UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

TESTIMONY

OF

DR. RICHARD D. WOODS

ON BEHALF OF THE APPLICANT

REGARDING LIQUEFACTION OF SATURATED SAND

DURING AN EARTHQUAKE AT THE MIDLAND SITE

Testimony of
Dr. Richard D. Woods
Regarding
Liquefaction of Saturated Sand
During an Earthquake
At The Midland Site

SS: STATE OF MICHIGAN
COUNTY OF WASHTENAW

UNITED SATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

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	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-329 OL

AFFIDAVIT OF RICHARD D. WOODS

Richard D. Woods, being duly sworn, deposes and says that he is the author of "Testimony of Richard D. Woods concerning Liquefaction Potential at the Midland Site," and that such testimony is true and accurate to the best of his knowledge and belief.

Richard D. Woods
RICHARD D. WOODS

Sworn and Subscribed Before Me this 15 Day of October, 1982

Beverly A Bross
Notary Public
Washtenaw County, Michigan

My Commission Expires November 30, 1982

BEVERLY A. BROSS
NOTARY PUBLIC, WASHTENAW CO., MICH
MY COMMISSION EXPIRES NOV. 30, 1982

LIQUEFACTION OF SATURATED SAND DURING EARTHQUAKE

1.0 BIOGRAPHICAL INFORMATION

This is the testimony of Dr. Richard D. Woods. My detailed resume is attached. The following is a summary of that resume. I received a Bachelor of Science degree in Civil Engineering from Notre Dame University in 1957 and a Master of Science degree from the same school in 1962. I worked for the Air Force Weapons Center, Albuquerque, New Mexico, on the design of blast resistant underground structures for one year and taught in the Civil Engineering Department at Michigan Technological University for one year before going to the University of Michigan for a Ph.D. in Civil Engineering, which I received in 1967. Since then I have been on the faculty of the Department of Civil Engineering at the University of Michigan, advancing to full Professor in 1976. My research interests have been in the field of soil dynamics and earthquake engineering. I have done part-time consulting in the fields of soil dynamics, earthquake engineering, structural vibrations, and general foundation engineering. My clients have included Bechtel, Corning Glass Works, Rockwell International, Eaton Corporation, TAMS, General Motors, Honeywell Inc., Woodward-Clyde Consultants, and Nuclen (Nuclear Brazil). I have directed research associated with liquefaction phenomena sponsored by the National Science Foundation and have been a consultant to Bechtel, TAMS, Woodward-Clyde, and Nuclen on liquefaction issues. I am a principal in the foundation consulting firm of Stoll, Evans, Woods, and Associates, Ann Arbor, Michigan and am a member of ASCE, ASEE, ASTM, and SSA.

2.0 INTRODUCTION

My testimony is concerned with the evaluation of the potential for liquefaction of loose sands in the plant area at the Midland plant. The liquefaction potential was evaluated using the simplified method based on blowcount as presented by Seed. The maximum ground acceleration was taken as 0.19g and a Richter magnitude of 6.0 was used to correlate with about 5 cycles of significant stress reversal for the Midland site. On the basis of my analysis and the proposed remedial measures, I have concluded that there is reasonable assurance that the plant area is safe with respect to liquefaction of the sand.

3.0 DISCUSSION

When earthquake excitation is part of the design loads for a structure or facility, the potential for liquefaction of any saturated loose sands supporting the structure must be evaluated. Liquefaction is the phenomenon by which cohesionless soil loses shearing strength because of ground shaking and develops a degree of mobility sufficient to permit large permanent displacements or liquid-like flow behavior. Some common manifestations of liquefaction include settlement and tilting of structures, cracking and lateral spreading of slopes and embankments, flow type failures of natural slopes and embankments, and sand boils or sand volcanos.

Whether or not a specific sand formation will liquefy depends on several factors associated with the soil and the earthquake. The primary consideration is whether or not loose sands occur below the groundwater table (GWT). Unless the sands are saturated, there will be no buildup of excess pore pressure or loss of shearing strength associated with the ground shaking. However, if the sands are dense, they will not liquefy even if they are below the GWT. The measure of denseness used in the analysis of liquefaction potential is called relative density. Other factors that influence the potential for liquefaction include the effective confining pressure on the sand and the intensity and the duration of ground shaking. Large, effective confining pressures reduce the potential for liquefaction, whereas more intense and longer durations of shaking increase the potential for liquefaction.

Sands that must be evaluated for liquefaction potential exist in several locations at the Midland plant. Some areas are concentrated under or around Category I structures, whereas other areas are distributed and support embedded pipelines and duct banks. Several techniques are used to remedy the susceptibility of certain sands to liquefaction, depending on their locations and extent. These include preventing saturation of the sand by lowering the GWT and total removal and replacement of the sand with materials that are not subject to liquefaction.

4.0 EVALUATION OF LIQUEFACTION POTENTIAL

Based on the factors influencing the potential for liquefaction, Seed and Idriss (1971) and Seed (1979) proposed an empirical method for evaluating the liquefaction potential for sands at level ground sites. Their method is based on the performance of sand deposits having certain known characteristics in previous earthquakes and a comparison with sands of measured characteristics at the new site when subjected to a specified design earthquake. For any specified location in a sand deposit, a key factor called the cyclic stress ratio can be estimated and is based on site conditions and the specified maximum ground surface acceleration. The relative density of the sand (as indicated by standard blowcount) required to sustain a certain minimum number of cycles of that cyclic stress ratio without liquefaction can be estimated from the experience gained from previous earthquakes. If the in situ standard blowcount at the specified location meets or exceeds the estimated blowcount, no potential for liquefaction exists.

The computations required to perform this evaluation are as follows:

a. Estimate cyclic stress ratio (T_{av}/σ_o')

$$(T_{av}/\sigma_o') = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} \times r_d \qquad (1)$$

where

T_{av} = average horizontal shearing stress induced by earthquake

a_{max} = maximum horizontal acceleration at ground surface

σ_o = total overburden pressure on sand

σ_o' = initial effective overburden pressure on sand

r_d = stress reduction factor

g = acceleration of gravity

b. Estimate in situ blowcount required to preclude liquefaction.

Values of cyclic stress ratio have been correlated with a modified penetration resistance (N_1) at sites that have and have not liquefied during actual earthquakes. For earthquakes of a Richter magnitude of 6.0,* this correlation is shown in Figure L-1, where all points on and to the right of the curve are safe with respect to liquefaction. The modified penetration resistance is related to standard penetration resistance by:

$$N_1 = C_N N \qquad (2)$$

where

N_1 = modified penetration resistance

C_N = a function of effective overburden pressure and relative density as shown in Figure L-2 (use curve for D_r 40 to 60%)

N = standard penetration resistance

*This magnitude was selected to provide a close correlation, based on number of cycles, with the Midland SSE.

c. Compare N computed from Equation (2) with N in situ.

If the standard penetration resistance measured at a specific location in the ground is equal to or exceeds N computed from Equation (2), the sand at that location will not liquefy under the design excitation.

