of the applicant's methods for selecting the spring constants and damping parameters to be used in the dynamic analysis of the borated water storage tank, the auxiliary building, and the service water pump structure at the Midland Nuclear Power Plant (Incl 1 and 2).

2 Incl
as

PAUL F. HADALA
Assistant Chief
Geotechnical Laboratory

CF:
Joe Kane, NRC
Harry Singh, NCD

Rec'd 3/25/82

FACSIMILE HEADER SHEET
ER 105-1-5

FROM NAME	OFFICE SYMBOL	TELEPHONE NO.	RELEASER SIGNATURE		
HAdAlA	WESGA	542-2110	Gaye McBee		
TO NAME	OFFICE SYMBOL	TELEPHONE NO.	PAGES	PRECEDENCE	DTG
Joseph KAne	US NRC	492-8153	11		

WESGA

9 March 1982

MEMORANDUM FOR RECORD

SUBJECT: Review of The Selection of Foundation Spring Constants and Damping Parameters For Use in Dynamic Analysis of the Service Water Pump Structure and the Auxiliary Building

Introduction

1. Messrs. Joe Kane of NRC and Mr. Harry Singh of North Central Division, CE, requested I review the following documents in November and December 1981:

a. "Auxiliary Building Seismic Model; Revision 3, for Midland Plant Units 1 and 2," Consumers Power Company, September 28, 1981.

b. "Service Water Pump Structure Seismic Model, Revision 1, for Midland Plant Units 1 and 2," Consumers Power Company, September 28, 1981.

c. "Testimony of Robert P. Kennedy before the Atomic Safety and Licensing Board in the Matter of Consumers Power Company (Midland Plant Units 1 and 2)," Docket Numbers 50-329 OM, 50-330 OM, 50-329 OL, 50-330 OL.

2. The documents were furnished to me by Mr. Singh. All three of them concern dynamic soil-structure interaction analyses to determine (a) seismic forces on specific structures and (b) seismic input to internal components in these structures. I reviewed them during the early part of December. On 11 December I participated in a telephone conference with Messrs. Mike Blume, Phil Steptoe, Robert Kennedy, Joe Kane, and Frank Rinaldi, which explored questions raised in my review that were not fully addressed in Documents a, b, and c. The purpo[se] of that conversation was to begin the process of getting additional facts on th[e] record. On 15 and 16 December I attended the Atomic Safety and Licensing Board (ASLB) hearing at Midland, Michigan. The same questions I raised on 11 December were asked of Mr. Kennedy when he was on the witness stand and his answers were made a matter of record.

3. The professional opinion I arrived at as a result of my review was given to the ASLB in oral testimony and is recorded on pages 6121-6286 of the transcript of the hearings. This memorandum contains essentially the same information presented in my testimony.

Soil Properties

4. The state-of-the-art basis for selection of equivalent spring constants and damping parameters is found in the solution for the vibration of a plate on homogenious isotropic linear elastic media. These solutions have been modifi[ed] to include the effect of foundation embedment. The specific theoretical solutions and equations used by the applicant in Documents a and b (See Appendix

A of Reference a) are entirely acceptable for use in analyses carried out for the purposes identified in paragraph 2. The matter requiring judgment is the selection of elastic constants (the shear modulus (G) and Poisson's Ratio (ν) to be used in the analysis.

5. The method used by the applicant to select elastic constants is discussed on pages 8 and 9 of Document a, pages 2-3, 3-2, and 3-3 of Document a, pages 2-1, 3-1, and 3-2 of Document b, and in Mr. Robert Kennedy's oral testimony. "G" was determined by laboratory tests on "undisturbed" soil samples from the site and at shear strain levels of 10^{-2} to 10^{-3} percent. A correction factor of 1.5 was applied to correct for sample disturbance. Additionally, empirical formulas for shear modulus as a function of effective stress level and shear modulus data obtained by Danes and Moore at the La Salle site were also used in arriving at the following parameters:

Glacial till: $\nu = 0.42$ $G = 7746$ kips/sq ft

Backfill : $\nu = 0.40$ $G = 1728$ to 2495 kips/sq ft (depending on elevation)

