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LIQUEFACTION POTENTIAL ANALYSIS OF BISCO LAKE FOUNDATION AND EMBANKMENT

RIO ALGOM MINING CORP.
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Moab, Utah

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LIQUEFACTION POTENTIAL ANALYSIS OF BISCO LAKE FOUNDATION AND EMBANKMENT

1.0 INTRODUCTION

This report presents the results of a geotechnical study of the Bisco Lake foundation and embankment of the Lisbon Uranium Mill for the Rio Algom Mining Corp. In accordance with requests from the Nuclear Regulatory Commission, this study involves a review of field investigations and laboratory analyses and includes geotechnical calculations to determine the potential of the Bisco Lake foundation and embankment to liquefy during seismic loading. The liquefaction analyses is based on a 1,000-year seismic risk analysis.

This report has been divided into 6 sections including this introduction. A site description is presented in Section 2 followed by the methods of investigation in Section 3. The liquefaction analysis is presented in Section 4. Section 5 contains the conclusions followed by the references in Section 6.

The field investigations and laboratory analyses used for this report were performed by others for the Rio Algom Mining Corp. and will be referenced as such.

2.0 SITE DESCRIPTION

2.1 General Seismicity

The Rio Algom site is located in the Paradox fold and fault belt of the Colorado Plateau, which is generally considered to be a region of low seismicity. The historic record of earthquakes of Magnitude 4.0 or greater (Richter scale) near the study area is presented in Figure 2-1. (modified from Dames & Moore, January, 1980). This data indicate that the historic earthquake nearest to the Rio Algom site was of Magnitude 4 at a hypocentral distance of approximately 53 miles northeast of the site. Dames & Moore (January, 1980) performed a seismic risk analysis of the study area, concluding that the peak horizontal acceleration which will occur at the Rio Algom site in a 1000-year interval is 9 percent of gravitational acceleration (g). This postulated acceleration is equivalent to a Magnitude 6 event with a hypocentral distance of between 25 and 31 miles.

In accordance with the seismic risk analysis performed by Dames & Moore, a peak horizontal acceleration of 0.09g will be used for the liquefaction analysis of the Bisco Lake foundation and embankment at the Rio Algom site.

2.2 Bisco Lake Embankment

As shown in Figure 2-2 (modified from Dames & Moore, 1982), the Bisco Lake embankment was constructed in the following 3 stages:

1. Barium chloride pond embankment constructed of nonengineered earth fill utilizing native soil to elevation 6716.
2. Initial embankment constructed of nonengineered earth fill utilizing native soils to elevation 6733. Native