In the above method of evaluating the potential for a specific sand to liquefy, both the intensity of earthquake shaking and the duration of the earthquake are considered. The intensity is included in Equation (1) for cyclic stress ratio where a maximum ground acceleration of 0.19 g has been used and the number of cycles of significant stress is covered by selection of the curve in Figure L-1, in this case, the curve for an earthquake of a Richter magnitude of 6.0.

This method of liquefaction evaluation presumes that the sand at the specific location being examined is saturated. Therefore, one method of preventing liquefaction is to drain the sand by lowering the GWT. Initial computations showed that some strata or pockets of sand would be susceptible to liquefaction with the GWT at elevation 627 feet, but that by lowering the GWT to 610 feet or below, the potential for liquefaction could be eliminated.

5.0 RESULTS OF EVALUATIONS OF LIQUEFACTION POTENTIAL

Sands for which the potential for liquefaction had to be evaluated occur under portions of two Category I structures and at some other locations around the plant site where pipelines and duct banks are buried. The key parameter reflecting the condition of the sand as measured in situ at each location is the standard penetration resistance, N. N was measured at various elevations in borings throughout the plant site. The locations of all plant site borings including those used in this evaluation of liquefaction potential are shown in Figures L-3, L-4, and L-5.

The method by which the liquefaction potential is resolved for the various locations is described separately in the following paragraphs.

5.1 DIESEL GENERATOR BUILDING AREA

Liquefaction evaluation of sand in this area is based on the blowcount and relative density data obtained from various investigations. Bechtel test borings drilled in September and October 1978 (DG series) and November 1979 (CH series) provided blowcount information before and after placement of surcharge, respectively. Additional data on blowcount were obtained from the Woodward-Clyde Consultants relative density data (FSAR Appendix 2H). These data were obtained during the fill investigation and are based on the COE series borings performed around the diesel generator building in April 1981. The boring location plan of the diesel generator building area is presented in Figure L-4.

Studies of the liquefaction potential are illustrated by the blowcounts versus elevation plots presented in Figures L-6 through L-8. Each figure has two sets of curves representing two GWT elevations (610 and 627 feet) and two factors of safety (1.0 and 1.5). The left-side curves form an approximate boundary that separates liquefaction from no liquefaction zones (i.e., Fs = 1.0). The curve on the right represents a boundary of the no-liquefaction condition with a safety factor of 1.5.

The factor of safety as used here means that the cyclic stress ratio computed from Equation (1) was multiplied by 1.5, and then the standard penetration resitance required to satisfy the higher cyclic stress ratio was determined.

Liquefaction is not possible above the GWT, and with the GWT lowered to elevation 610 feet or lower, only two locations beneath the structure representing separate pockets of sand show blowcounts that are potentially liquefiable (Figure L-6). Because of the limited extent of these pockets, they should have no effect on the stability of the structure. Penetration resistance for all other locations representing the major portion of the volume of sand under the diesel generator building (Figures L-6 through L-8) indicates that the sands are safe with respect to liquefaction.

5.2 RAILROAD BAY AREA OF AUXILIARY BUILDING

Three of the Bechtel AX series borings represent soil conditions beneath the railroad bay of the auxiliary building (see Figure L-3). The liquefaction analysis of the sand in this area is presented in the blowcounts versus elevation plot in Figure L-9. The lower set of curves in this figure for factors of safety of 1.0 and 1.5 show that only one location beneath the building had a factor of safety less than 1.5, so liquefaction is not a problem when the GWT is maintained at elevation 610 feet or lower.

5.3 OTHER AREAS

Sands in the plant area outside the diesel generator building and the railroad bay area of the auxiliary building were analyzed for liquefaction potential by separately evaluating three horizontal strata: below elevation 605 feet, between elevations 605 and 610 feet, and above elevation 610 feet.

5.3.1 Plant Area Natural Sands Below Elevation 605 Feet

Sands existing below elevation 605 feet are primarily natural sands, although some fill sands were also placed in backfill around deep structures below elevation 605 feet. To evaluate the liquefaction potential of these sands, the standard penetration resistance in situ was compared with that required to prevent liquefaction, which was computed as described in Section 3.0 using a factor of safety of 1.5. This analysis showed that the sands in the plant area below elevation 605 feet have a few pockets with in situ blowcounts lower than required. The location of these pockets are identified in Figure L-10 with pertinent data from the analysis also shown in the figure. Table L-1 lists all borings in which low-blowcount sands were identified and shows the low-blowcount sands in relation to the other soils above and below.

Some of the low-blowcount pockets are not located near any Category I structure, pipeline, or duct bank. The remaining pockets represent single isolated blowcounts surrounded by soils with significantly higher blowcounts above and below or by nonliquefiable soils above and below (e.g., see boring CT-1, elevation 602.0 feet, Figure L-10, and Table L-1).

Based on this analysis, the natural sands below elevation 605 feet throughout the plant area present no hazard due to liquefaction.

5.3.2 Plant Area Fill Sand Between Elevations 605 and 610 Feet

Sands between elevations 605 and 610 feet are mainly fill sands, but relatively small, localized pockets of natural sands were also encountered in this elevation range. Sands in this stratum were analyzed in the manner described in Section 5.3.1. That analysis showed that scattered pockets of low-blowcount sand exist in the fill. The locations of borings in which these low-blowcount sand pockets were found are shown in Figure L-11, and Table L-2 lists those borings and contains pertinent data relative to the analysis and resolution of liquefaction potential in the low-blowcount sand pockets.

Some of these low-blowcount pockets are located such that they do not affect the stability of Category I structures; some are within zones that will be excavated and backfilled; the remaining are located between high-blowcount sands or other nonliquefiable soils.

Based on this analysis, the fill sands between elevations 605 and 610 feet do not constitute a liquefaction hazard.

5.3.3 <u>Plant Area Sand Between Elevations 610 and 627 Feet Outside of Both Diesel Generator Building and Railroad Bay of the Auxiliary Building</u>

Sands between elevations 610 and 627 feet are fill material. The susceptibility to liquefaction of any loose sands in this stratum depends on their location relative to the permanently dewatered regions as well as other factors.

The locations of borings in which pockets of low-blowcount sands have been identified are shown in Figure L-12. The low-blowcount sand pockets were analyzed for liquefaction potential in the manner described in Section 5.3.1. Table L-3 lists the borings shown in Figure L-12 and provides pertinent data relative to the analysis and resolution of liquefaction potential in low-blowcount pockets.

Two of the areas in this stratum where several pockets of low-blowcount sands occur were south of the diesel generator building and northeast of the railroad bay area. Both of these areas will be within the zone of dewatering and therefore not subject to liquefaction. Another area with pockets of low-blowcount sand occurs northwest of the service water pump structure and the circulating water intake structure. The zones where these sand pockets exist will be excavated to elevation 610 feet and replaced with suitable backfill. Other pockets are bounded by higher blowcount or nonliquefiable materials. Finally, some low-blowcount sand pockets are outside the area and do not influence the stability of structures.

6.0 SUMMARY AND CONCLUSIONS

Limited pockets of loose natural sand and loose fill sand exist in the plant area and under two Category I structures at the Midland plant. The potential for these sands to liquefy during an earthquake with a maximum ground acceleration of 0.19 g and Richter magnitude 6.0 has been evaluated.

