Commonwealth Edison
One First National Plaza, Chicago, Illinois
Address Reply to: Post Office Box 767
Chicago, Illinois 60690

April 18, 1984

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Subject: Byron Generating Station Units 1 and 2
River Screenhouse Seismic Design
NRC Docket Nos. 50-454 and 50-455

References (a): June 15, 1983 letter from T. R. Tramm to H. R. Denton.

(b): February 7, 1983 letter from B. J. Youngblood to L. O. DelGeorge.

(c): February 28, 1983 letter from T. R. Tramm to H. R. Denton.

Dear Mr. Denton:

This letter provides additional information regarding the seismic design basis for the river screennouse at Byron Generating Station. NRC review of this information should help close Confirmatory Issue 1 of the Byron SER.

Enclosed are the responses to two FSAR questions regarding the seismic design of the Byron river screenhouse which were transmitted in reference (a). The response to FSAR question 241.8 regarding the dynamic soil properties includes the results of recent geophysical measurements made at the screenhouse. Figure Q241.8-9 shows the average G_{max} values for each soil layer used in our analysis. A report of the crosshole testing which supports these shear modulus values was provided in reference (c). This response to FSAR question 241.8 was apparently omitted from that letter.

The response to FSAR question 362.1 regarding the screenhouse seismic design basis is enclosed because it was also omitted in reference (c). Both of these responses will be incorporated into the Byron/Braidwood FSAR at the earliest opportunity.

B001
1/1

Please direct further questions regarding this matter to this office.

One signed original and fifteen copies of this letter and the enclosures are provided for your NRC review.

Very truly yours,

T. R. Tramm

T. R. Tramm
Nuclear Licensing Administrator

lm

Attachment

8485N

QUESTION 362.1

"The safe shutdown earthquake for the Byron Station site is based on the postulated occurrence of a maximum Modified Mercalli intensity VIII (body wave magnitude 5.8) earthquake near the site. (See Byron Station Safety Evaluation Report (SER), NUREG-0876). The staff's position as stated in the Byron Station SER is that a Regulatory Guide 1.60 response spectrum with a high frequency anchor of 0.20g at the foundation level of structures founded on rock is an adequately conservative representation of the vibratory ground motion from this size earthquake.

"The Byron Station river screen house is founded on soil. Under certain conditions, the presence of soils can amplify vibratory ground motion. The amplitude and frequency of the amplified motion is a function of the physical properties of the material and its thickness.

"Demonstrate the adequacy of the design basis for the river screen house by directly calculating a site-specific response spectrum and/or by calculating the amplification of an appropriate rock spectrum resulting from the presence of the soil.

"It has been the staff's practice in the past (Sequoyah SER, 1979, Watts Bar SER, 1982; Midland SER, 1982; Fermi SER, 1981) to accept the 84th percentile response spectral level (mean plus one standard deviation) calculated from a suite of accelerograms recorded at distances of about 25 kilometers or less at locations with foundation conditions similar to the site from earthquakes with magnitudes in the range of plus or minus 0.5 units of the target magnitude."

RESPONSE

The staff has accepted the use of a 0.2g Regulatory Guide 1.60 spectrum as input for the analysis of the main plant at Byron, which is founded on rock. As stated in the SER, the 0.2g Regulatory Guide 1.60 spectrum envelopes the 5.8 m_b site-specific spectrum for rock sites developed by TVA for the Sequoyah, Watts Bar, and Bellefonte nuclear power plants, and that developed by LLL/TERA for use in the NRC-sponsored seismic hazard analysis program. In the following paragraphs a similar justification is developed for the Byron river screen house input design spectrum.

The SSE at the Byron site is based on the postulated occurrence of a maximum MM Intensity VIII earthquake near the site. Nuttli and Hermann (1978) have developed a relation between

maximum MM intensity and magnitude for the central United States. Using this relation results in an estimated magnitude of 5.75 for an MM Intensity VIII. Nuttli and Brill (NUREG/CR-1577) estimate the magnitude of the May 26, 1909 Northern Illinois earthquake as 5.1. Estimates of the magnitude of the 1937 Anna, Ohio earthquake (MM VII to VIII) range from 5.0 to 5.3 (Nuttli and Hermann, 1978; Nuttli and Brill, NUREG-CR-1577). Therefore, using the site specific spectrum developed from magnitude 5.8 earthquakes provides a conservative estimate of the vibratory ground motion expected at the site.

There are two site specific spectra for $m_b = 5.8$ that are suitable for use in establishing the adequacy of the Byron river screen house design spectrum. One of these was generated by Illinois Power Company for the justification of the seismic design of the Clinton Power Plant (Reference 1) and the other was generated by LLL/TERA (NUREG/CR-1582) for use in the NRC-sponsored seismic hazard analysis program. Figure Q362.1-1 provides a comparison of the Clinton site specific spectra, LLL/TERA soil spectra and the Byron river screen house design basis time-history spectra. All spectra are plotted for a 5% oscillator damping. The Clinton and the LLL/TERA site specific spectra are the 84th percentile spectra for $m_b = 5.8$ earthquake and soil sites. Note that the Byron river screen house design spectrum is significantly higher than both the Clinton and the LLL/TERA spectra.

The site soil column frequency significantly affects the site specific spectrum. The following paragraph compares the Clinton and the Byron river screen house sites for this effect to show that the Clinton site specific spectra provides a good estimate for the Byron river screen house site specific spectra.

The subsurface section under the Byron river screen house is shown in FSAR Figure 2.5-60. The founding material consists of approximately 90 feet of interbedded layers of fine to coarse sand and fine to coarse gravel. This soil layer is underlain by sandstone. The properties of the soil and rock layers pertinent to the site specific spectrum are shown in FSAR Figures 2.5-89 and 2.5-62, respectively.

The geophysical average shear wave velocity at the Byron river screen house site is computed to be 935 to 1110 fps, compared to 2100 fps at Clinton. The soil depth at the Byron river screen house site is 90 feet, where as at Clinton it is approximately 180 feet. Thus, the soil column frequencies (based on $V_s/4H$ criteria) at the two sites are close to each other and based on the soil column frequency characteristics, one would expect the spectra at Byron river screen house to be similar to that at Clinton. The large margin inherent in the Byron river screen house design spectrum when compared to the Clinton site specific spectrum (Figure Q362.1-1) provides us with the

assurance that the Byron river screen house design time-history is a conservative estimate of the vibratory ground motion expected at the site.

In addition to the site column frequency, the impedance contrast between the soil strata and the underlying rock half space is also believed to affect the soil amplification and the shape of the site specific spectra. Since the rock-soil impedance contrast at the Byron river screen house is significantly higher than the rock-soil impedance contrast at the Clinton site, and the rock-soil impedance contrast information at most strong motion recording stations is often not known at 100 to 200 feet depths to permit empirical studies, a theoretical site specific spectrum at the Byron river screen house site was developed to evaluate the effect of the higher impedance contrast at the Byron river screen house as follows:

a. Conservative spectral amplification curves for the Byron river screen house site profile were obtained using the upper bound, mean, and lower bound soil properties. The most probable site spectral amplification was obtained by assigning 25%, 50%, and 25% weights to the upper, mean, and lower bound soil properties respectively.

b. The theoretical ground response spectrum at the Byron river screen house site was obtained by applying these spectral amplification factors to the average of the 84th percentile 5.8 m_b rock spectrum generated by the LLL/TERA (NUREG/CR-1582) and TVA (Reference 2).

The spectral amplifications for the Byron river screen house site including the effect of impedance contrast were obtained using the SHAKE program (Reference 3). The SHAKE analysis was performed for the upper bound, mean, and lower bound soil properties. The [illegible] thickness, shear modulus, Poisson's ratio, unit wei[illegible] damping values used for the three sets of analyses [illegible]n in Table Q362.1-1.

Fourteen diffe[illegible]otions listed in Table Q362.1-2 were used as input [illegible]E analysis. The fourteen rock motions listed in Tab[illegible]re all the rock motions available through the Ca[illegible]titute of Technology with maximum ground acceler[illegible]n b[illegible]en 0.05 and 0.5g.

The input in the SHAKE analysis was specified at the rock (half space) outcropping. The spectral amplification functions were computed as the ratio of the response spectrum of the resulting surface (top of soil [illegible]ayer 1) motion and the response spectrum of the rock outcropping input motion. The spectra and the spectral amplification rations were computed in the frequency range of 0.5 Hz and 20 Hz at a frequency interval consistent with Regulatory Guide 1.122 requirements. Thus,

$$A(f) = \frac{S(f)}{H(f)} \quad \text{(Equation 1)}$$

where

$A(f)$ = Spectral amplification for frequency f

$S(f)$ = Response sepctrum value at frequency f of the resulting surface motion

$H(f)$ = Response spectrum value at frequency f of the input motion at rock outcropping

For each soil property (upper bound, mean, and lower bound), an average spectral amplification function was determined as the average of the amplification function derived from each of the 14 earthquakes. The most probable site spectral amplification function was then obtained by assigning 25%, 50%, and 25% weights to the upper, mean and lower bound soil properties case amplification functions. This most probable site spectral amplification functions is shown in Figure Q362.1-2.

The theoretical site specific spectrum at the Byron river screen house site was obtained by multiplying the average of the 84th percentile 5.8 m_b rock spectrum generated by TVA (Reference 2) and LLL/TERA (NUREG/CR-1582) by the spectral amplification function presented in Figure Q362.1-2.

Figure Q362.1-3 presents the comparison of the Byron river screen house history spectrum, the theoretical site specific spectrum and the 84th percentile 5.8 m_b rock site specific spectrum based on LLL/TERA and TVA references. All spectra are plotted for a 5% oscillator damping. It can be observed that the design basis spectrum essentially envelops the theoretical site specific spectrum for all frequency except in the 1.25 to 2 Hz frequency range. In this frequency range, maximum exceedance is approximately 20%. As the lowest predominant structural frequency of the Byron river screen house structure is 4.8 Hz and the design basis time-history spectrum envelops the theoretical site specific spectrum for all frequencies in the greater than 2 Hz frequency range, it can be concluded that the Byron river screen house design basis time history is a conservative estimate of the ground motions expected at the Byron river screen house site during the SSE.