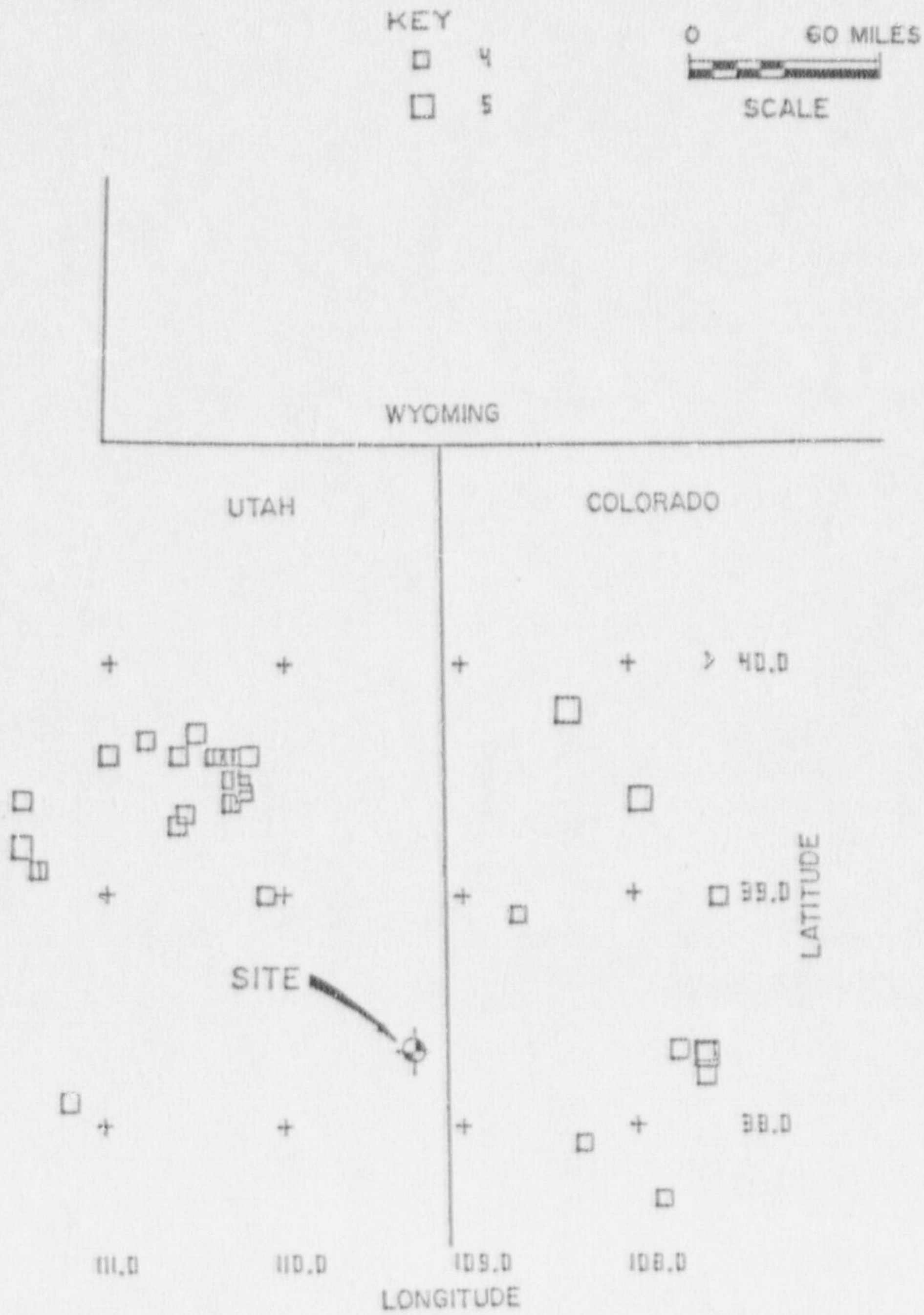

FIGURE 2-1. RECORDED EPICENTERS - MAGNITUDE OF 4.0 OR GREATER

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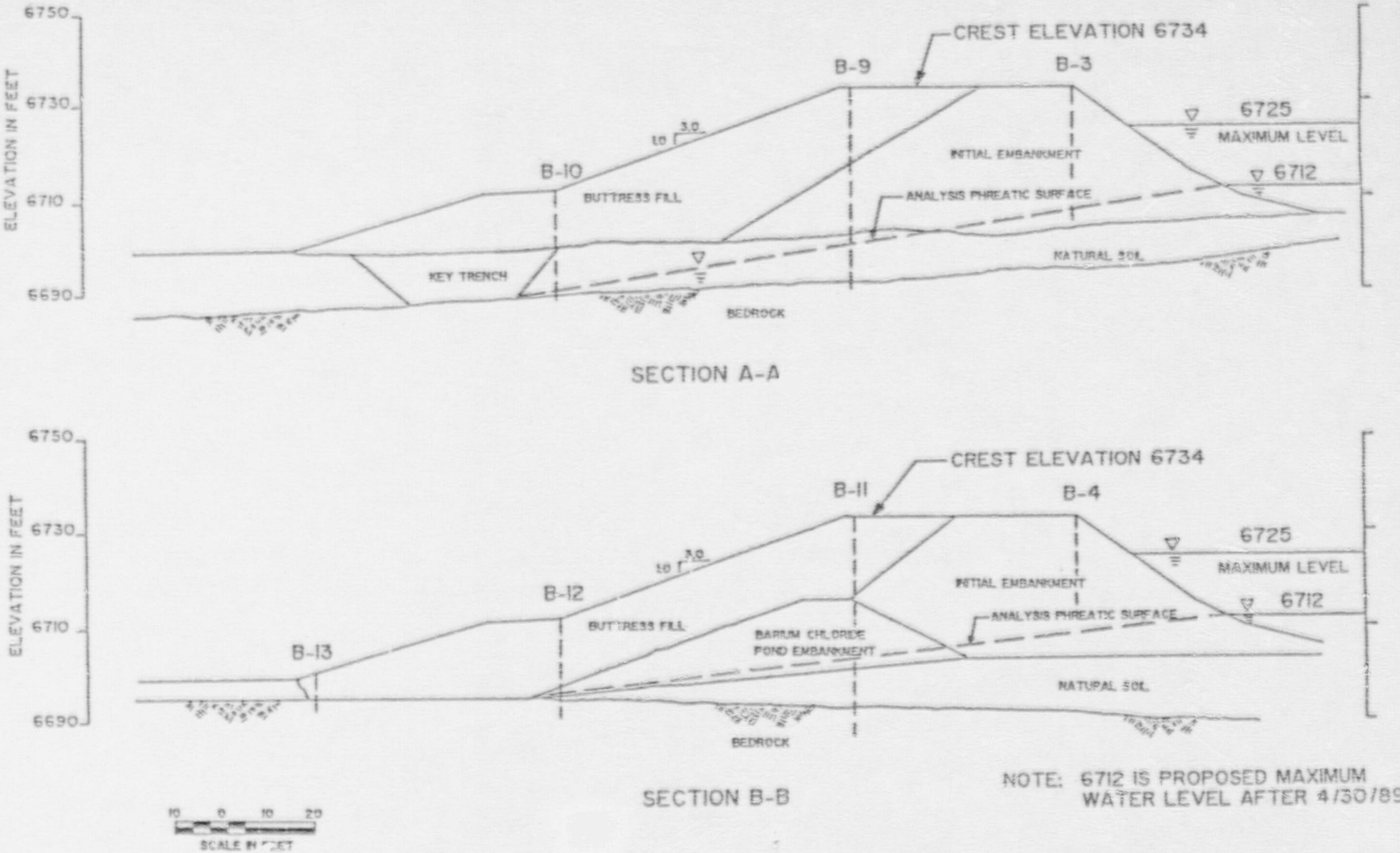

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FIGURE 2-2. BISCO LAKE EMBANKMENT.

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soils consisted of reddish brown clayey silts with some fine-grained sand (Dames & Moore, 1982).

3. Buttress fill constructed to elevation 6734 for a maximum embankment height of approximately 33 feet.

The buttress fill material was placed as a structural fill and consists of reddish brown silt and clay with some fine to coarse sand and fine gravel (Dames & Moore, 1982). The fill was placed to 90% of the maximum dry density as determined by the AASHTO T-180 method of compaction. The buttress fill includes a key trench and blanket drain consisting of free draining material.

The Rio Algom Mining Corp. has proposed that the Bisco Lake embankment be used to temporarily contain surface water runoff. The probability of the peak maximum flood corresponding with the peak horizontal acceleration is low.

2.3 Bisco Lake Foundation

The native soils which underlie the Bisco Lake embankment consist of medium stiff reddish brown silty clay with some fine to coarse sand (Dames & Moore, 1982). The native soil ranges between 0 and 20 feet thick.

3.0 METHODS OF INVESTIGATION

Geotechnical information pertaining to the subsurface conditions within and beneath the Bisco Lake embankment was provided by 11 exploration borings emplaced by Dames & Moore (1982). The exploration borings are presented in Appendix A and designated as B-3 through B-13 in Figure 3-1.

Field investigations conducted by Dames & Moore included Standard Penetration tests, Unified Soil classifications, and water level measurements. Laboratory analyses included in-situ moisture and dry density. Results of the field investigations and laboratory analyses are included in the boring logs in Appendix A.