For most of the sand pockets which exhibited a potential for liquefaction, remedies are provided which eliminate the potential by permanently lowering the GWT or by totally removing the loose sands and replacing them with suitable materials. For other sand pockets, liquefaction is not a hazard because they occur in location where they do not influence any Category I structures. The remaining pockets are situated in limited zones between other nonliquefiable soils and therefore present no hazard.

Because of the widely scattered occurrence of the loose sand pockets in the plant area, the potential for liquefaction was small before remedial measures were adopted; therefore, after the implementation of remedial measures, the plant area will be safe with respect to liquefaction of the sands.

7.0 REFERENCES

1. Seed, H.B. and I.M. Idriss (1971), "Simplified Procedure of Evaluating Soil Liquefaction Potential," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Volume 95, SM 9 (September), pp 1249-1272

2. Seed, H.B. (1979), "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Volume 105, No. GT2 (Feburary), pp 201-255

RICHARD D. WOODS, Ph.D., P.E.
Professor of Civil Engineering
University of Michigan

RÉSUMÉ

RICHARD D. WOODS, Ph.D., P.E.

Professor of Civil Engineering
University of Michigan

August, 1980

Home

700 Mt. Pleasant
Ann Arbor, MI 48103
(313) 769-4352

Office

2322 G. G. Brown Lab
University of Michigan
Ann Arbor, MI 48109
(313) 764-4303

PERSONAL DATA

Age:	45, born U.S. citizen
Physical:	Height 6'; weight 220 lb
Health:	Excellent
Military:	U.S. Marines
Married:	Wife, Dixie Lee (Davis)
	Daughter, Kathleen Ann, age 23
	Daughter, Cecilia Marie, age 15
	Daughter, Karen Teresa, age 12

EDUCATION

High School, J. W. Sexton, Lansing, Michigan, 1953
B.S. Civil Engineering, University of Notre Dame, 1957
M.S. Civil Engineering, University of Notre Dame, 1962
Introductory (non-degree) Course, ASEE-AEC Basic Institute in Nuclear Engineering, North Carolina State College, 1964
Ph.D. Civil Engineering, University of Michigan, 1967

ORGANIZATIONS

American Society of Civil Engineers
American Society for Testing and Materials
American Society for Engineering Education
Chi Epsilon
Society of the Sigma Xi
Seismological Society of America

AWARD

Collingwood Prize of American Society of Civil Engineers, 1969

EMPLOYMENT (Full Time)

1976 to Present	Professor, Civil Engineering, University of Michigan. Courses taught: Basic Soil Mechanics, Field Sampling and Laboratory Testing of Soils, Foundation Engineering, Soil Dynamics, Civil Engineering Dynamics Measurements, Plane Surveying, Statics and Strength of Materials, Reinforced Concrete. Research performed: See separate paragraph below.
1971 to 1976	Associate Professor, Civil Engineering, University of Michigan. Courses taught: Included above.
1967 to 1971	Assistant Professor, Civil Engineering, University of Michigan. Courses taught: Included above.
1965 to 1967	Graduate Student, University of Michigan, supported on NSF Traineeship.
1964	Instructor, Civil Engineering, Michigan Technological University, Houghton, Michigan. Courses taught: Included above.
1963	Project Engineer (GS-11), Air Force Weapons Laboratory, Kirtland, AFB, Albuquerque, N.M. Supervised contracts which were directed at determining engineering properties of soils under dynamic loads.
1960 to 1962	Graduate Student, University of Notre Dame, teaching assistantship, taught surveying camp.
1957 to 1960	Lieutenant, U.S. Marine Corps, Camp Pendleton, California. Six months as platoon leader, movable bridge company. Remainder of service as hydraulic engineering officer preparing evidence for water rights litigation.

EMPLOYMENT (Short Courses and Special Appointments)

1976 Fugro Fellow, University of Florida. On sabbatical leave from University of Michigan. Investigating use of static cone penetrometer with built-in pore pressure transducer to predict liquifaction potential of sands.

1974 Invited Author for Chapter on Soil Dynamics for U.S. Army Corps of Engineers Soils Manual, with F. E. Richart.

1973 Invited Lecturer, Woodward-Clyde Consultants Symposium, Berkeley. Topic: "Seismic Methods to Measure Shear Wave Velocity of Soils and Rock."

1973
1972 Taught Extension Courses (evening), "Applications of Soil Mechanics to Foundation Engineering," 2-10 week lecture series for Commonwealth Associates, Jackson, Michigan.

1972 Visiting Professor, Institute for Soil and Rock Mechanics, University of Karlsruhe, Germany. Taught Soil Dynamics and helped establish soil dynamics laboratory. Research on propagation of Rayleigh Waves in region of obstacles.

1971 Visiting Professor, Indian Institute of Technology, Kanpur, India. Helped establish basic soil dynamics laboratory and field measurements capability.

1971 Invited Lecturer, Earthquake Engineering Seminar, University of Massachusetts, sponsored by National Science Foundation. Lectures on basic vibrations, wave propagation and dynamic soil properties.

1970
1969 Chairman and Principal Lecturer, two 2-day short courses, "Behavior of Soils for the Construction Industry, Continuing Engineering Education Program, College of Engineering, University of Michigan.

1968 Co-Chairman and Lecturer, Two-week short course, "Vibration of Soils and Foundations," Continuing Engineering Education program, College of Engineering, University of Michigan. Lectures on basic vibrations, wave propagation and field and laboratory measurements.

RESEARCH

At University of Michigan

Holographic Interferometry - Investigation of basic wave propagation and surface wave propagation in region of barriers.

Response of Pile Foundations to Dynamic Loads - with F. E. Richart.

Dynamic Properties of Soils - Laboratory and field measurement of compression and shear wave velocity and shear modulus of soils at both low and high amplitudes.

Isolation of Earthwaves by Barriers - Study of effectiveness of trenches and cylindrical holes at screening waves.

Dutch Static Cone Penetrometer - Study of use of penetrometer for identification of soils.

At Michigan Technological University

Mechanics of Slide Dams - Investigation of creation of dams by blasting material from canyon walls.

At Notre Dame University

Preliminary Design of Dynamic Direct Shear Device

CONSULTING EXPERIENCE

Areas of Consulting

Vibration Measurements - on machines, in soil, on structures

Measurement of Dynamic Soil Properties, in lab and in field

Stability of Soil Masses (Reserve Mining tailings delta)

Analysis and Design of foundations for dynamic loads

Site Investigations with Dutch, cone penetrometer

Blasting Damage Evaluations

Blasting Code Drafting

Seismic Site Investigations

Principal Clients

Bechtel Power Corporation, Ann Arbor, Michigan

Attorney General, State of Michigan (Reserve Mining Case)

CONSULTING EXPERIENCE--*Continued*

Giffels and Associates, Detroit, Michigan

Smith, Hinchman and Grylls, Detroit, Michigan

City of Rockwood, Michigan

City of Ann Arbor, Michigan

Honeywell Corporation, Minneapolis, Minnesota

Woodward-Clyde Consultants, Orange, California, Oakland, California and Philadelphia, Pennsylvania

Halpert, Neyer Associates, Farmington, Michigan

U. W. Stoll and Associates, Ann Arbor, Michigan

Eaton Brake Division, Detroit, Michigan

Tippetts-Abbett-McCarthy-Stratton, New York (Tarbela Dam)

Site Engineers, Inc., Cherry Hill and Montclair, New Jersey

Corning Glass Works, Corning, N.Y. and three other plants

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Woods, R. D. (1977), " Lumped Parameter Models for Dynamics Footing Response," Karlsruhe (as above).