REFERENCES

1. "Site Specific Response Spectra Clinton Power Station - Unit 1 of Illinois Power Company," Revision 1, May 1982, prepared for Sargent & Lundy by Weston Geophysical Corporation.

2. Tennessee Valley Authority, Division of Engineering Design, "Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar and Bellefonte Nuclear Power Plants - Phase II," August 1978.

3. SHAKE, Soil Layer Properties and Response for Earthquake Motions (09.7.119-3.3). S&L modified program written by J. Lysmer and P. B. Schnabel of the University of California-Berkeley which computes response in a horizontally layered semi-finite system subjected to vertically traveling shear waves based on the continuous solution of the shear wave equation.

TABLE Q362.1-1

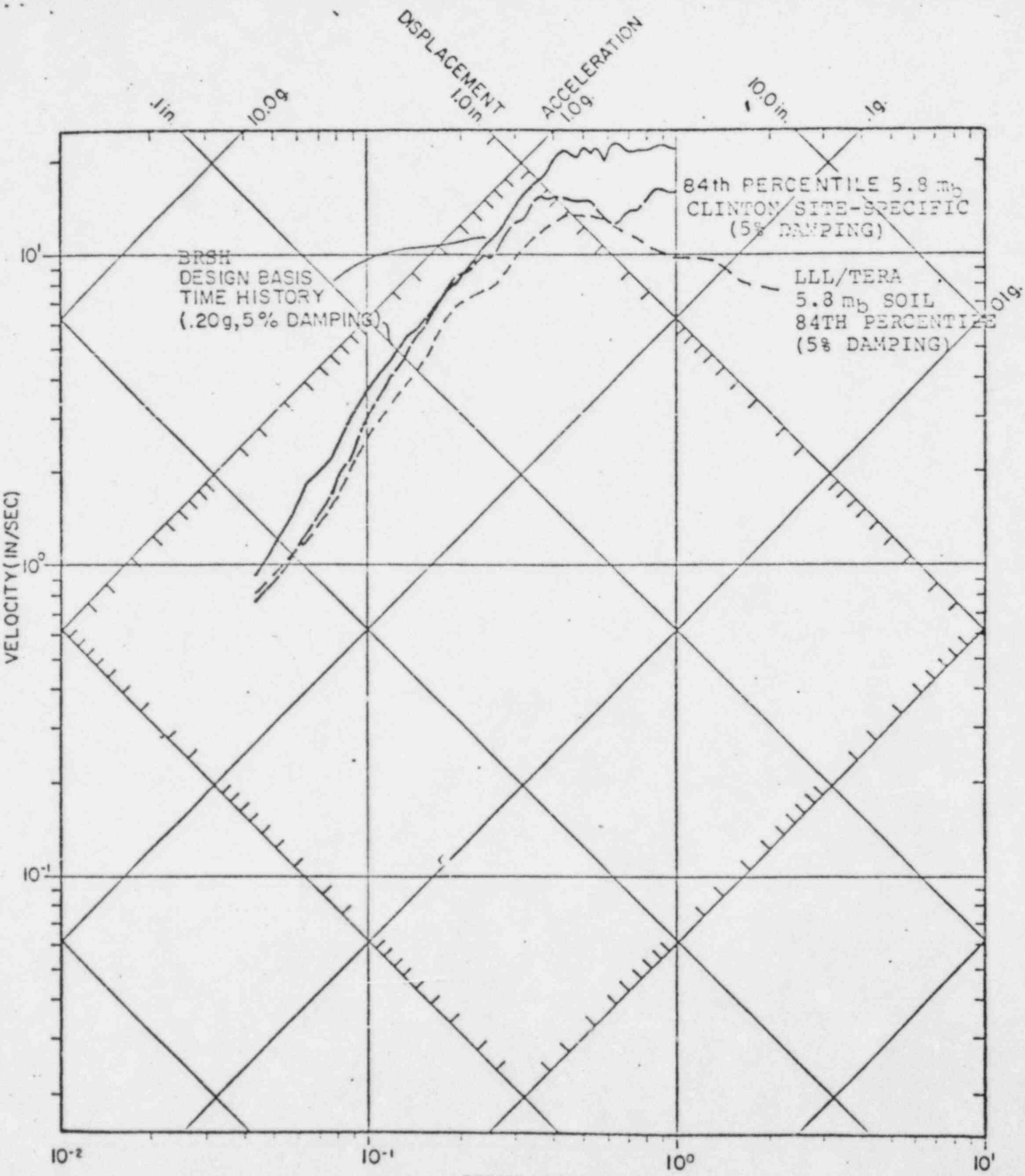
BYRON RIVER SCREEN HOUSE SITE SOIL PROPERTIES

SOIL LAYER	LAYER DEPTH (ft)	WEIGHT DENSITY (K/ft^3)	DAMPING RATIO	SOIL SHEAR MODULUS (K/ft^2) UPPER	MEAN	LOWER
1	16.	0.123	0.10	1850.	1157.	771.
2	18.	0.123	0.10	1850.	1157.	771.
3	24.	0.123	0.10	1850.	1157.	771.
4	32.	0.123	0.10	1850.	1157.	771.
5	Half Space	0.150	0.10	450000.	450000.	450000.

TABLE Q362.1-2

EARTHQUAKES USED TO DETERMINE SPECTRAL AMPLIFICATION FACTORS

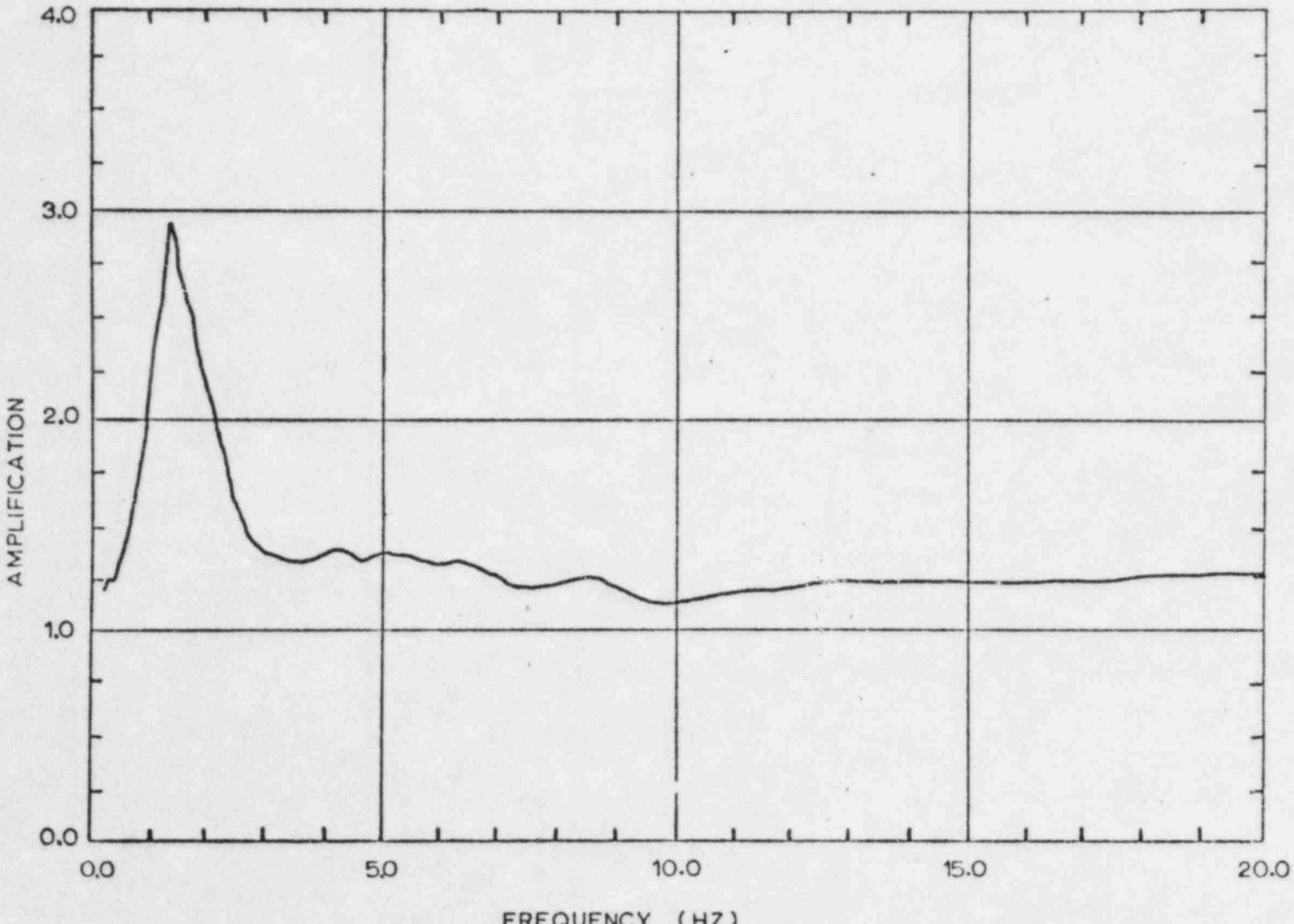
EARTHQUAKE	DATE/TIME	RECORDING STATION	MAGNITUDE	EPICENTRAL DISTANCE (km)	INSTRUMENT ORIENTATION	MAXIMUM ACCEL-ERATION (g)
Helena, Montana	10-31-35/1138 MST	Caroll College	6.0	7	S00W S90W	.146 .145
Eureka, California	12-21-54/1156 PST	Eureka Federal Building	6.6	25	N11W N79E	.168 .258
San Francisco, California	3-22/57/1144 PST	Golden Gate Park	5.3	13	N10E S80E	.084 .105
Parkfield, California	6-27-66/2026 PST	Temblor	5.5	7	N65W S25W	.270 .348
Parkfield, California	6-27-66/2026 PST	Chalame-Shandron, Array No. 5	5.5	6	N05W N85E	.355 .434
San Fernando, California	2-9-71/0600 PST	Castiac Old Ridge Route	6.6	30	N21E N69W	.316 .271
San Fernando, California	2-9-71/0600 PST	Pacoima Dam, After Shock at 104.6 sec	3.0-5.0	9	574W 516E	.112 .115