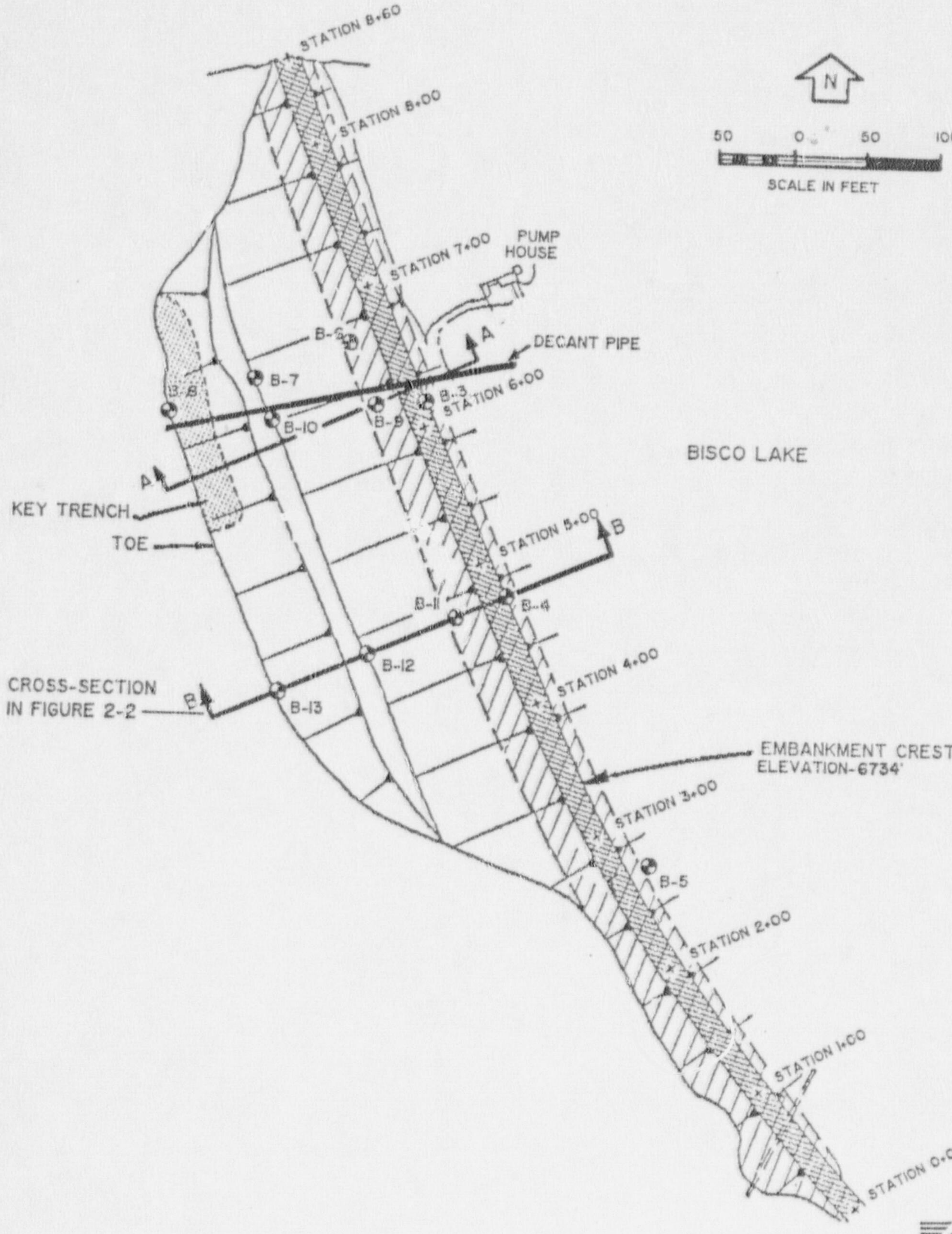

FIGURE 3-1. LOCATION OF SUBSURFACE EXPLORATION BORINGS.

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4.0 LIQUEFACTION ANALYSIS

4.1 Liquefaction Analysis Method

The liquefaction potential analysis was performed by a method proposed by Seed and Idriss (1982). This method involves correlating known values of cyclic shear stress induced in sand deposits under actual earthquake shaking conditions with some readily measurable in-situ characteristics of soils.

The average shear stress (τ_h) induced by an earthquake on horizontal sand deposits can be expressed approximately by the equation:

$$\tau_h \approx 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_o \cdot r_d$$

where a_{max} = estimated peak horizontal acceleration at the site (0.09g)
g = gravitational acceleration
σ_o = total stress on the soil element (psf)
r_d = stress reduction coefficient with a value less than 1

The stress reduction coefficient (r_d) is the ratio of the actual shear stress induced on a soil element by an earthquake event to the maximum shear stress induced on the soil element if it behaved as a rigid body. Seed and Idriss (1982) provide a range of values for r_d as shown in Figure 4-1. In the upper 30 or 40 feet of the soil profile in Figure 4-1, the scatter of the results is not great and the error involved in using the average values would generally be less than about 5%. This depth range is normally the range over which liquefaction will occur.

It is useful to normalize the above equation by dividing both sides of the equation by the effective stress on the soil element to arrive at the following equation:

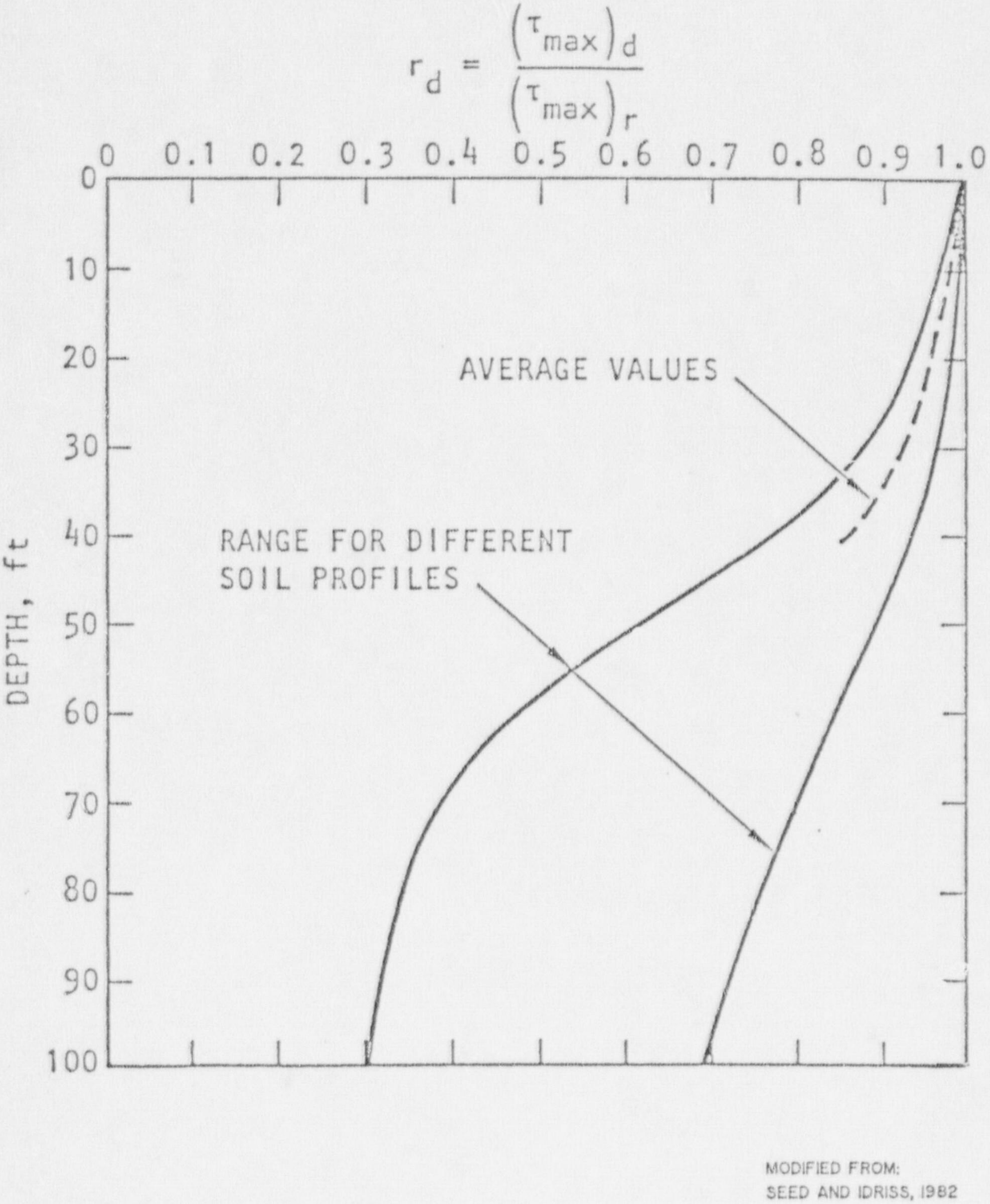

FIGURE 4-1. RANGE OF VALUES OF r_d

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$$\frac{\tau_h}{\sigma_o'} \approx 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d \qquad \text{(Equation 1)}$$