Woods, R. D. (1977), "Holographic Interferometry to Study Seismic Wave Isolation," Karlsruhe (as above).

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Partos, A., Woods, R.D. and Welsh, J. (1982), "Soil Modification for Relocating Die Forging Operation," International Symposium on Grouting in Geotechnical Engineering, New Orleans, Feb.

Richart, F.E. Jr., and Woods, R.D. (1982), "Foundations for Auto Shredders," Proceedings of International Conference on Soil Dynamics and Earthquake Engineering, Southampton England, July 13-15, Vol. 2, pp.811-824.

TABLE L-1[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT

AREA SANDS BELOW ELEVATION 605 FEET

Boring Number[3]	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
AX-13	635.0	595.5	25	-	Sandy clay	High blowcount above and clay below
		593.0	42	-		
		590.5	10	25		
		588.0	17	-	Silty clay	
		585.5	145	-		
CT-1	634.0	612.0	23	-	Silty clay	High blowcount below and clay above
		607.5	7	-	Silty clay	
		602.0	11	21		
		599.0	24	-		
		597.0	29	-		
DF-5	634.0	606.5	28	-	Silty clay	Clay above and below
		604.0	17	-	Silty clay	
		601.5	8	21		
		599.0	8	-	Sandy clay	
		596.5	10	-	Sandy clay	
DG-7	631.0	602.0	25	-		High blowcount above and clay below
		600.5	17[5]	-		
		599.0	10	21		
		597.5	15	-	Silty clay	
		588.5	43	-	Silty clay	
DG-28	629.0	605.5	16	-		Clay above and high blowcount below
		603.0	15	-	Sandy clay	
		600.5	9	21		
		598.0	37	-		
		595.5	89	-		
Q-12	634.0	607.5	5	-	Silty clay	Not near a structure
		605.0	7	-	Silty clay	
		602.5	13	22		
		600.0	11	23		
		597.5	29	-		
		595.0	75	-		
PD-5B	634.0	605.0	15	-	Silty clay	Clay above and below
		602.5	7	-	Silty clay	
		600.0	4	21		
		597.5	15	-	Silt	
		595.0	27	-	Silty clay	

TABLE L-1 (continued)

Boring Number[3]	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	SPT Information: Blowcounts: In-situ	SPT Information: Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
PD-20	634.0	608.5	25	-		Not near a structure
		606.0	19	-		
		603.5	16	22		
		601.0	13	22		
		598.5	52	-		
		596.0	63	-		
PD-20A	634.0	609.0	40	-		Not near a structure
		606.5	23	-		
		604.0	8	21		
		601.5	14	22		
		599.0	50	-		
		596.5	130	-		
PD-20C	634.0	607.0	47	-		Not near a structure
		604.5	30	-		
		602.0	8	22		
		599.5	24	-		
		597.0	63	-		
LOW-9	634.5	605.0	20	-	Silty clay	Clay above and below
		603.0	27	-	Silty clay	
		601.0	9	21		
		599.0	24	-		
		597.0	21	-	Silty clay	
L_N	622.0	595.0	19	-	Sandy clay	Clay above and high blowcount below
		590.5	10	-	Sandy clay	
		586.0	20	22		
		584.5	100+	-		
		582.5	100+	-		

(1) This table excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
(2) Standard penetration test
(3) Boring location shown in Figures L-3, L-4, and L-5
(4) Ground surface elevation
(5) Nonstandard spoon used

TABLE L-2[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT AREA FILL
BETWEEN ELEVATIONS 605 AND 610 FEET

Boring[3] Number	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	SPT Information: Blowcounts: In-situ	SPT Information: Blowcounts: Required for M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
CH-5A	633.8	612.3	6	-		Within excavation zone
		607.3	17	21		
		602.3	30	-	Silty clay	
		597.3	85	-		
PD-20	634.0	611.0	45	-		Not near a structure
		608.5	25	-		
		606.0	19	21		
		603.5	16	-		
		601.0	13	-		
Q-9	634.0	610.5	34	-		Clay below and high blowcount above
		609.0	27	-		
		606.5	11	19		
		604.0	23	-	Sandy clay	
		601.5	82	-		
SW-2	634.0	617.0	36	-		Outside service water pump structure; does not affect stability of the structure
		612.5	10	-		
		607.5	11	18		
W-4	633.0	619.0	9	-		Outside service water pump structure; does not affect stability of the structure
		613.0	5	-	Sandy clay	
		609.0	12	17		
		606.5	23	-	Sandy clay	
		603.0	24	-	Sandy clay	

Table L-2 (continued)

Boring[3] Number	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	SPT Information: Blowcounts: In-situ	SPT Information: Blowcounts: Required for M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
DG-28	629.0	610.5	15	-		Outside diesel generator building
		608.0	33	-		
		605.5	16	19		
		603.0	15	-	Sandy clay	
		600.5	9	-		
DG-29	630.0	618.5	64	-		Outside diesel generator building
		614.5	93	-		
		610.0	5	17		
		605.5	10	-	Sandy clay	
		601.5	26	-		

[1] This table excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
[2] Standard penetration test
[3] Boring location shown in Figures L-3, L-4, and L-5
[4] Ground surface elevation

TABLE L-3[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT AREA FILL

BETWEEN ELEVATIONS 610 AND 627 FEET

		SPT Information				
			Blowcounts			
Boring[3] Number	GSE[4] At Time of Drilling (feet)	Sample Elevation (feet)	In-situ	Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
DF-1	633.0	628.0	30	-	Sandy clay	Zone of 3 foot sand fill layer with clay above and below
		623.0	10	11		
		621.5	3	12		
		620.0	12	-	Sandy clay	
		618.5	10	-	Sandy clay	
DF-2	634.0	629.0	47	-		This area has been excavated and later backfilled with sand. The tank foundation is resting on sandy clay with high blowcounts. These low blowcounts in sand occur around but not under tanks and do not affect tank stability.
		624.0	10	-	Sandy clay	
		622.5	3	12		
		621.0	8	13		
		619.5	11	14		
		618.0	16	-		
		616.5	9	16		
		615.0	13	17		
		612.5	6	-	Sandy clay	
		608.0	38	-	Sandy clay	
PD-19	634.0	630.0	9	-	—	Not near a structure
		627.5	4	-		
		623.5	3	12		
		620.0	21	-		
		617.5	23	-		
PD-20	634.0	631.5	7	-	Silty clay	Not near a structure
		629.0	6	-		
		626.5	7	9		
		624.0	16	-	Sandy clay	
		621.0	8	13		
		618.5	11	-	Clayey silt	
		616.0	3	-	Clayey silt	
		613.5	14	18		
		611.0	45	-		
		608.5	25	-		

TABLE L-3 (continued)

Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
PD-20A	634.0	630.0	9	-	Silty clay	Not near a structure
		627.5	3	-		
		625.5	5	10		
		622.5	9	12		
		620.0	11	14		
		617.5	3	16		
		614.0	11	-	Clay & sand	
		611.5	24	-		
PD-20C	634.0	631.5	19	-		Not near a structure
		629.0	4	-		
		626.5	7	9		
		622.0	7	13		
		619.5	31	-		
		617.0	37	-		
SWL-1	634.0	616.0	14	-	Sandy clay	Zone of 2.5 foot sand fill layer with clay above and below
		613.5	9	-	Sandy clay	
		611.0	13	19		
		608.5	4	-	Sandy clay	
		606.0	29	-	Sandy clay	
PD-13	634.0	630.0	5	-		Above maximum ground water table
		627.5	1	-		Silty clay below
		625.0	6	11		
		622.5	5	-	Silty clay	
		620.0	10	-	Silty clay	
Q-9	634.0	629.0	5	-	Sandy clay	Within excavation zone
		624.0	9	-	Sandy clay	
		617.5	7	14		
		615.5	13	15		
		614.0	7	16		
		610.5	34	-		
		609.0	27	-		
SWL-8	634.0	630.0	6	-	Silty clay	Within dewatering zone
		627.5	5	-	Silty clay	
		625.0	4	11		
		622.5	16	-		
		620.0	7	14		
SWL-8A	634.0	622.5	2	12		Within dewatering zone
		620.0	9	14		
		617.5	7	16		

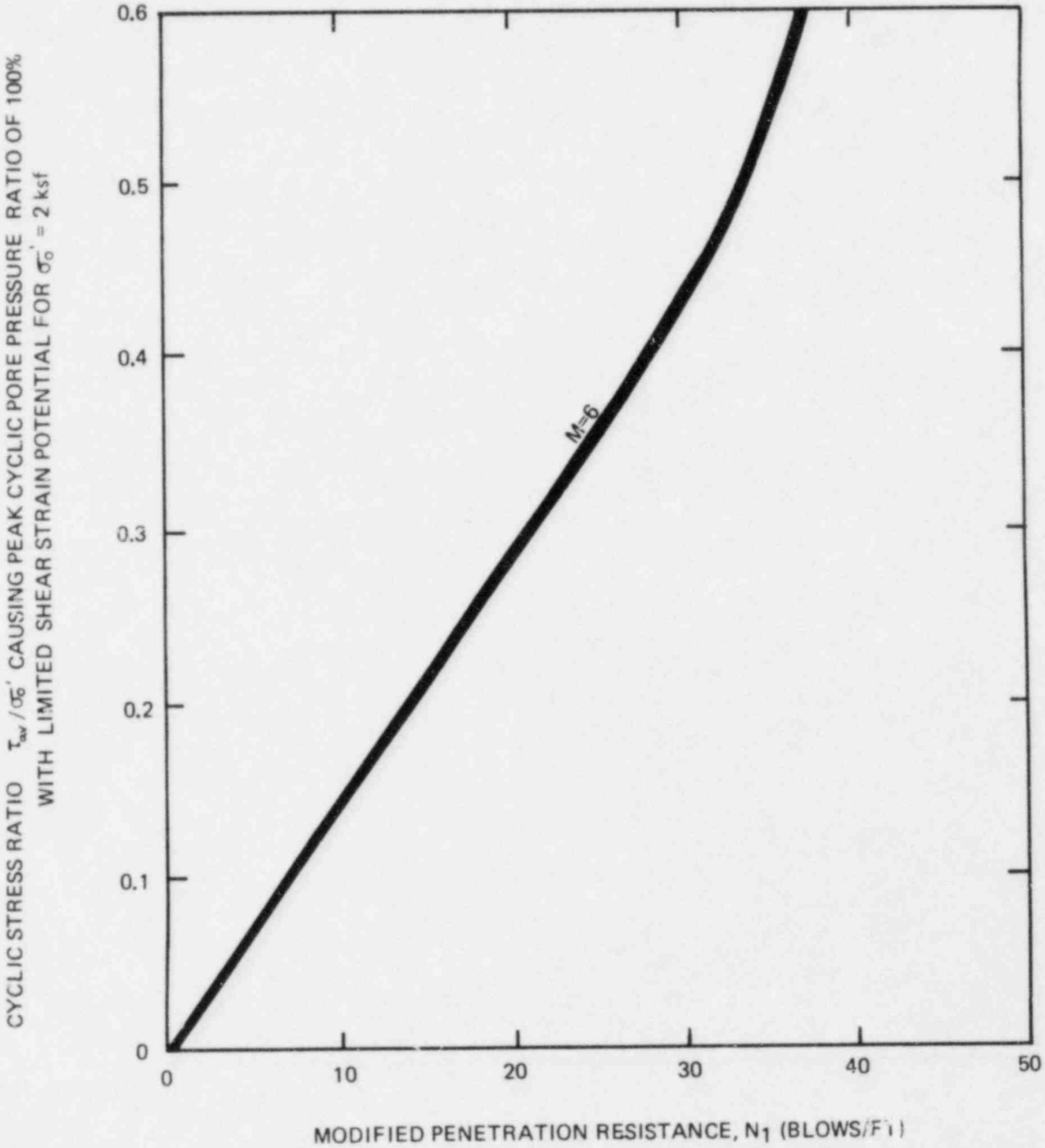
TABLE L-3 (continued)

Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information Sample Elevation (feet)	Blowcounts In-situ	Blowcounts Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
SWL-6	634.0	617.5	8	-	Silty clay	Zone of 2 foot sand fill layer with clay fill above and below
		615.0	14	-	Silty clay	
		612.5	15	18		
		610.0	33	-	Silty clay	
		607.5	12	-	Silty clay	
SW-7	635.0	626.0	21	-		Within excavation zone
		623.5	24	-		
		621.0	12	14		
		618.5	9	16		
		616.0	19	-		
		613.5	11	-	Silty clay	
CH-2	633.8	622.3	4	12		Within excavation zone
		617.3	4	16		
		612.3	13	-	Silty clay	
		607.3	11	-	Silty clay	
CH-4	634.6	623.1	4	12		Within excavation zone
		618.1	45	-		
		613.1	17	18		
		608.1	24	-		
		603.1	33	-	Sandy clay	
CH-5	633.8	622.3	20	-		Within excavation zone
		617.3	38	-		
		612.3	9	18		
CH-6	634.0	622.5	17	-		Within excavation zone
		617.5	5	16		
		612.5	6	18		
PD-27	634.0	625.0	31	-		Within excavation zone
		622.5	8	-		
		620.0	4	13		
		617.5	16	-		
		615.0	33	-		
SW-2	634.0	621.5	51	-		Outside the service water pump structure and does not affect the stability of the structure
		617.0	36	-		
		612.5	10	16		
		607.5	11	-		

TABLE L-3 (continued)

Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
SW-5	634.5	625.5	28	-		Outside the service water pump structure and does not affect the stability of the structure
		623.0	6	-	Silty clay	
		620.5	3	14		
		618.0	6	16		
		615.5	11	17		
		613.0	16	-	Silty clay	
		610.5	35	-		
DW-1	634.0	617.5	9	-	Sandy gravel	Excavated and backfilled during duct bank repair
		612.5	16	18		
		610.0	30	-	Silty clay	
DW-2	634.0	612.5	13	18		Isolated in clay fill
		609.5	31	-	Silty clay	