BYRON-FSAR

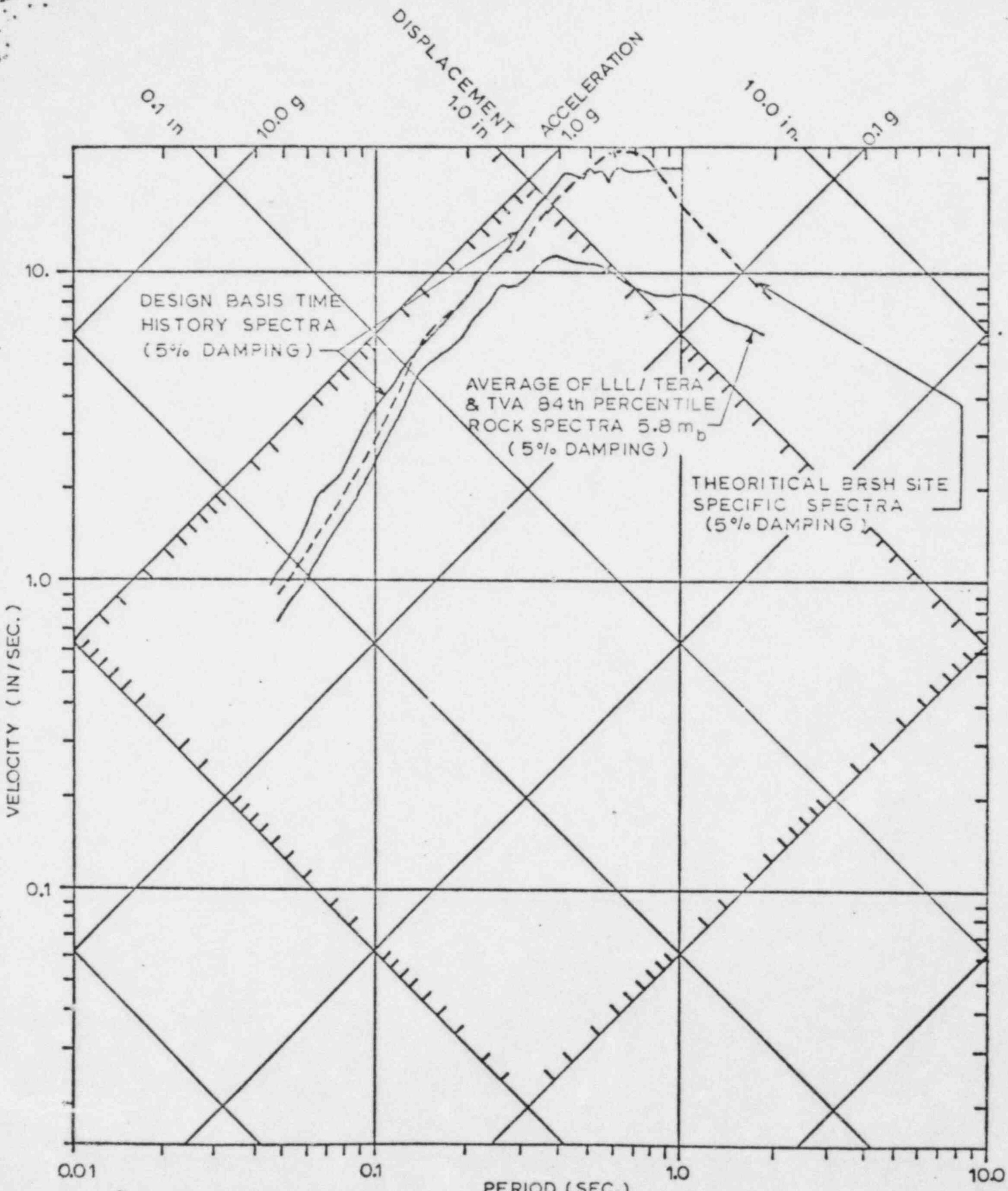
FIGURE Q362.1-1:

Comparison of Byron Design Basis Time History Spectra with the 5.8 m_b 84th Percentile Clinton Site Specific Spectra and the 5.8 m_b 84th Percentile LLL/Tera Soil Site Spectra



BYRON-FSAR

FIGURE Q362.1-2: Most Probable Site Response Spectrum Amplification Function



BYRON-FSAR

FIGURE Q362.1-3: Comparison of Spectrum From Design Basis Time-History With Theoretical Site Specific Spectrum Using Soil Amplification Applied to Rock Spectra

QUESTION 241.8

"To verify the seismic analyses of the river screen house foundation are sufficiently conservative at higher frequency range, perform a confirmatory seismic response analysis. The staff requires that the higher shear moduli values obtained from the laboratory tests (FSAR, Figure 2.5.89), i.e., the moduli of the reconstituted samples, be used in the analyses. To account for variability of the soil properties, the analyses using the upper bound shear moduli plus or minus 50 percent of their values should also be included. The response to this question is a necessary element of the response to Q362.1 because of the need for proper characterization of the soil conditions for a site specific spectrum or amplification calculation."

RESPONSE

EVALUATION OF EXISTING TEST RESULTS

Figure Q241.8-1 shows the results of the cyclic triaxial tests on both intact and recompacted samples plotted in a normalized form. (It should be noted that the foundation for the river screen house was placed on natural soils, and that the river screen house, therefore, is not resting on recompacted material.) The points plotted on this figure were established by anchoring the shear modulus (G) at the lowest strain level obtained during the triaxial tests on the Seed and Idriss (Reference 5) normalized shear modulus (G/G_{max}) strain degradation curve. By obtaining the normalized shear modulus value at this strain level, a G_{max} value for the tests was obtained based on the mean Seed and Idriss (Reference 5) strain degradation curve for strains smaller than the minimum shear strain for the tests. The other data points were then normalized based on the obtained G_{max} value.

The results of normalizing the test data show that the undisturbed samples exhibit strain degradation characteristics within the range postulated by Seed and Idriss (Reference 5); however, the recompacted soil samples follow strain degradation curves unreasonably steep compared to the normalized curve. We believe these characteristics are the result of the test procedures rather than actual soil properties. The tests on the recompacted samples, completed approximately 1 year prior to the tests on the intact samples, were performed on a machine that had been calibrated according to standard procedures, but not corrected for piston friction. A correction for piston friction is usually not necessary for property tests at high strain values, where high loads are required to obtain the desired deformation. At small strains, where smaller loads are required, the piston friction represents a significant portion of the recorded load. It should be

noted that data from dynamic triaxial tests were at the time of the tests considered reliable only for shear strains greater than 0.01%. Regulatory Guide 1.138 shows that the strain range for the cyclic triaxial test is limited to shear strains greater than 0.01%. The calculated moduli at lower strains are, therefore, believed to be larger than the true values, which results in an apparent very steep strain degradation relationship from these incorrectly high shear moduli for low strains. In conclusion, it is our opinion that the curves for the reconstituted samples are probably in error at low strains and are not representative of the granular material underlying the river screen house. This opinion is supported further by more recent test results (Reference 6) which show that the strain degradation curves for granular material may be flatter than those suggested by Seed and Idriss.

SHEAR MODULUS FACTORS FROM EMPIRICAL RELATIONSHIPS

The test data on undisturbed samples are presented in the form of normalized shear modulus factor K_2 versus strain on Figure Q241.8-2. The resonant column data were recorded at a shear strain on the order of 10^{-4} percent and, thus, represent anchor points. The data indicate normalized shear modulus factors in the range of 40 to 85.

The shear modulus factors for the deposits underlying the river screen house were also evaluated using the empirical expression given by Hardin and Drnevich (Reference 2). The average K_{2max} obtained using this procedure was 68 (Figure Q241.8-3). However, Hardin (Reference 3) has proposed that the shear modulus for granular material is also a function of grain size, in particular the particle size at which 5 percent of the sample is finer (D_5). Using the procedure proposed by Hardin and a D_5 of 0.2 mm (see FSAR Figure 2.5-49), a shear modulus factor of 75 was obtained. Thus, the K_{2max} values obtained using the empirical relationships proposed in References 2 and 3 (68 to 75) fall within the range (65 to 90) given in FSAR Figure 2.5-89.

In addition to using the empirical relationship proposed by Hardin and Drnevich, the shear modulus factor for the desposits underlying the river screen house were evaluated using the porcedures proposed by Ohsaki and Iwasaki (Reference 4). This procedure is based on an empirical relationship between dynamic shear modulus as determined from field measurements and standard split spoon resistance (SPT). This method has been used for sandy and gravelly soils successfully, and therefore should be applicable to sandy and gravelly soils at the river screen house. Thus, using the SPT values at the river screen house, shear modulus factors in the range of 57 to 122 were obtained, with a mean shear modulus factor of 85

(Figure Q241.8-3). It should be noted, however, that evaluation of field data (Reference 7) indicates that the procedure proposed by Ohsaki and Iwasaki generally overestimates the field shear modulus by approximately 25 percent.

DATA FROM FIELD GEOPHYSICAL MEASUREMENTS AT OTHER SITES

The shear modulus factors obtained for the materials underlying the river screen house were also compared with those calculated from field data obtained at other sites (Reference 1). The results, shown in Table Q241.8-1 and in Figure Q241.8-4, show shear modulus factors in the range of approximately 40 to 100 for sites with penetration resistance similar to those encountered under the river screen house.