where $\frac{\tau_h}{\sigma_o'}$ = cyclic stress ratio

σ_o' = effective stress on soil element

A comparison of field and laboratory results indicates that the cyclic response of soils and the Standard Penetration Resistance test are affected by the same factors in the same general way. Thus, it is convenient to use the penetration resistance of a soil deposit as an index of liquefaction resistance. In order to use the penetration resistance as an index value, it is necessary to normalize the Standard Penetration Resistance value for a single effective overburden pressure. Seed and Idriss use a normalization pressure of 2000 psf. The Normalized Penetration Resistance for any sand deposit can be determined from the equation:

$$N_1 = C_n \times N \qquad \text{(Equation 2)}$$

where N_1 = Normalized Penetration Resistance
N = Standard Penetration Resistance measured in the field
C_n = Normalization coefficient

Values for C_n are presented in Figure 4-2 (Seed and Idriss, 1982).

Utilizing field data and Equations 1 and 2, Seed and Idriss developed a relationship presented in Figure 4-3 between the Standard Penetration Resistance (normalized) and the cyclic stress ratio for various magnitude earthquake events. Figure 4-3 is a[illegible]ble for sand deposits only (mean particle size greater than 0.25 mm). Since the Bisco Lake foundation and embankment soils are predominantly silts and clays with some fine grained sands, Figure 4-3 cannot be used directly. However, Seed and Idriss have

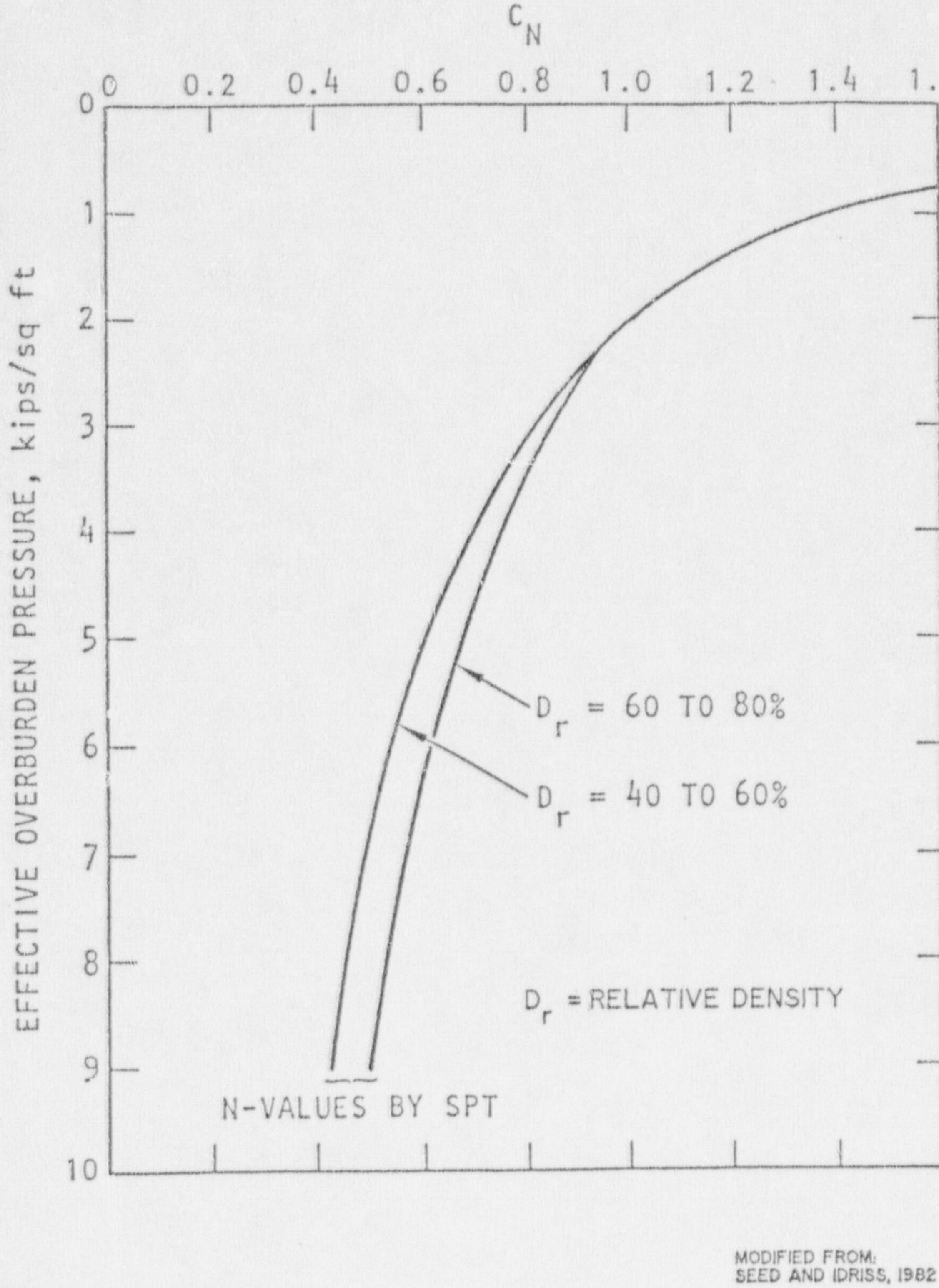

FIGURE 4-2. Cn VALUES.