[1]This table excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
[2]Standard penetration test
[3]Boring location shown in Figures L-3, L-4, and L-5
[4]Ground surface elevation



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MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION-CYCLIC STRESS RATIO VS "MODIFIED" PENETRATION RESISTANCE FOR EARTHQUAKE MAGNITUDE OF 6, AFTER SEED (2)		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-1	0

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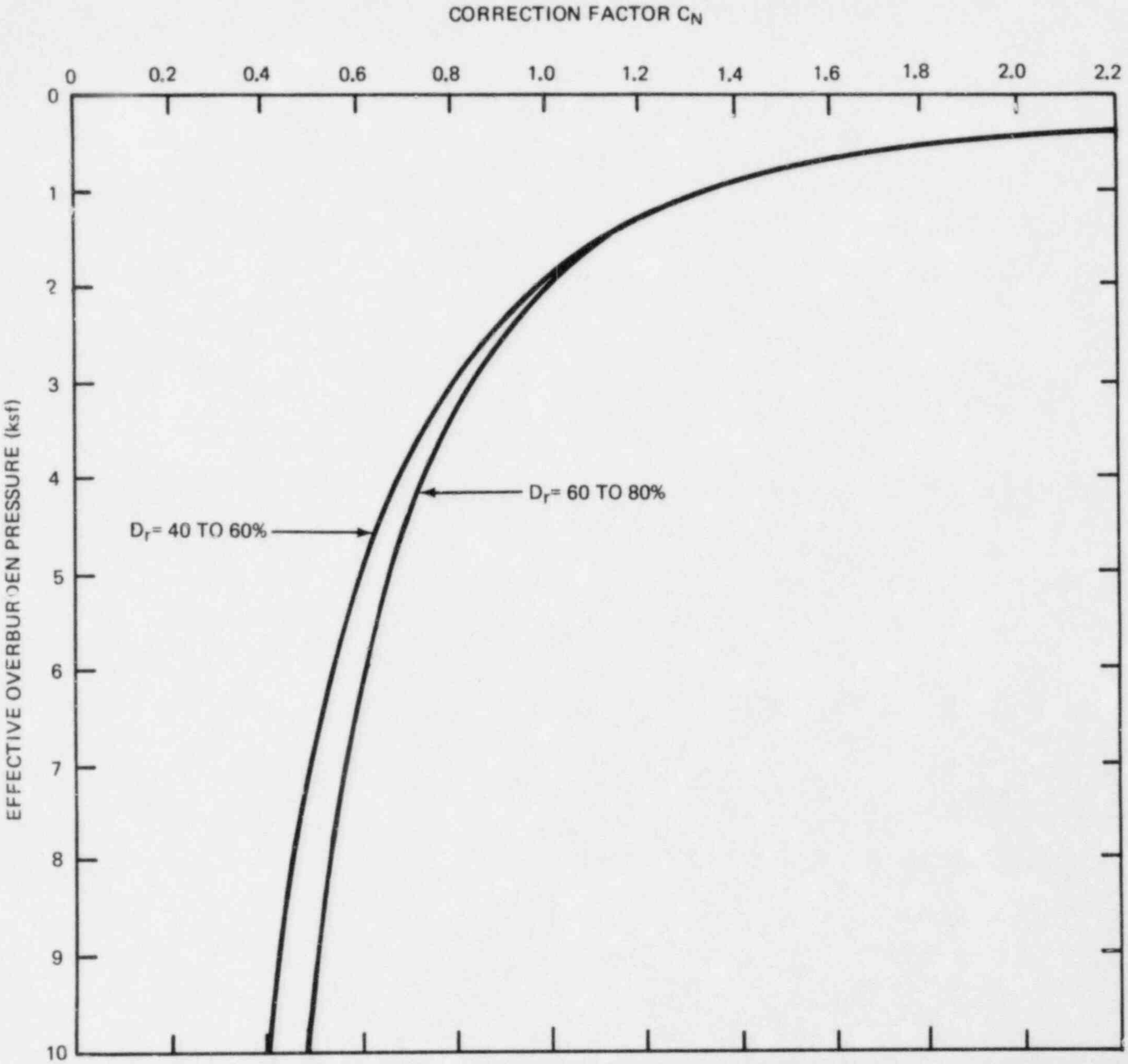

EXPLANATION

D_r – RELATIVE DENSITY

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION-CORRECTION FACTOR FOR BLOWCOUNT AS A FUNCTION OF OVERBURDEN PRESSURE, AFTER SEED (2)		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-2	0

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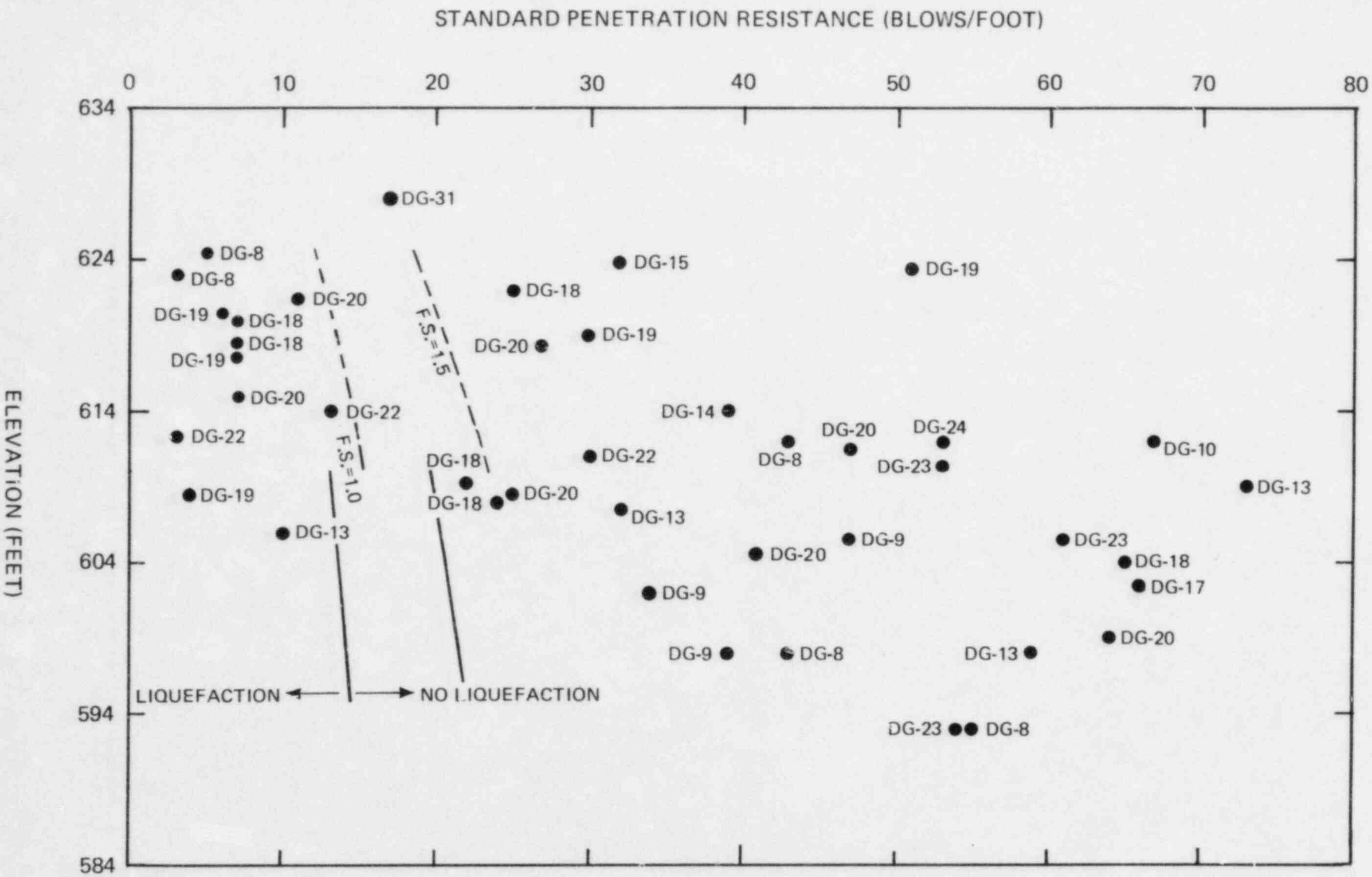