6. While I do not prefer the applicant's method because of uncertainty in the correction factor for sample disturbance, I agree with the applicant's results. I obtained comparable answers as follows: Field shear wave velocities have been measured in the plant fill and the underlying glacial till. In the case of the till, soil at depths at least equal to the building exterior dimensions are involved in its rigid body motion. Field shear wave velocities (V_s) of approximately 850 ft/sec for the first 60 ft below the foundation and 2300 ft/sec for greater depths were measured at very small strain levels.

$$G = \rho V_s^2 \quad \text{when,} \quad \rho = \text{mass density} = \frac{135 \text{ lbs}}{ft^3} \div 32.16 \frac{ft}{sec^2}$$

A depth weighted average shear modulus for the top 100 ft of till is:

$$G_{avg} = \left(\frac{40}{100}\right) \frac{(2300)^2}{1000} \times \frac{135}{32.16} + \left(\frac{60}{100}\right) \frac{(850)^2}{1000} \times \frac{135}{32.16} = 1$$

kips

This value of G is appropriate for the strain levels in the field shear wave velocity test. These strain levels are much smaller than those in the earthquake. Thus, a reduction is in order. The basis for the reduction are the Seed-Idriss 1970 curves which indicate the following:

Type of Soil	Reduction in G @ 10^{-3}% Strain %	Reduction in G @ 10^{-2}% Strain %
Clay	26	60
Sand	10	30

Based on these data a 25% reduction is appropriate for the till to correct for strain level differences:

$$G = (1-.25) \times 10{,}700 = 8026 \text{ kips/sq ft}$$

This is only three percent different from the value of G used by the applicant, an insignificant difference. Similar computations were performed using the shear wave velocities measured in the plant fill. In the plant fill V_s varies from ~ 500 ft/sec to ~ 1200 ft/sec and there is a trend of increasing V_s with depth in the fill. The computed* range of shear moduli is from 700 to 2800 kips/s and if depth weighted would be in close agreement with values actually used by the applicant.

Spring Constants

7. In the selection of spring constants using the shear moduli and Poisson's ratio values discussed above along with the various equations for the translational and rocking stiffness of a rigid plate on or in an elastic half space, the applicant chose to vary the spring constants ± 50 percent and envelope the results in estimating upper bound forces in the buildings. In the reviewer's opinion, this conservative practice more than overcomes any uncertainty about either the appropriateness of a particular value of G or about the use of equations for a rigid plate when the actual footings and/or mat are not quite rigid.

Damping Parameters

8. The soil around the structure provides a means by which energy may be absorbed via hysteretic damping and a way in which energy can radiate away from the structure toward infinity. In the mathematical solution for the vibration of a plate on an elastic half space, the term which accounts for this radiation is identical in form with the damping term in the equation of motion for a single degree of freedom system. Thus, the term radiation damping was introduced. The radiation damping term is a function of the structure's dimensions and the square root of the product of mass density and shear modulus.

9. In their analyses, the applicant limited the radiation damping to 75 percent of the value calculated for a footing or mat on an elastic half space. This was done because radiation in a layered system is not as effective as in a half space in terms of the amount of damping that appears to take place. By taking only 3/4 of the theoretical damping value, the shortcoming in the theoretical model has been adequately compensated. The applicant added to the radiation three percent of critical damping as hysteretic soil damping. This is definitely conservative as several state-of-the-art papers

* ρ = 120/32.16; strain factor 25% (same as for the till)

have recommended as much as five percent. Finally, the total damping (soil, structural, and radiation) was limited to ten percent in all except the rigid body modes.

<u>General</u>

10. In documents a and b, the actual spring constants and damping parameters used in the analyses are presented. Sample calculations for one case are discussed in detail. This sample calculation illustrates a number of various assumptions which have to be made to select spring constants. The effect of embedment is usually less than a 20 percent increase in the spring constant (k) and damping parameter (c). This is illustrated in tabulations given in Documents (a) and (b). In view of the applicant's decision to use a ± 50% variation in k and c, details having a small effect on c or k depending on assumptions relating to how embedment is treated are moot.