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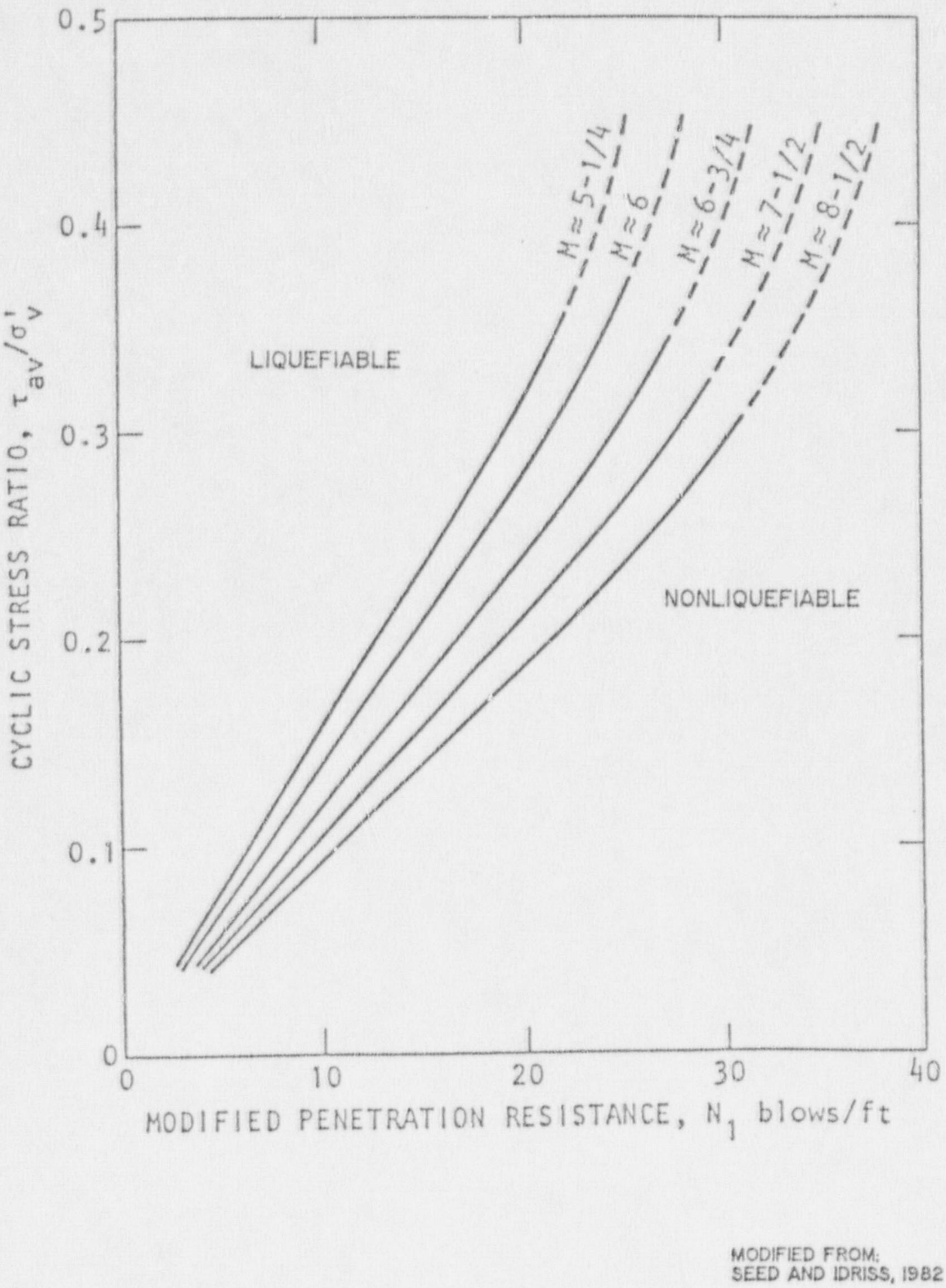

FIGURE 4-3. LIQUEFACTION POTENTIAL OF SANDS FOR VARIOUS MAGNITUDE EARTHQUAKES.

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extended their investigations to include silty-sandy soils. In general, Seed and Idriss conclude that, "silty-sands are considerably less vulnerable to liquefaction than sands with similar penetration resistance values." In a procedure similar to the one discussed above, Seed and Idriss developed the curves presented in Figure 4-4 from observations following earthquake events for silty sand deposits.

From Figure 4-4, it can be seen that the lower boundary established for sands is approximately parallel to that established for silty sands. The two curves are offset from one another by an average modified penetration resistance value of 7.5 for a constant cyclic stress ratio. Seed and Idriss conclude that Figure 4-3 can be used for silty-sandy soils by adding 7.5 to the modified penetration resistance value of the soil before entering the curve.

In general, sandy silts are less vulnerable to liquefaction than are silty sands. Hence, the sandy silt native materials at the Rio Algom site are less susceptible to liquefaction than indicated by the curves in Figures 4-3 and 4-4. Thus, the results of the analysis will be conservative.

Both laboratory tests and field performance data of clayey soils indicate that clayey soils generally do not liquefy during earthquakes. Therefore, clayey soils may be considered non-vulnerable to liquefaction.

The liquefaction potential of a sand or a silty sand deposit can be evaluated by the following procedure:

1. Estimate the peak horizontal acceleration and earthquake magnitude at the site (0.09g and Magnitude 6, respectively).

2. At the depth in question, calculate the total and effective stress.

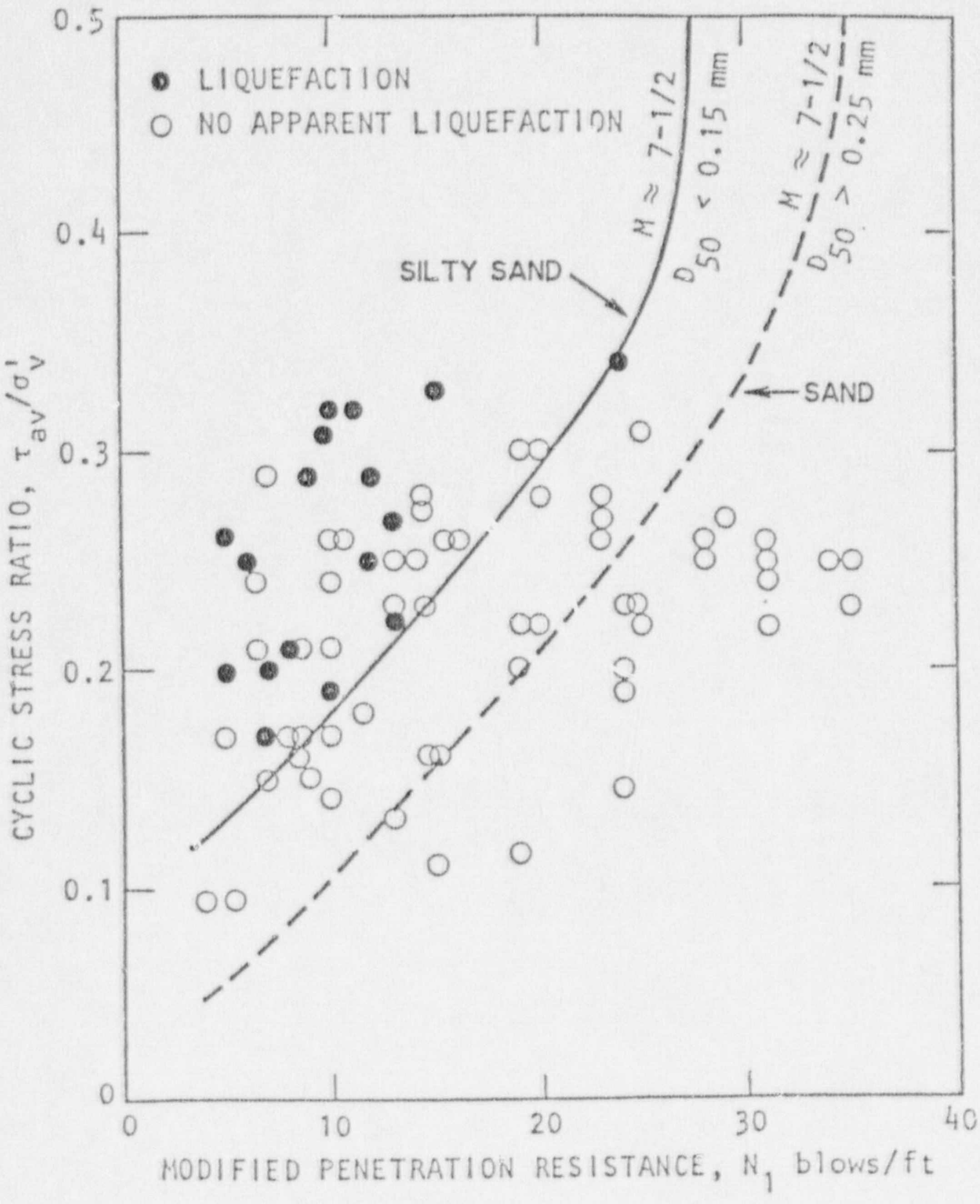

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FIGURE 4-4. LIQUEFACTION POTENTIAL FOR SILTY SAND.

3. From Figure 4-1, select the appropriate value for r_d for the depth in question.

4. Calculate the estimated cyclic stress ratio for the soil element from Equation 1 presented above.

5. Using an estimated relative density and the calculated effective overburden pressure, determine C_n from Figure 4-2.

6. From the boring logs in Appendix A, determine the Standard Penetration Resistance (N) of the soil at the depth in question.

7. Using Equation 2 presented above, calculate the Normalized Penetration Resistance value (N_1).

8. If the soil is a silty sand or sandy silt, add 7.5 to the penetration resistance value found in Step 7.

9. Plot the calculated cyclic stress ratio (Step 4) against the penetration resistance value (Steps 7 and 8) on Figure 4-3. If the point plots below the line for the estimated earthquake magnitude, then field observations indicate that the soil will not liquefy. If the point plots above the line, then the soil may liquefy.

4.2 Liquefaction Potential Analysis

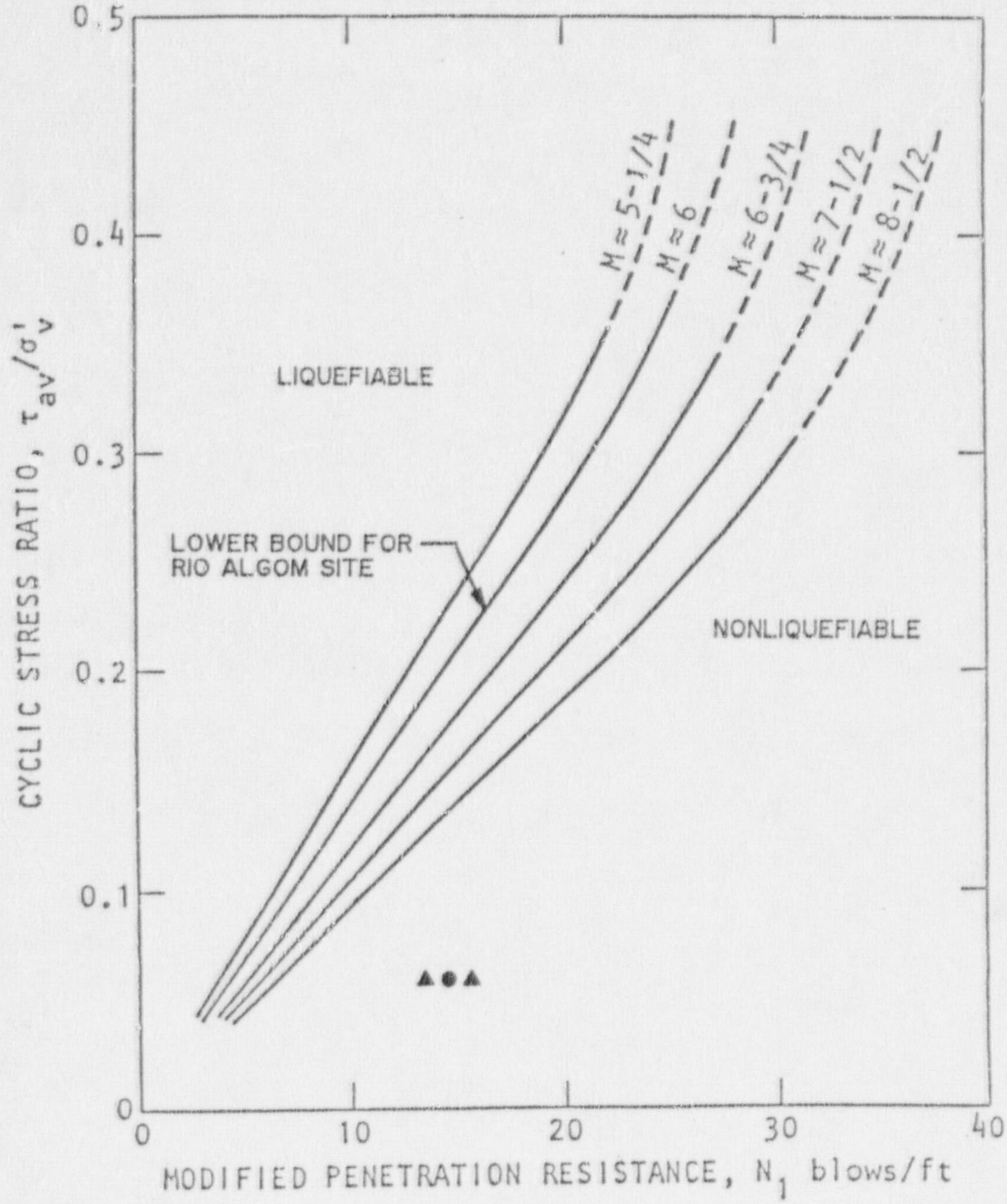
A liquefaction potential analysis of the Bisco Lake embankment and foundation is presented in Table 4-1. The design phreatic surface for the analysis is as shown in Figure 2-2. In the interest of conservatism, the analysis was conducted for saturated locations which appeared to have the greatest liquefaction potential (low Standard Penetration test results and sandy soils). Results of the analysis are plotted on Figure 4-5.