<u>EXPLANATION</u>

— — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

GWT – GROUND WATER TABLE

<u>NOTES:</u>

1. BLOWCOUNTS WERE CORRECTED TO ACCOUNT FOR ADDED SURCHARGE DUE TO THE BUILDING LOAD AND LOWERED WATER TABLE.
2. BORINGS PRESENTED ARE LOCATED WITHIN THE DIESEL GENERATOR BUILDING.

SK-G-702

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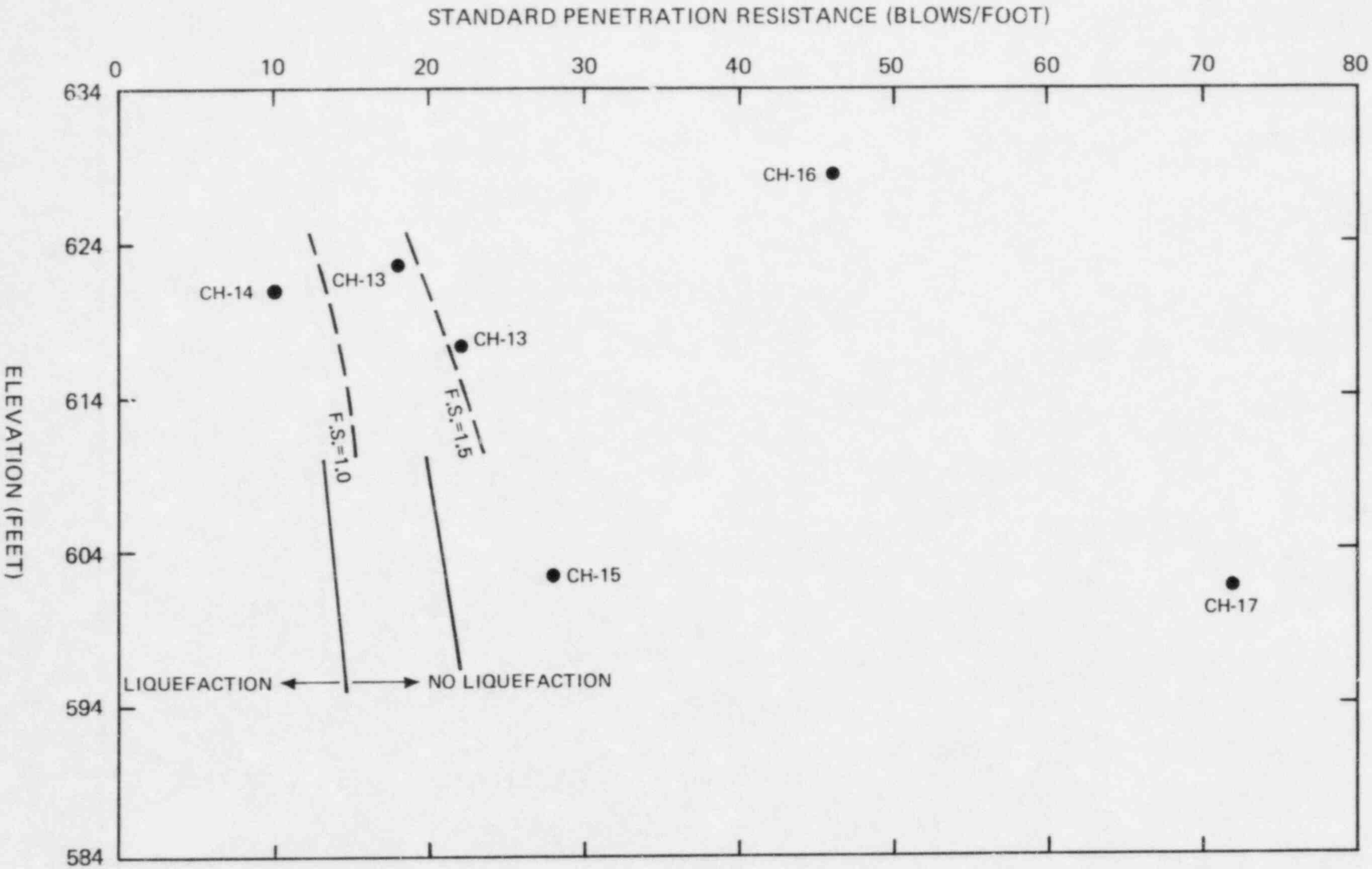
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1978 "DG" BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L 6	2