The shear wave velocities presented at other sites (Reference 1) were also plotted versus mean standard penetration resistance. The results are shown on Figure Q241.8-5 together with the shear wave velocity calculated using the shear moduli obtained from the procedures of Ohsaki and Iwasaki. The data plotted in Figure Q241.8-5 show that the procedures proposed by Ohsaki and Iwasaki are in good agreement, although close to an upper bound, with field data presented by Shannon and Wilson (Reference 1).

Figure Q241.8-6 shows calculated shear wave velocities versus depth based on a wide range of normalized shear moduli. Also shown are the shear wave velocities corresponding to the shear moduli obtained using the empirical relationships proposed by Hardin and Drnevich (Reference 2), Hardin (Reference 3), and Ohsaki and Iwasaki (Reference 4). Based on the empirical relationships, the shear wave velocity at the site of the river screen house may vary between approximately 750 and 1600 fps. These velocities are in good agreement with the field data presented by Shannon and Wilson (Reference 1 and Figure Q241.8-5) which show shear wave velocities in the same range for sites with similar standard split spoon penetration resistances to those encountered under the river screen house at Byron.

RESULTS FROM FIELD GEOPHYSICAL MEASUREMENTS AT BYRON RIVER SCREEN HOUSE

A seismic crosshole survey was conducted adjacent to the location of the river screen house to obtain seismic shear and compressional wave velocities of soils representative of those underlying the screen house. A report presenting procedures and results of the survey is given in Byron FSAR Attachment 2.5J. Figure Q241.8-7 shows measured shear wave velocities versus depth. These velocities range between 750 and 1600 fps as predicted. Figure Q241.8-8 shows the shear modulus factor K_{2max} versus depth based on the measured shear wave velocity. The data presented show a mean value of K_{2max} of 79. This value is within the range (K_{2max} between 65 and 90) shown in Figure 2.5-89.

CONCLUSIONS

Results of K_{2max} obtained from calculations based an empirical relationships (References 2, 3, and 4) and geophysical measurements at the Byron river screen house and at other sites indicate that the values presented in Figure 2.5-89 are reasonable and correct for the materials underlying the river screen house.

The variation of shear modulus (G_{max}) at low strain levels (10^{-6} inch/inch) calculated from the measured shear wave velocities given in Figure Q241.8-7 is presented in Figure Q241.8-9. The cumulative average with depth of G_{max} indicates that G_{max} varies from a low of 2794 ksf at a 25-foot depth to a high of 6104 ksf at a 110-foot depth.

The range in shear modulus values used in the SHAKE analysis for the Byron river screen house was given in Table Q362.1-1. Figure Q241.8-10 presents shear modulus versus shear strain variations determined by anchoring the shear modulus values used in the SHAKE analysis at their appropriate strain and matching the mean Seed and Idriss strain degradation curve to obtain corresponding variations of G_{max} at very low strains. The range in G_{max} from these curves are a low of 3083 ksf to a high of 7398 ksf. The measured G_{max} values presented in Figure Q241.8-9 substantially fall within these values and confirm that the shear modulus variation presented in the response to Question 362.1 is acceptable.

REFERENCES

1. Shannon and Wilson, Inc. and Agbabian Associates, 1980, Geotechnical data from accelerograph stations investigated during the period 1975-1979; Summary Report, prepared for U.S. Nuclear Regulatory Commission, NUREG/CR-1643 (September).

2. Hardin, B.O., and Drnevich, V.P., 1972, Shear modulus and damping in soils: measurement and parameter effects: Journal of the Soil Mechanics and Foundation Division, ASCE, vol. 98, no. SM6 (June).

3. Hardin, B.O., 1973, Shear modulus of gravels: University of Kentucky Publ. no. TR74-73-CE19 (September).

4. Ohsaki, Y., and Iwasaki, R., 1973, On dynamic shear moduli and Poisson's ratios of soil deposits: Soils and Foundations, vol. 13, no. 4 (December).

5. Seed, H.B., and Idriss, I.M., 1970, Soil moduli and damping factors for response analysis: University of California, Earthquake Engineering Research Center, Berkeley, Report no. EERC70-10 (December).

6. Arango, I., Moriwaki, Y., and Brown, F., 1978, In-situ and laboratory shear velocity and modulus: Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference on Earthquake Engineering and Soil Dynamics, Pasadena, California (June).

7. Anderson, D.G., Espana, C., and McLamore, V.R., 1978, Estimating in-situ shear moduli at competent sites: Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference on Earthquake Engineering and Soil Dyanmics, Pasadena, California (June).

8. Gibbs, H.J., and Holtz, W.G., 1957, Research on determining the density of sand by spoon penetration test: Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, vol. I, pp. 35-39.

9. Mayne, P.W., and Kulhawy, F.H., 1982, K_o-OCR relationships in soil: Journal of the ASCE Geotechnical Engineering Division vol. 108, no. GT6 (June).

10. Marcuson, W.F., and Bieganousky, W.A., 1976, Laboratory standard penetration tests on fine sands: ASCE Annual Convention and Exposition, Liquefaction Problems in Geotechnical Engineering, Philadelphia, Pennsylvania (September).

TABLE Q241.8-1

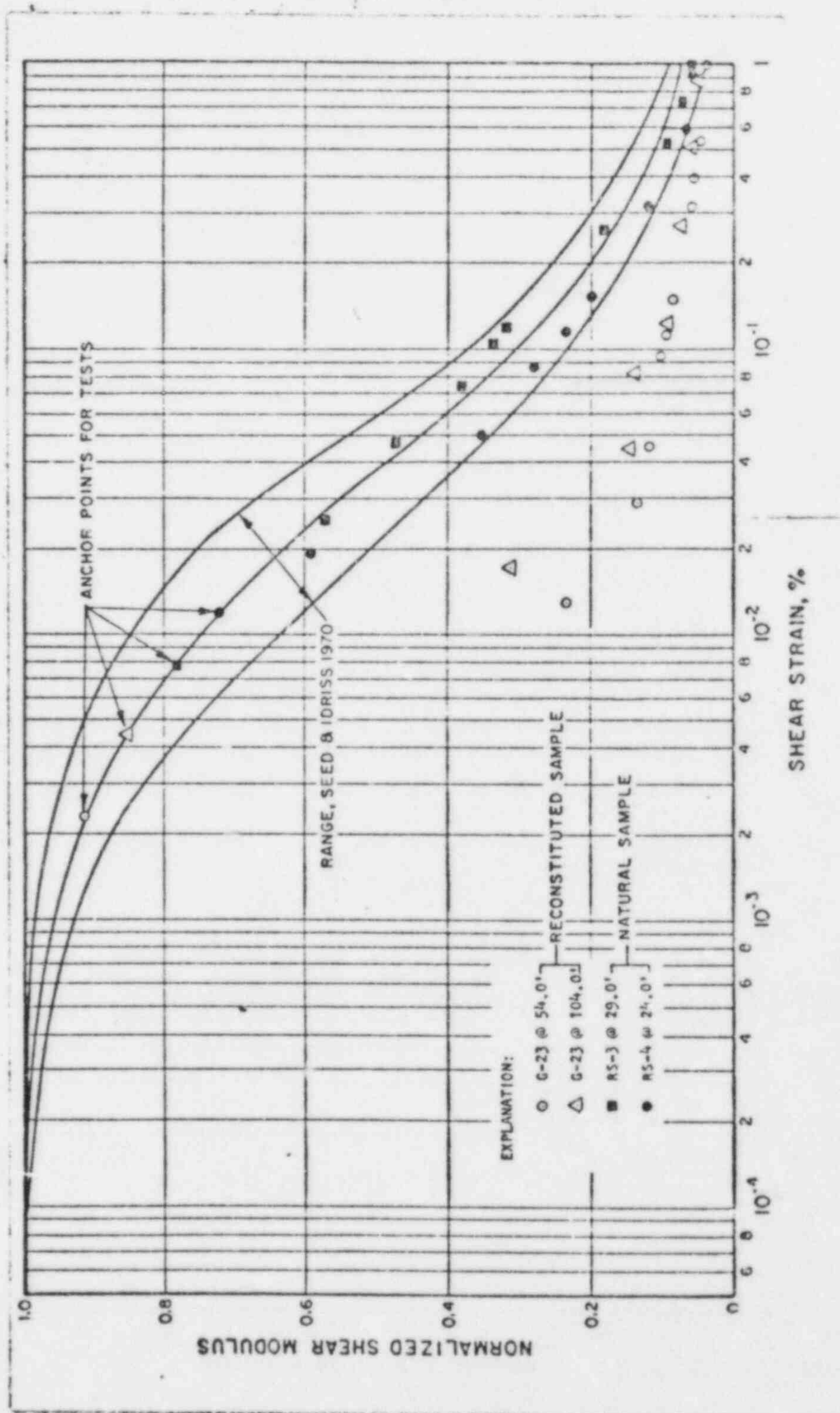
GEOPHYSICAL PROPERTIES AND NORMALIZED SHEAR MODULUS FACTOR FOR GRANULAR MATERIAL

SITE	SOIL CONDITIONS	MEAN BLOW COUNT SPT	DEPTH (ft)	SHEAR WAVE VELOCITY (fps)	DEPTH TO WATER TABLE (ft)	K_0*	K_2
Cholame-Shandon Array California (In-situ Impulse/ Downhole, 1975)	SM Alluvium (Holocene)	81	135	900	26	1.0	31
		56	165	1,100		1.0	43
Terminal Substation El Centro, California (Downhole, 1975)	SM Lake Deposit (Quartenary)	88	95	850	33	1.0	31
Highway Test Lab Olympia, Washington (Crosshole, 1974)	SP-SM Glaciolacustrine Deposit	29	25	750	12	0.6	53
Cit Millikan Library Pasadena, California (Downhole, 1975)	SM Alluvium (Pleistocene)	126	40	1,550	238	1.0	132
		180	80	1,550		1.0	93
		155/5"	120	1,950		1.0	121
4800 Oak Grove Pasadena, California (Downhole, 1978)	SW-GW	42	20	1,100	225	0.8	103
	SM-SW	103	40	1,600		1.0	143
	SM-SW	(very dense)	80	1,600		1.0	101
	SM-SW-GW Alluvium (Pleistocene)	(very dense)	140	2,000		1.0	120
State Building San Francisco, California (Downhole, 1978)	SP (Quaternary) Sediments)	36	25	1,000	20	0.6	87
		49	55	1,100		1.0	70
		122	80	1,600		1.0	127

*Estimated based on Gibbs and Holtz, 1957 (Reference 8); Marcuson and Bieganousky, 1976 (Reference 10); and Mayne and Kulhawy, 1982 (Reference 9).