Table 4-1

Liquefaction Potential Analysis Calculations

Boring	B-3(a)	B-9(b)	B-11(b)
Depth (ft.)	27	32	32
Depth to Phreatic Surface (ft.) (from Figure 2-2 and Appendix A)	24	30	29
Unified Soil Classification	ML/CL	CL/ML	CL/ML
Total Stress (psf) 120 pcf wet density	3240	3840	3840
Effective Stress (psf)	3050	3710	3650
r_d (average from Figure 4-1)	0.94	0.91	0.91
Cyclic Stress Ratio (from Equation 1)	0.06	0.06	0.06
C_n (from Figure 4-2)	0.82	0.76	0.77
Standard Penetration Resistance (N) (from Boring logs)	9	9	11
Normalized Penetration Resistance ($N_1 = C_n \times N$)	7	6	8
Modified Penetration Resistance (add 7.5 to N_1 for silty soils)	14.5	13.5	15.5
Liquefaction Potential (From Figure 4-5)	None	None	None

(a) Embankment soil
(b) Foundation soil



● EMBANKMENT SOIL

▲ FOUNDATION SOIL

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FIGURE 4-5. LIQUEFACTION POTENTIAL OF BISCO LAKE FOUNDATION AND EMBANKMENT.

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As can be seen from Figure 4-5, the analysis results plot below the Magnitude 6 lower boundary, indicating that the embankment and foundation soils are not vulnerable to liquefaction.

5.0 CONCLUSIONS

Based on a liquefaction analysis method presented by Seed and Idriss (1982), the Bisco Lake foundation and embankment soils will not liquefy while impounding the probable maximum water volume and subjected to the projected 1000-year peak horizontal acceleration (0.09g). In general, these materials are nonliquefiable for two reasons:

1. The high silt and clay content of the native soils used to construct the embankment.
2. The relatively low peak horizontal acceleration at the Rio Algom site.

The probability of the maximum water impoundment volume corresponding with the peak horizontal acceleration is low. Therefore, the analysis is conservative.

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Dames & Moore. January 18, 1980. Report of Preliminary Tailings Management Study, Lisbon Valley Mine Tailings Disposal System, Near La Sal, Utah, for the Rio Algom Corporation. Salt Lake City, Utah.

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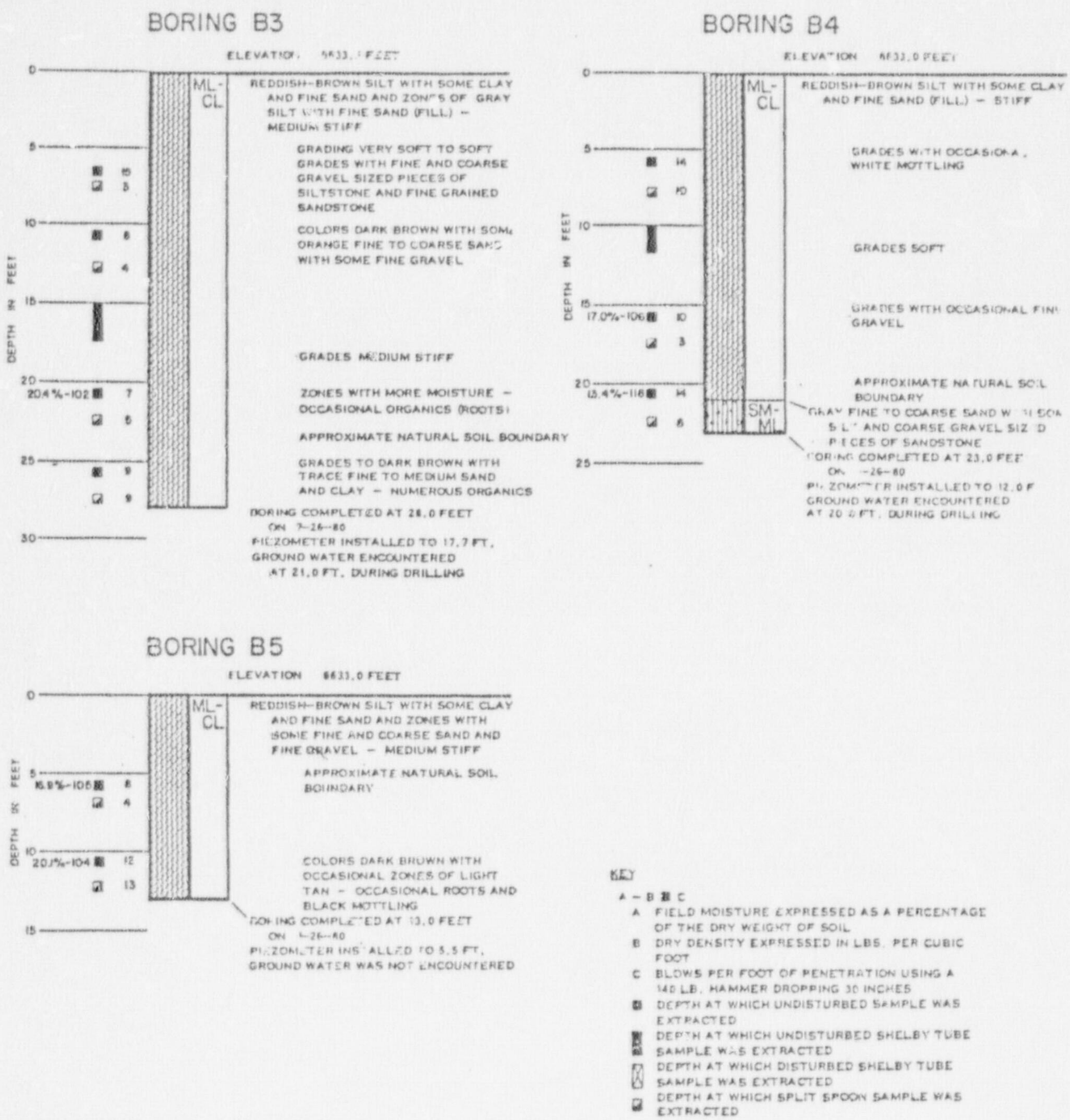
APPENDIX A