EXPLANATION

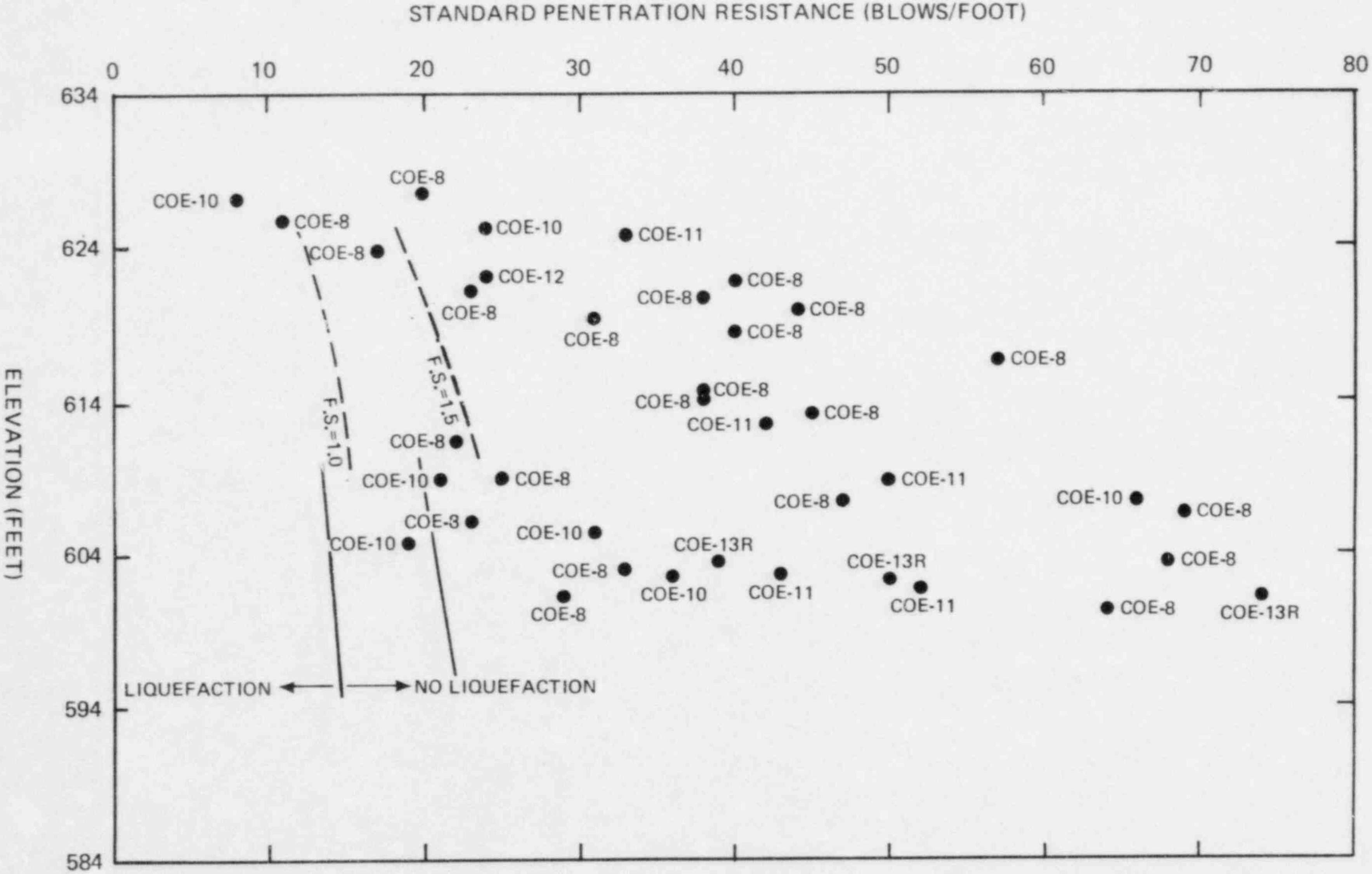
— — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

GWT – GROUND WATER TABLE

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1979 "CH" BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE 5-7	0

SK-G-703



EXPLANATION

— — — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———————— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

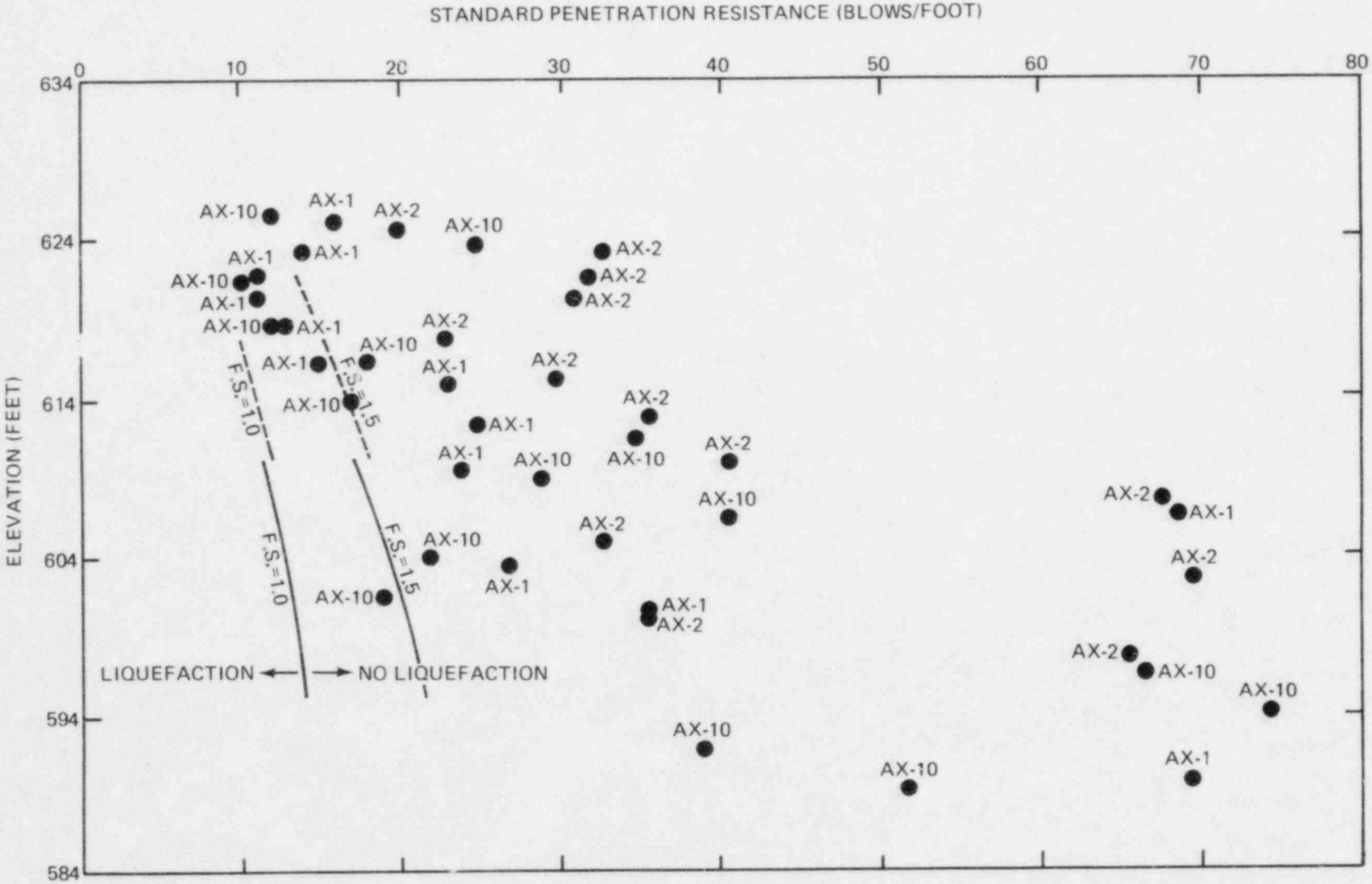
GWT — GROUND WATER TABLE

NOTE :

BLOWCOUNTS WERE CONVERTED FROM THE RELATIVE DENSITY VALUES OBTAINED FROM WOODWARD–CLYDE CONSULTANTS TEST DATA

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1981 "COE" SERIES BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-8	1

SK-G-704



EXPLANATION

----- BOUNDARY OF LIQUEFACTION, GWT AT 622.0'

—— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

GWT – GROUND WATER TABLE

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1979 BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR THE RAILROAD BAY AREA OF THE AUXILIARY BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-9	0

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