TABLE Q241.8-1 (Cont'd)

SITE	SOIL CONDITIONS	MEAN BLOW COUNT SPT	DEPTH (ft)	SHEAR WAVE VELOCITY (fps)	DEPTH TO WATER TABLE (ft)	K_0*	K_2
Lincoln School Tunnel	SM	71	25	1,200	<200	1.0	98
Taft, California	Alluvium	121	50	1,200		1.0	69
(Downhole, 1976)	(Quaternary)	103	80	1,600		1.0	97
Noranda Aluminum Plant	SP/SP-SM	23	25	850	11	0.4	77
New Madrid, Missouri	Alluvium	32	75	900		0.9	45
(Downhole, 1979)	(Quaternary)	69	120	1,000		1.0	43
MSU Roberts Hall	GW	35	14	750	8	1.0	60
Bozeman, Montana	Alluvium	85/6"	25	1,300		1.0	146
(In-situ Impulse, 1976)	(Quaternary)						
PSU Cramer Hall	GW	106/6"	105	1,800	133	1.0	112
Portland, Oregon							
(Downhole, 1978)							
USU Old Main Building	SW-GW	23	15	900	150	0.6	86
Logan, Utah	Alluvium	-	50	1,300		0.5	103
(Downhole, 1976)	(Quaternary)	66	90	1,600		1.0	96
1900 Avenue of the Stars	SP/SM	102/6"	80	1,300	64	1.0	70
Los Angeles, California	Pleistocene	124	120	1,800		1.0	119
(Downhole, 1975)	(Some Cementation)						
Hollywood Storage Bldg.	SM w/Gravel	61	120	1,400	40	1.0	77
Los Angeles, California	Alluvium						
(Downhole, 1979)	(Quaternary)						

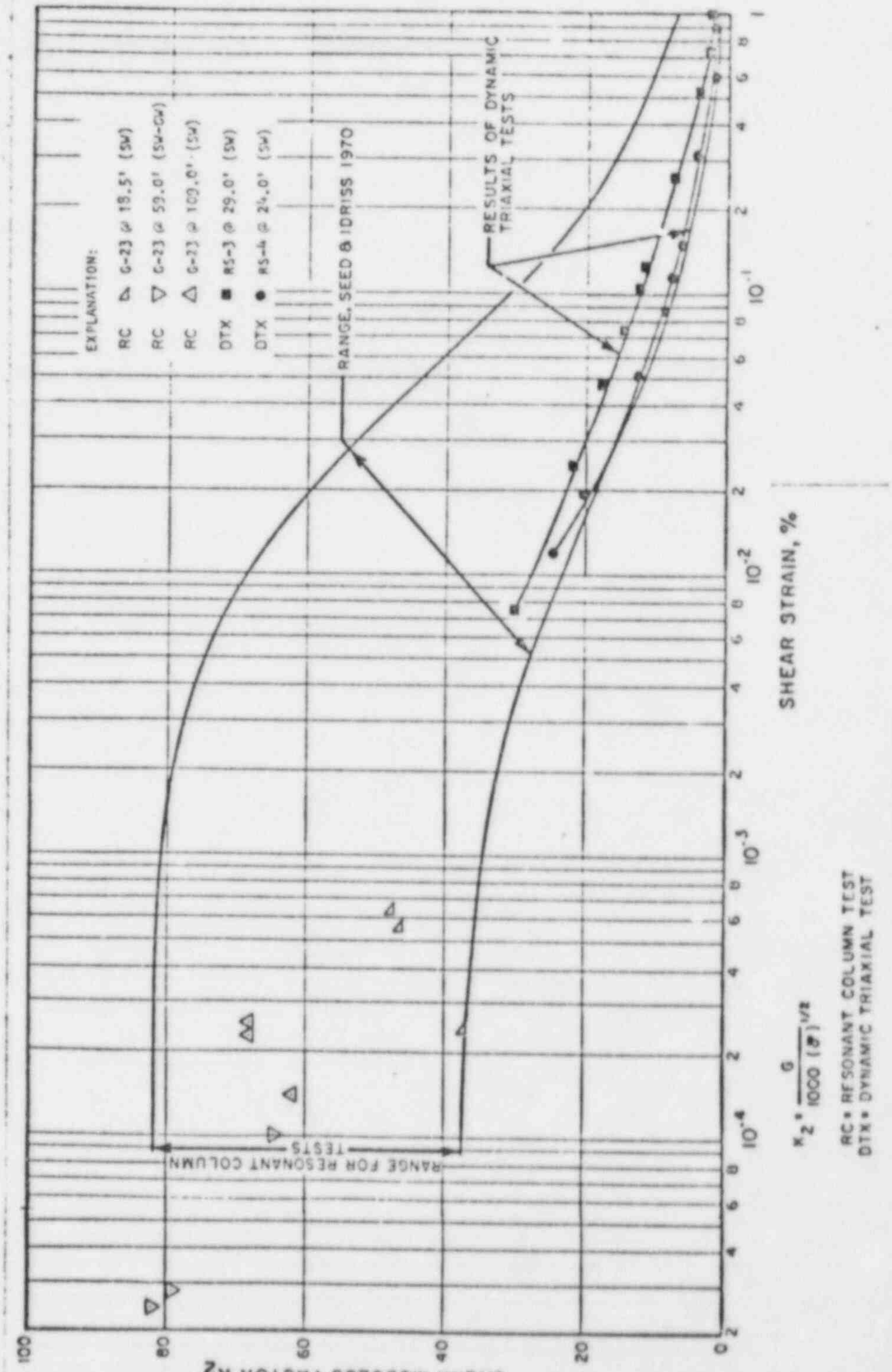


BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-1

NORMALIZED SHEAR MODULUS
VERSUS SHEAR STRAIN

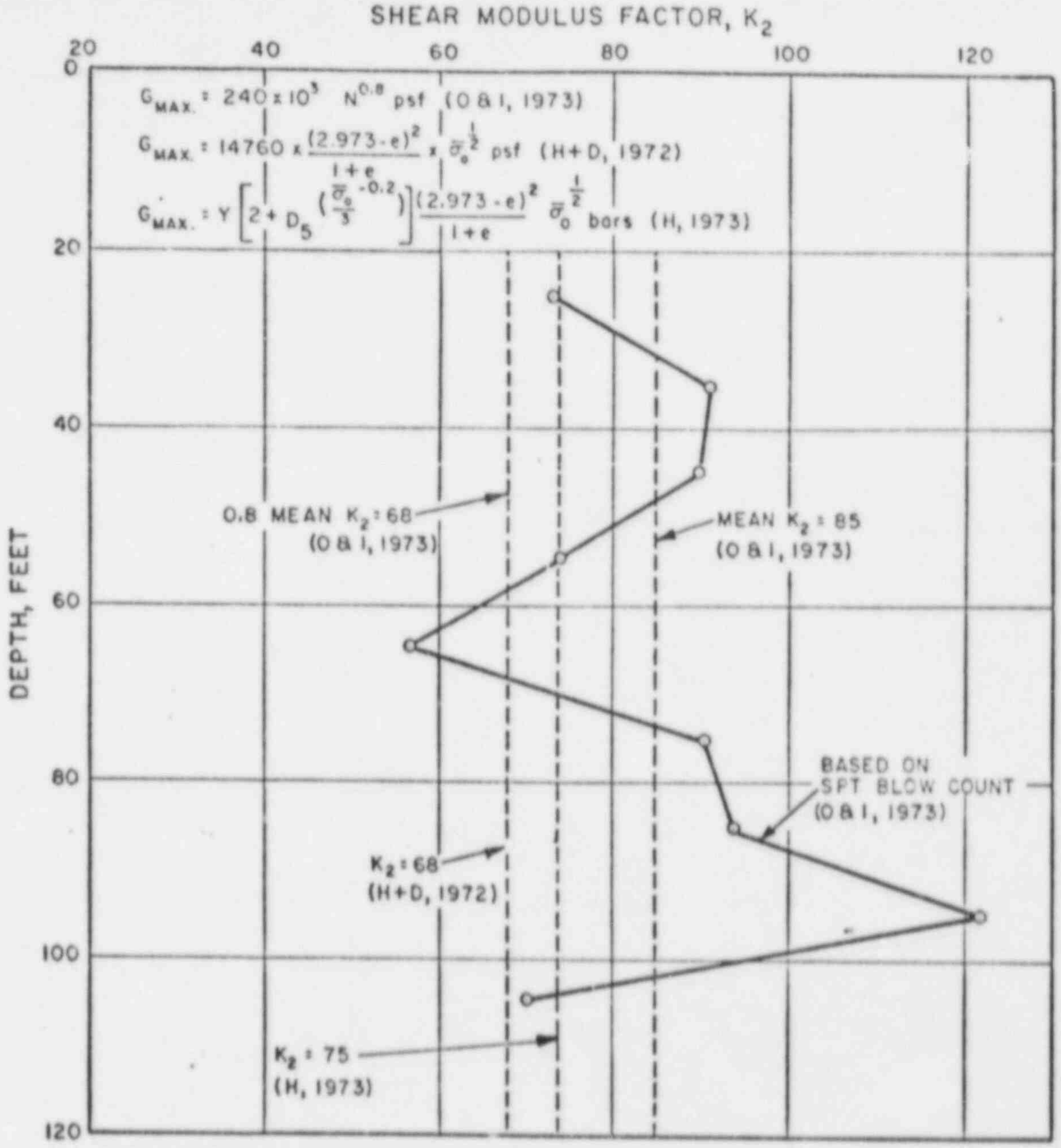


BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-2

SHEAR MODULUS FACTOR K_2
VERUS SHEAR STRAIN



N = STANDARD PENETRATION TEST BLOW COUNT

e = VOID RATIO

$\bar{\sigma}_o$ = MEAN EFFECTIVE CONFINING STRESS = $\frac{\bar{\sigma}_v}{3}(1+2K_o)$