11. A review of these documents indicates that the applicant has used elementary but entirely adequate spring and dashpot models to account for soil structure interaction in the dynamic analyses of the Auxiliary Building and The Service Water Pump Structure. The applicant's choice of soil properties to use in calculating interaction parameters is reasonable and his decision to vary parameters ± 50% and envelope results is considered prudent and sufficient to envelope the effect of any uncertainties in the model or the site properties.

Paul F. Hadala

PAUL F. HADALA
Assistant Chief
Geotechnical Laboratory

WESGA 11 March 1982

SUBJECT: Review of the Selection of Spring Constants and Damping Parameters for the Borated Water Storage Tank

1. I reviewed Addendum No. 1, "Design Report for the Borated Water Storage Tank Foundation Analysis," contained in a letter to J. G. Keppler for J. W. Cook dated 24 November 81. I also revised portions of the prefiled testimony of Mr. Robert P. Kennedy concerning the subject tank.

2. I found the basic approach, equations, assumptions, and ranges of parameters ($\pm$ 50%) selected for use in the analyses very similar to those described in my Memorandum for Record of 9 March 1982, subject, "Review of The Selection of Foundation Spring Constants and Damping Parameters for Use in Dynamic Analysis of the Service Water Pump Structure and the Auxiliary Building." I am completely satisfied with the applicant's methodology in the subject area.

3. The applicant selected $G = 1510$ kips/sq ft, $\nu = 0.45$, and $\rho = .00357$ kip-sec^2/ft^4 as the soil properties to use in the analysis. He also decided to use a $\pm$ 50% variation in G. Then he performed analyses in which spring constants (k) and radiation damping parameters (c) were calculated for an annulur foundation mat using $G = 755$ and 2265 kip/sq ft.

4. The borated water storage tank is founded on about 30 ft of fill. The tank's radius is 26 ft. Thus, the engineering properties properties of the fill will control the selection of k's and c's for use soil structure interaction analyses. Incloaure 1 shows shear-wave velocity V_s data measured in the plant fill. The G values (min, average, and max) selected by the applicant correspond to strain levels higher than those in the field V_s test. Hence, in order to compare directly with field data they should be increased 25 to 35%. If this is done the V_s range computed is 500 to 900 ft/sec. As shown by Incl 1, this is definitely within the realm of the data. The value of ρ is consistent with measured densities and the value of Poisson's Ratio (ν) is reasonable. Any uncertainty in its most appropriate value is more than taken care of by the $\pm$ 50% variation in G since ν appears in the form:

$$\left(\frac{G}{1-\nu}\right) \text{ or } \frac{(1-\nu)G}{7-8\nu}$$ in the spring constant equations.