LOGS OF BORINGS B-3 THROUGH B-13



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LOG OF BORINGS

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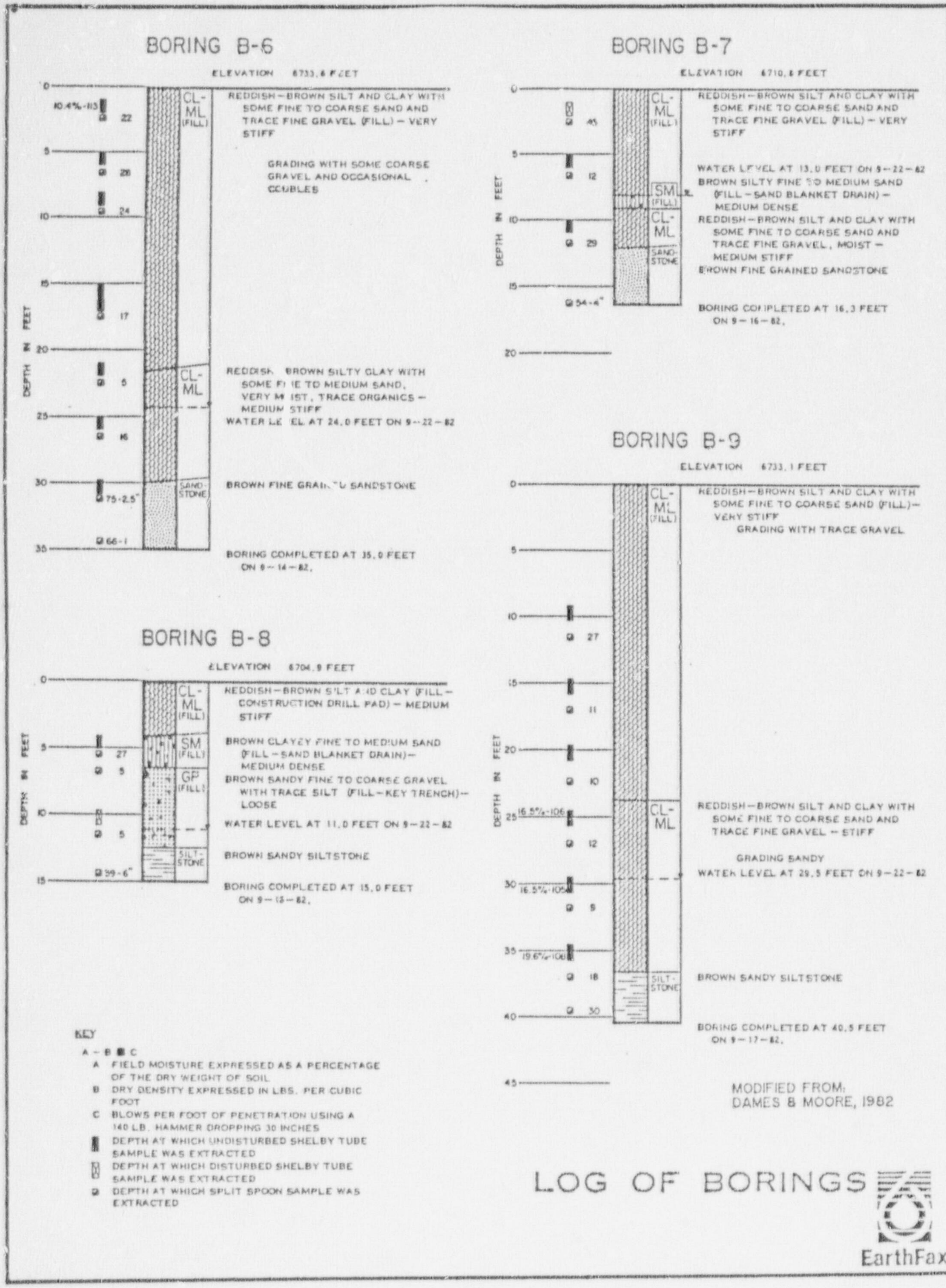

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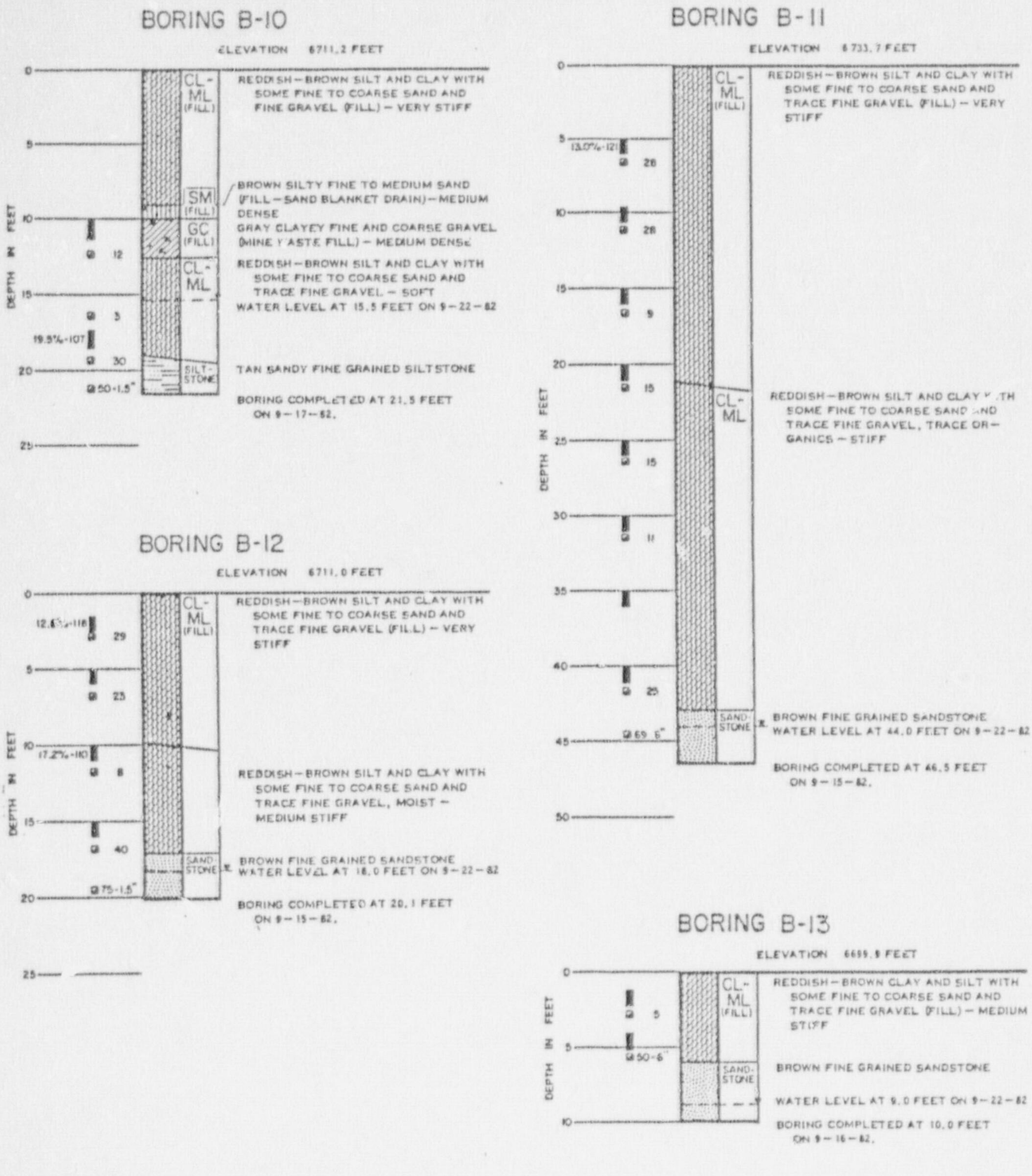

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