$\bar{\sigma}_v$ = EFFECTIVE VERTICAL STRESS

K_o = AT-REST COEFFICIENT OF LATERAL EARTH PRESSURE

D_5 = GRAIN SIZE (mm) FROM GRADATION CURVE AT 5% FINER THAN

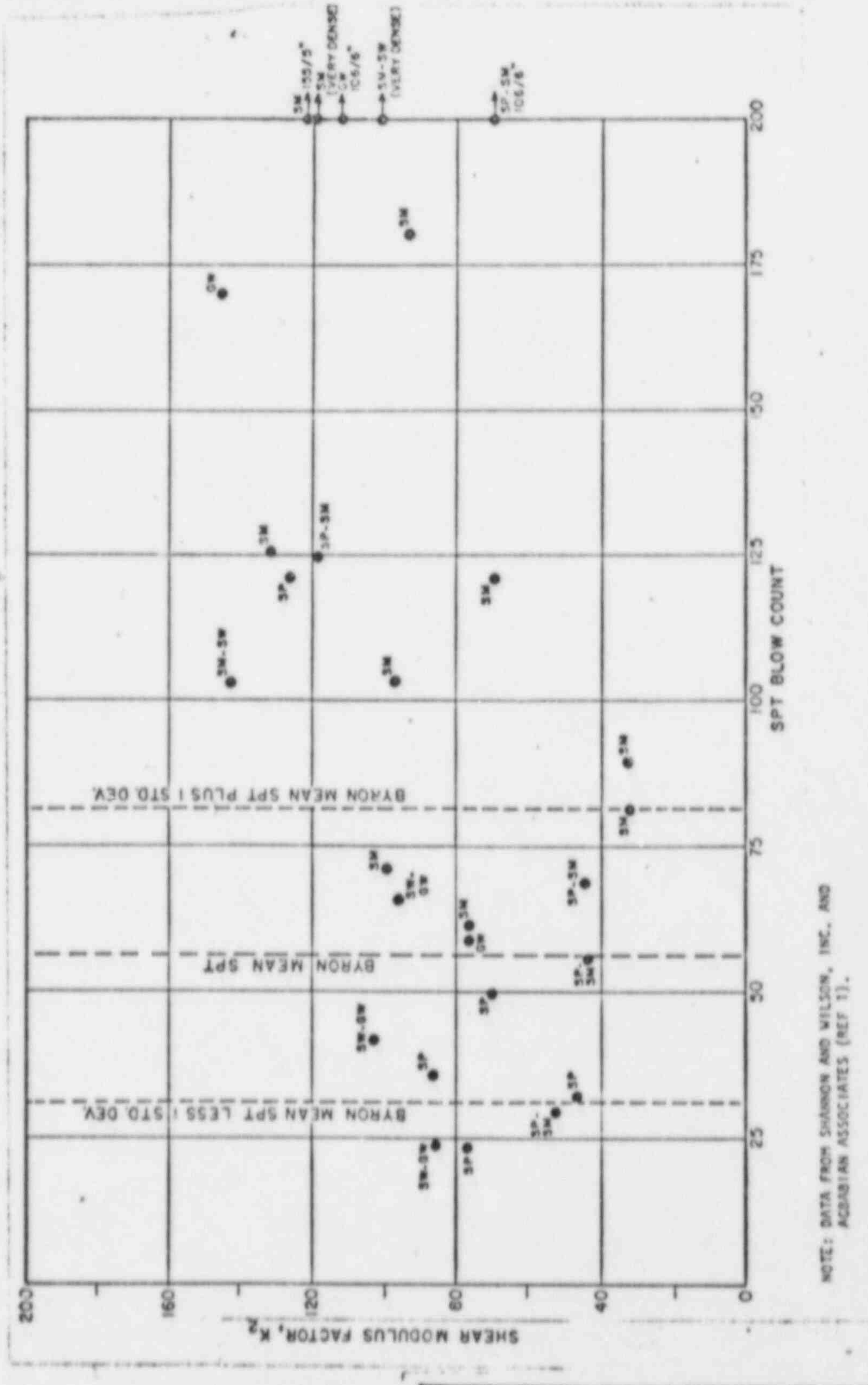
BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-3

SHEAR MODULUS FACTOR K_{2MAX} VERSUS DEPTH

(CALCULATED USING REF. 2, 3, and 4)