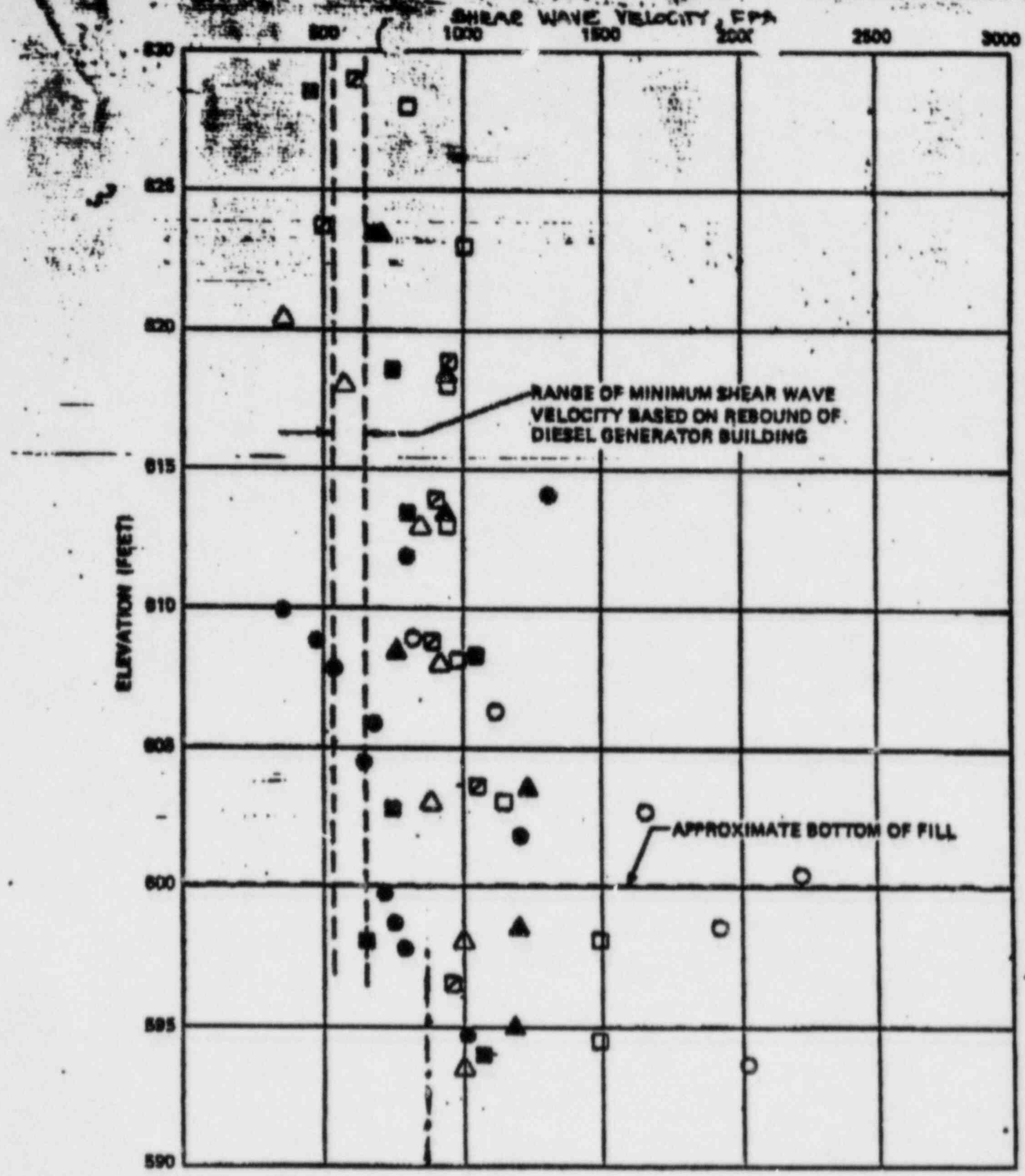
5. In conclusion I am fully satisfied with the selection of the spring constants and damping parameter range for the analysis of the borated water-storage tanks.

Paul F. Hadala

1 Incl
as

PAUL F. HADALA
Assistant Chief
Geotechnical Laboratory

Incl 2



LEGEND:

- ▨ CONDENSATE TANKS AREA
- □ ■ BORATED WATER STORAGE TANKS AREA
- ○ ● SERVICE WATER PUMP STRUCTURE
- △ ▲ DIESEL GENERATOR BUILDING

Weston Survey (FSAR 2.5.4.1.2)

BECHTEL
ANN ARBOR

MIDLAND POWER PL

SHEAR WAVE VELOCITY PR
PLANT AREA FILL

50-329
50-330
Rec'd 3/30/82
3/B8

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO:
WESGA

19 March 1982

Mr. Joseph Kane
US Nuclear Regulatory Commission
Phillips Building
7920 Norfolk Avenue
Bethesda, MD 20014

RECEIVED
MAR 2 3 1982

Dear Joe:

Transmitted herewith is my Memorandum for Record dated 15 March 1982 (Incl 1).

Sincerely,

Paul F. Hadala

PAUL F. HADALA
Assistant Chief
Geotechnical Laboratory

1 Incl
As stated

CF:
Mr. Harry Singh, NCD
Mr. Neil Gehring, DET DIST

XE02
S 1/1

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO:
WESGA

15 March 1982

MEMORANDUM FOR RECORD

SUBJECT: Liquefaction Potential in Areas of the Midland Plant Which the Applicant Does not Commit to Permanently Dewater

1. On 12 March 1982 I received three site drawings from Mr. James K. Meisenheimer which indicated pipe and duct bank locations and elevations, boring locations and the areas around the deisel generator building and the railroad bay of the auxiliary building which the applicant commits to dewater. The drawings also show in pen and ink notations near some of the borings the elevation, standard penetration resistance, and the standard penetration resistance required to prevent liquefaction in the safe shut down earthquake. In a telephone conversation on 15 March 1982, Mr. Meisenheimer said the locations identified on the drawing were the only places (outside the areas to be dewatered) where sands with unacceptably low blow counts have been encountered at the site.

2. I found that numbers reported as standard penetration resistance required to prevent liquefaction in a safe shutdown earthquake with five exceptions closely agreed with my own computations by the Seed-Idriss Simplified Method for the following combination of parameters:

wet unit weight	120 lbs/cu ft
groundwater elevation	627 ft
A_{max}	19 g
Magnitude	5.3
Factor of Safety	1.5

My computations are given in Incl 1 and the safe N value versus depth curves for factors of safety of 1.0 and 1.5 are compared to the individual values reported on the drawings in Incl 2.

3. There are a large number of unsatisfactorily low blow counts in cohesionless soils above elevation 610 on the north west side of the service water pump and circulating water intake structures. Since this area will recharge rapidly if the dewatering system is inoperable, and since this area contains four category 1 pipelines supported on or above this liquifiable material it is recommended that material be removed and replaced with dense cohesionless or cohesive soil in the following locations:

Incl 1

WESGA 15 March 1982
SUBJECT: Liquefaction Potential in Areas of the Midland Plant Which the Applicant Does not Commit to Permanently Dewater

a. Above elevation 610 underneath 26" OHBC-55 and 26" OHBC-54 east of the north-south line defined by site coordinate line E575.

b. Above elevation 610 underneath 26" OHBC-19 and 26" OHBC-16 south of the east-west line defined by the site coordinate line S4950.

Paul F. Hadala

HADALA

2 Incl
as

SUBJECT: Seed - IDRISS @ 627	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

EL.	H ft	$\bar{\sigma}_v$ PSF	r_d	τ_{avg} PSF	$\tau/\bar{\sigma}_v$		Mod. Pen. Restore N$_1$ blows/ft		C_N	Penetration Pen Presstore N blows/ft	
					F.S. 1.0	F.S. 1.5	F.S. 1.0	F.S. 1.5		F.S. 1.0	F.S. 1.5
634	0	—	—	—	—		—		—	—	
630	4	—	—	—	—		—		—	—	
627	7	840	.99	104	.124	(.186)	8	(12)	1.47	5	(8)
625	9	955	.98	133	.139	(.209)	9	(13)	1.40	6	(10)
620	14	1242	.97	207	.167	(.251)	11	(16)	1.24	9	(13)
615	19	1530	.95	282	.184	(.276)	12	(18)	1.12	10 ~~11~~	(16)
610	24	1818	.94	356	.196	(.294)	13	(19)	1.04	11 ~~12~~	(18)
605	29	2105	.92	430	.204	(.306)	13	(20)	0.97	12 ~~14~~	(19) ~~(20)~~
600	34	2393	.90	504	.210	(.315)	14	(20)	0.92	13 ~~15~~	(21) ~~(22)~~
595	39	2680	.86	579	.216	(.324)	14	(21)	0.87	14 ~~16~~	(21) ~~(24)~~

$F.S = 1.5$ $\quad$ $M = 5.3$

$\gamma_m = 120\,PCF$

$\gamma' = 120 - 62.5 = 57.5\,PCF$

$\tau_{avg} = .65\,\gamma_m\,H\,\frac{A_{max}}{g}\,r_d \quad \left(\frac{A_{max}}{g} = 0.19\right.$

$\left(\frac{\tau}{\bar{\sigma}_v}\right)_\ell = \frac{.31}{20} N_1 \quad$ for $M = 5.3$

$C_N = \frac{N_1}{N}$

$\frac{356}{1818} = 0.196$

$0.196 \times 1.5 = .294$

went to Seed's curves w/ 0.196 & 0.294 & pick of N = 13 & (19)