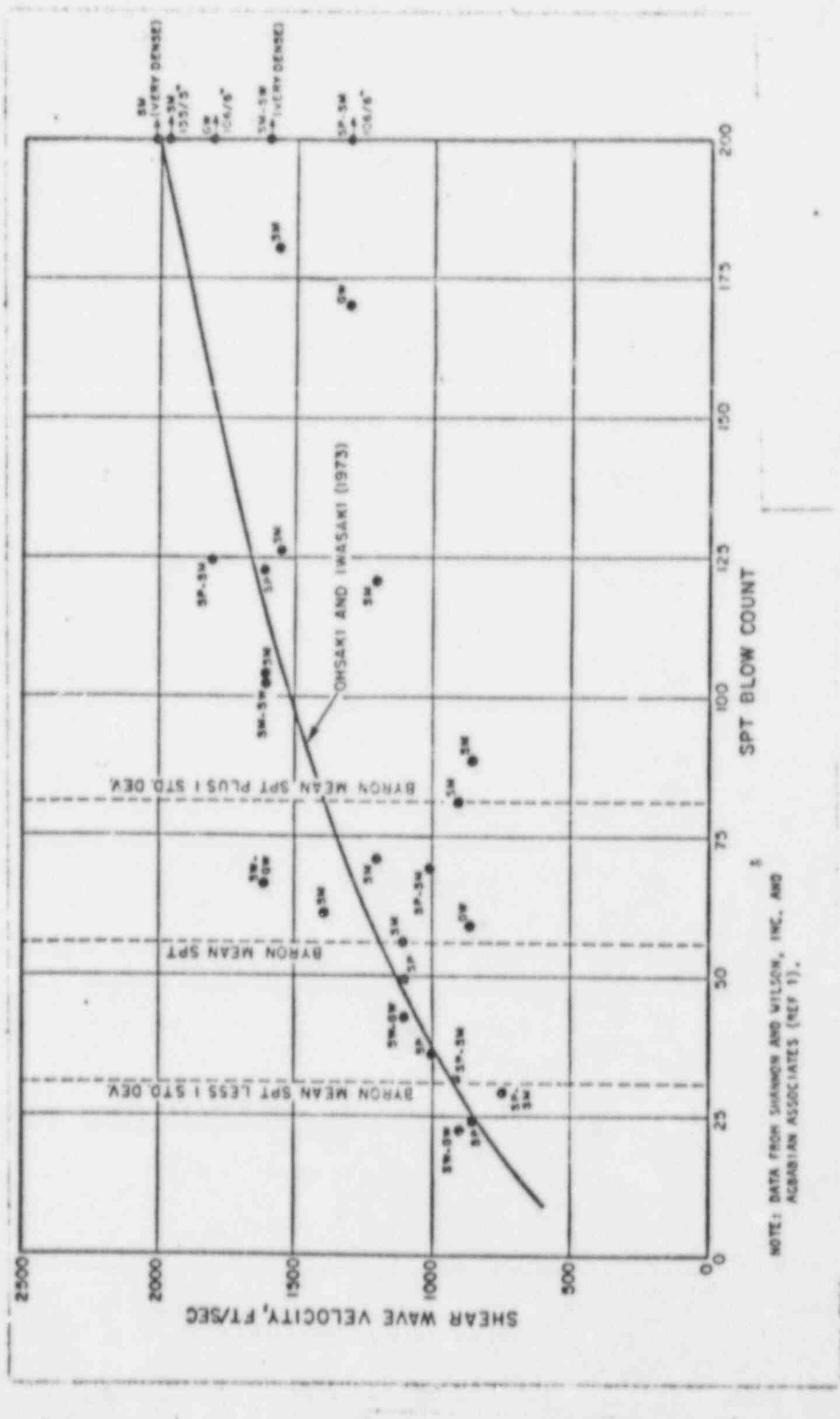


BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-4

SHEAR MODULUS FACTOR K_2 VERSUS SPT BLOW COUNT; GRANULAR MATERIAL

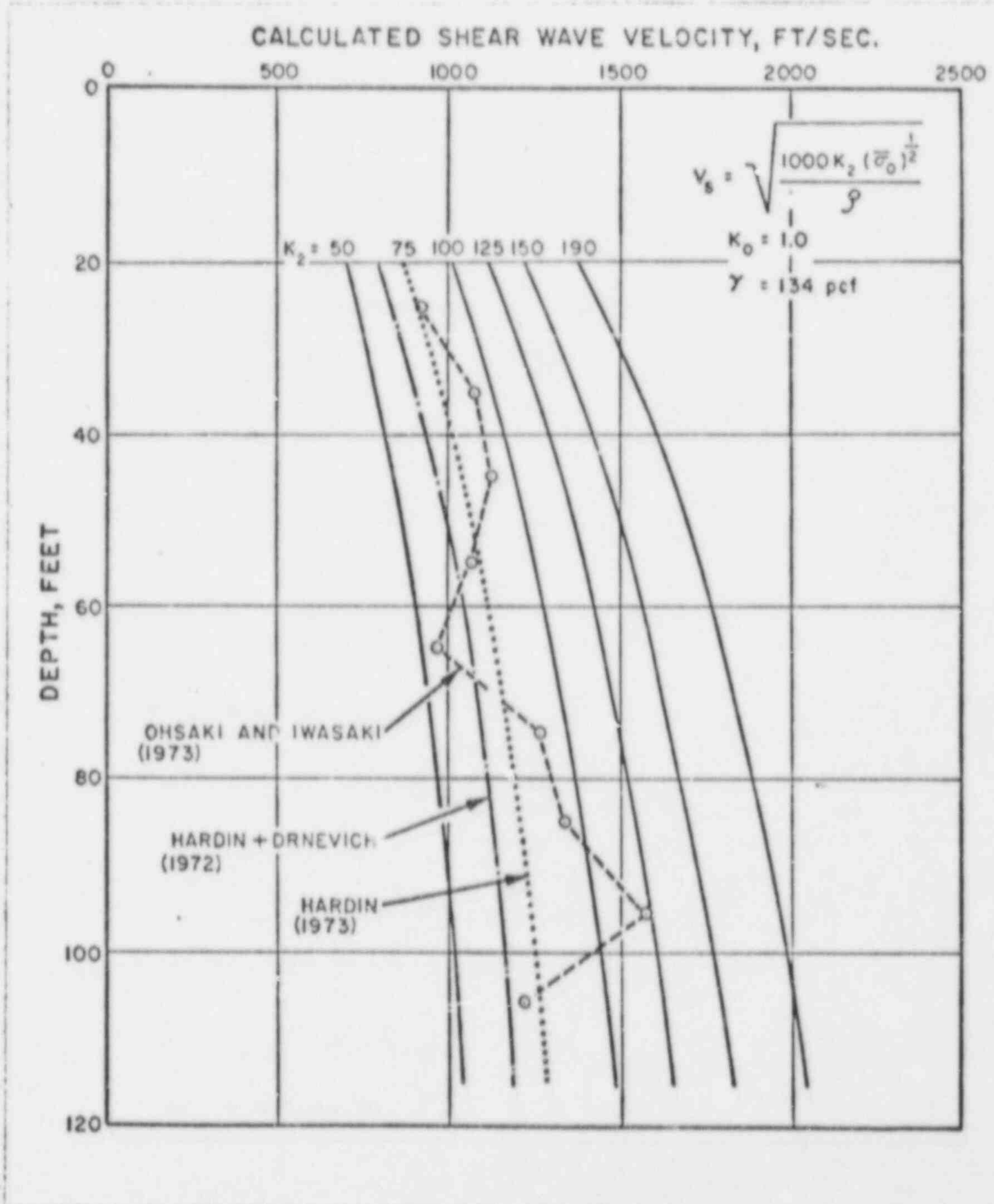


BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-5

SHEAR WAVE VELOCITY
VERSUS SPT BLOW COUNT
GRANULAR MATERIAL

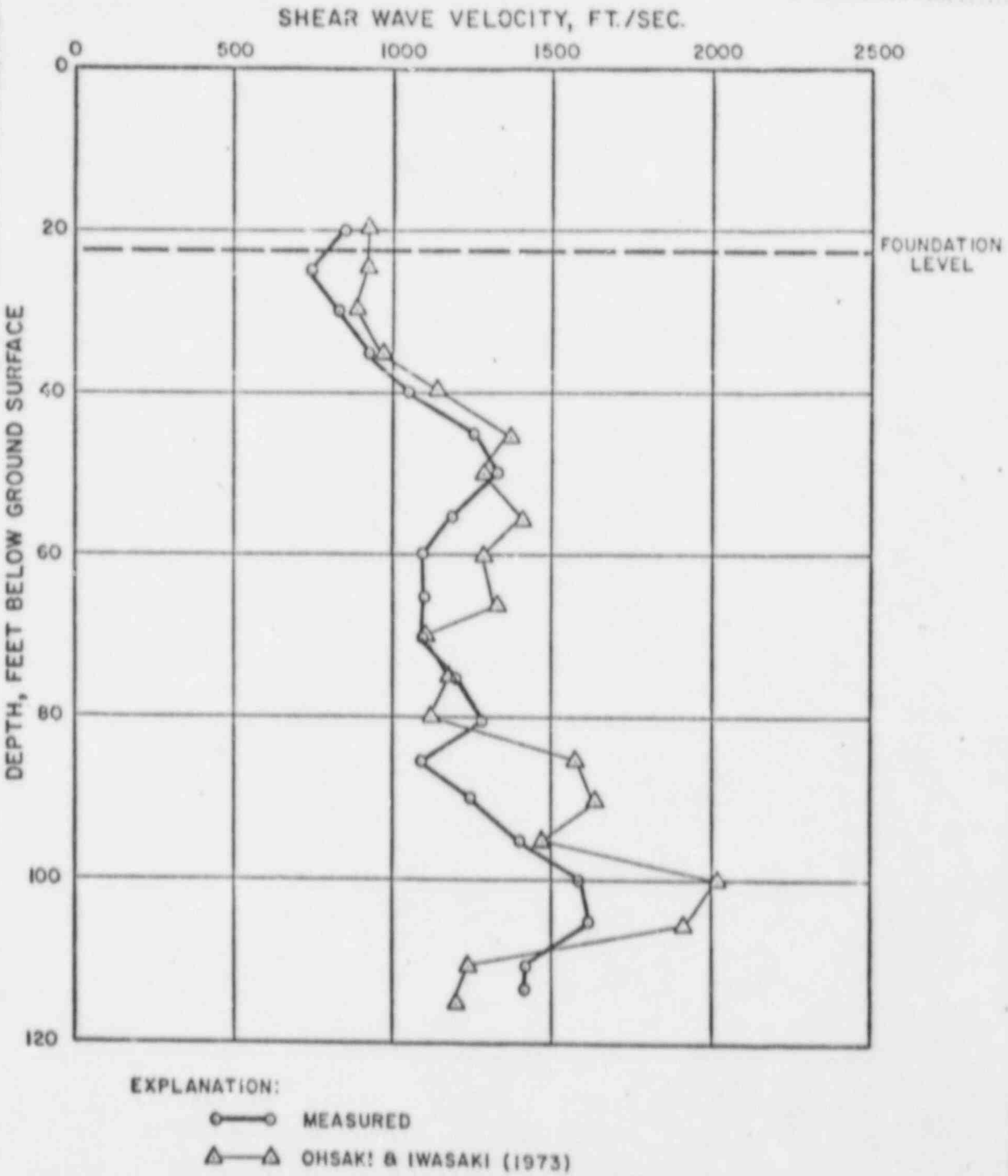


BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-6

CALCULATED SHEAR WAVE VELOCITIES VERSUS DEPTH



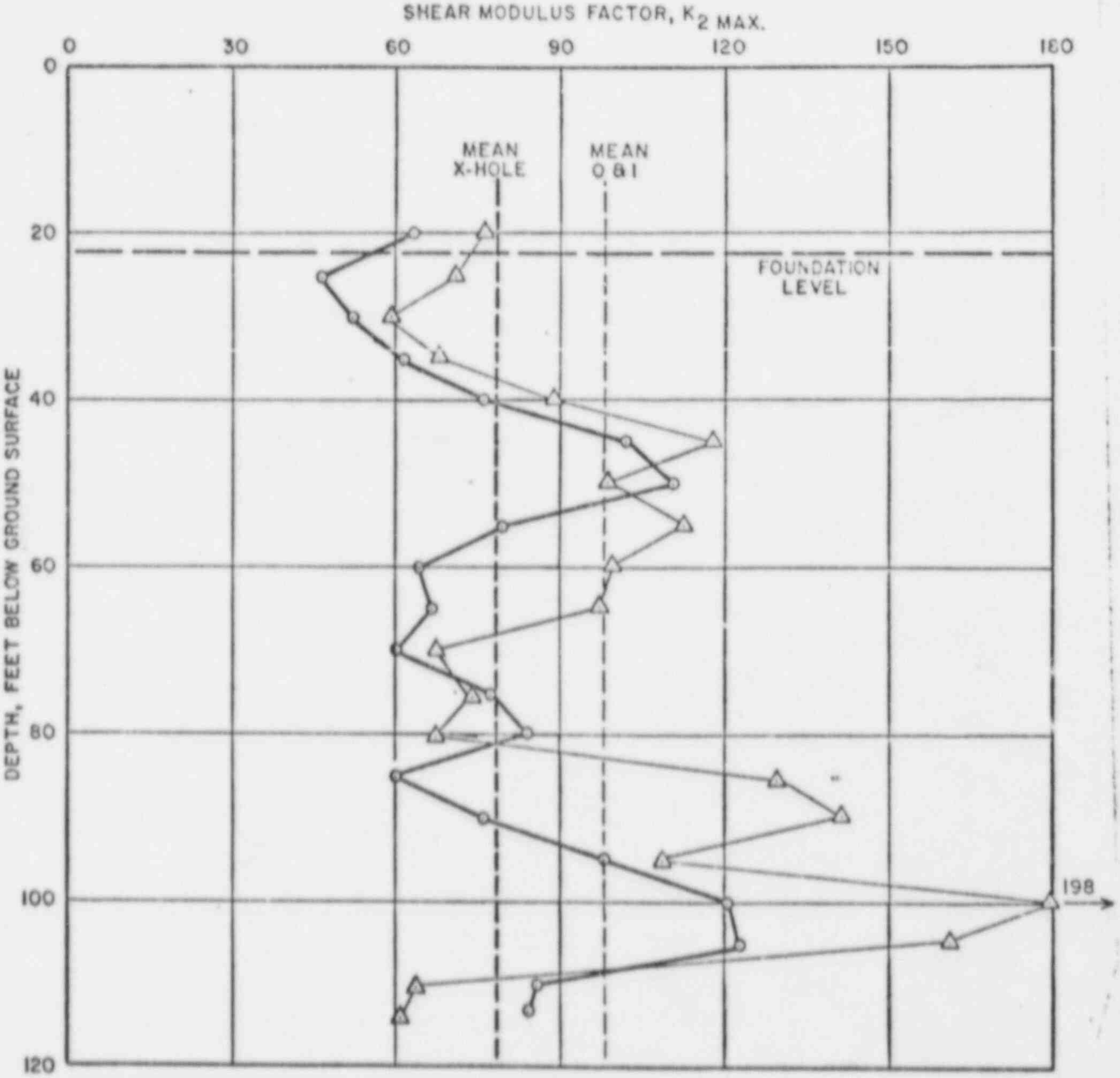
NOTES: 1. THE VELOCITIES DETERMINED FROM THE CROSS-SURVEY WERE DERIVED FROM MEASUREMENTS MADE FOR BORINGS XH-1 AND XH-2, AND ARE CORRECTED TO ELIMINATE REFRACTED WAVE PATHS.
2. GROUND SURFACE AT ELEVATION 683.

BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-7

SHEAR WAVE VELOCITY VERSUS DEPTH



EXPLANATION:

o——o X-HOLE (MEASURED)

△——△ OHSAKI & IWASAKI (O&I)

NOTES: 1. $K_2 = \frac{G}{1000\sqrt{\bar{\sigma}_0}}$

$\bar{\sigma}_0$ = MEAN EFFECTIVE STRESS

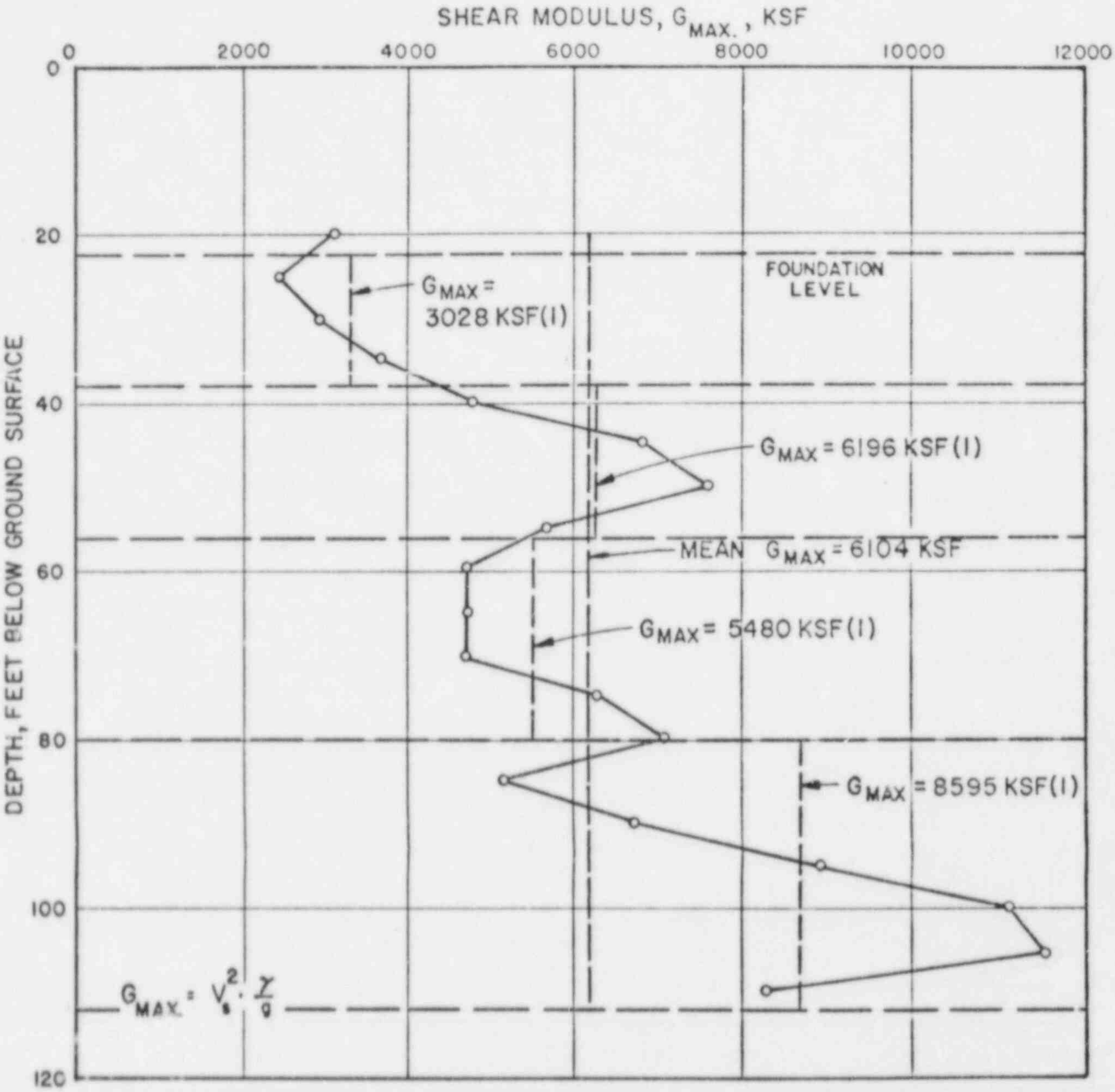
2. GROUND SURFACE AT ELEVATION 683.

BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-8

SHEAR MODULUS FACTOR K_{2MAX} VERSUS DEPTH



NOTES:

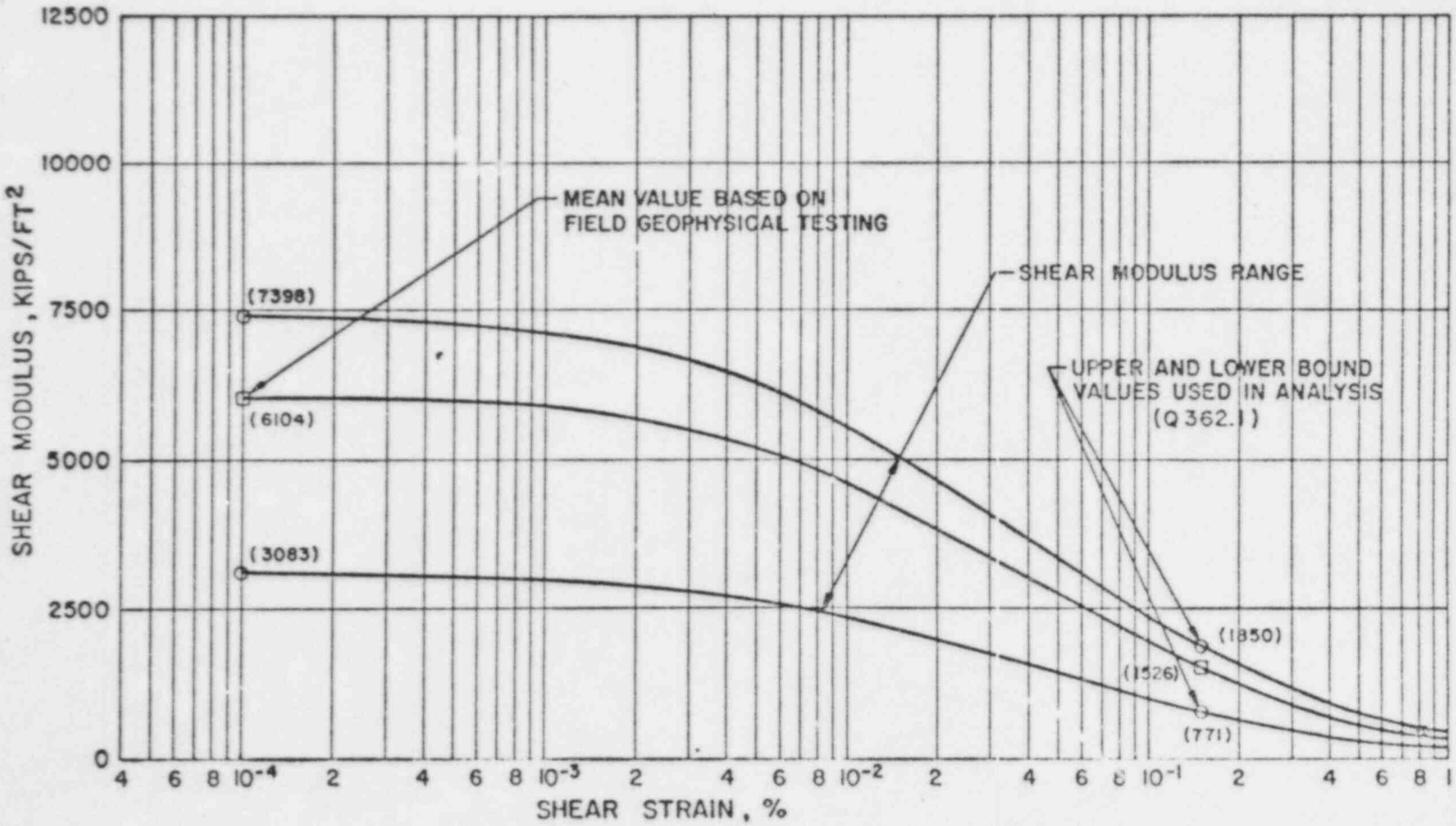
(1) AVERAGE G_{MAX} VALUES FOR INDIVIDUAL SOIL LAYERS. SOIL LAYERS IDENTICAL TO THOSE USED IN ANALYSIS PRESENTED IN RESPONSE TO Q362.1

BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-9

SHEAR MODULUS G_{MAX} VERSUS DEPTH



BYRON STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.8-10

SHEAR MODULUS VERSUS SHEAR STRAIN VARIATIONS