Incl 1

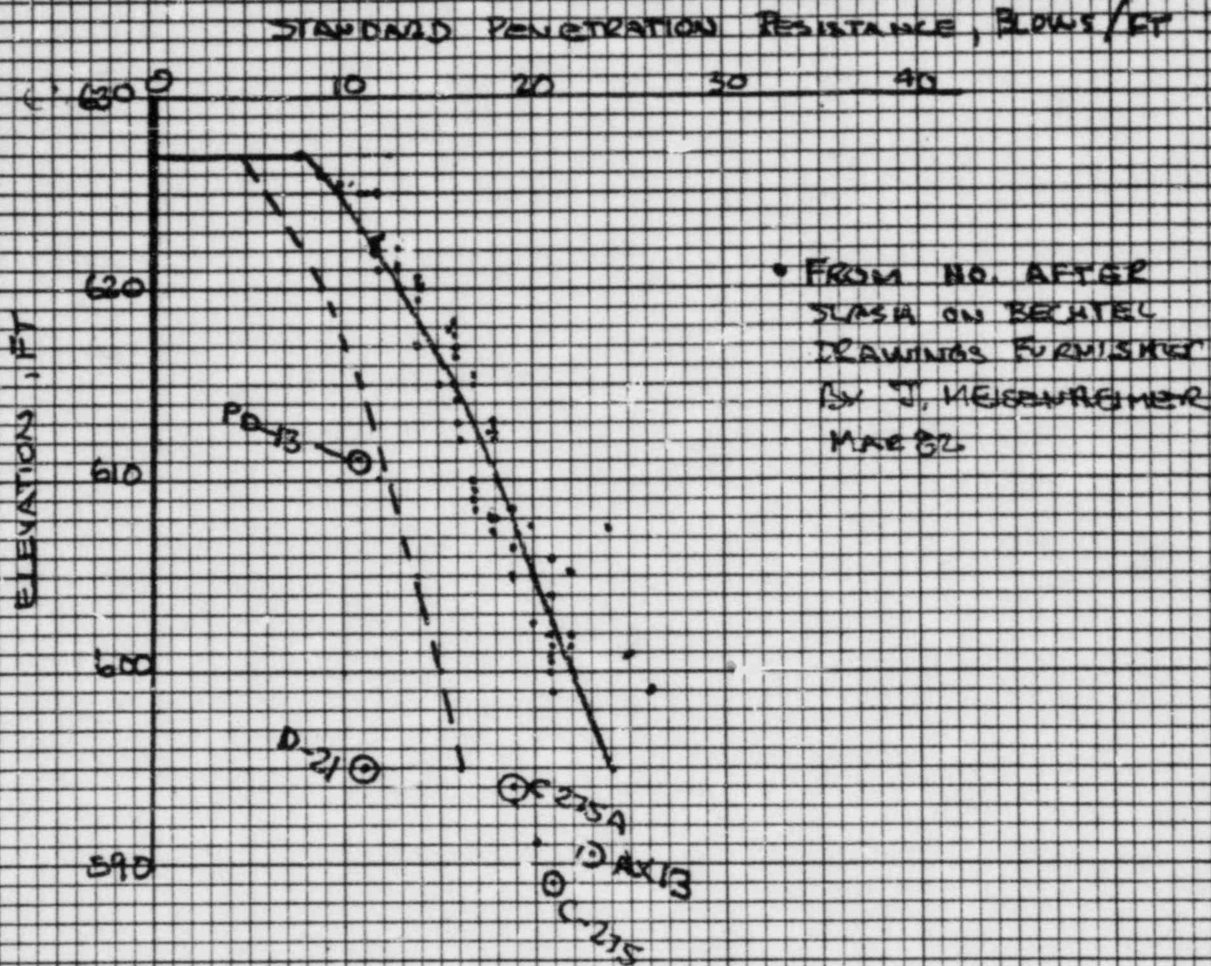
WES FORM NO. 1253
REV OCT 1958

SEED - IDRISS SIMPLIFIED ANALYSIS OF THE PLANT FILL

a_{max} = 0.19 g M = 5.3   GWT @ EL 627. γ_m = 120 PCF

FS = 1.5 ——

FS = 1.0 - - -



Incl 2