KLEINFELDER
An employee owned company

February 24, 2000
File: 30-2759-01.001

RECEIVED
AUG - 8 2000
PBS & J

Mr. Noel J. Suan, Principal
BRG Engineering
3841 North Freeway Blvd., Suite 175
Sacramento, California 95834

**SUBJECT: Geotechnical Investigation Report
Proposed Bridge G-29 Replacement
Lovelock, Nevada**

Dear Mr. Suan:

The attached final report presents the results of our geotechnical investigation for the proposed replacement of bridge No. G-29, located on State Route 396 approximately 12 km (7.5 miles) northeast of Lovelock, Nevada. Two replacement bridge alternatives are being considered including a 2-span cast-in-place box girder and a simple span post-tension structure with MSE walls. Both alternatives have been considered in this report. Our work consisted of subsurface exploration, laboratory testing, engineering analyses, and report preparation. A draft report was submitted to NDOT for review. NDOT's review comments have been incorporated into this final report.

Based on our work completed to date, we have drawn the following general conclusions:

- Subsurface conditions consist of 9.1 to 10.6 m (30 to 35 ft) of granular fill for the bridge approaches over fine grained Pleistocene lake sediments. The approach fill consists of a shallow surface layer of dry, dense, sandy gravel (possibly aggregate base) over dry, loose to medium dense silty sand. The lake deposits are made up of dry, hard, high plasticity clays. No groundwater was encountered during our investigation.

- Our stability analysis performed using the XSTABL5.2 computer program indicates the existing embankment is marginally stable under static conditions. Additional fill will be required to buttress the existing embankment. Alternatively, portions of the fill can be reconstructed with select fill to improve the embankment stability.

- We recommend that the proposed bridge abutments be supported on deep foundations extending below loose approach fill soils into the hard Pleistocene lake deposits. Recommendations for driven pipe pile foundations, sizes PP305, PP406 and PP457, and H piles, sizes HP250 x 85 and HP310 x 125 are provided.

- If a two span bridge alternative is selected, the central bridge pier can be supported on a conventional shallow spread foundation or driven pile foundations. Recommendations for shallow and deep foundations are provided.

- Resistivity testing indicates potentially corrosive soil conditions. Carbon steel corrosion rates are anticipated to be on the order of 12 μm/yr. Assuming a design life of 100 years for the proposed bridge structure and a uniform loss model, steel pipe piles will have a sacrificial wall thickness loss of less than 1.5 mm. We recommend a sacrificial wall thickness of 1.5 mm be used in determining the required pipe pile wall thickness.

These and other conclusions and recommendations, along with restrictions and limitations on these conclusions, are discussed in the attached report.

We appreciate this opportunity to be of service to you, and look forward to future endeavors. If you have any questions regarding this report or need additional information or services, please feel free to call one of the undersigned in our Reno office.

Sincerely,

KLEINFELDER, INC.

2-24-00

Mark Doehring, P.E.
Project Engineer

Al Stilley, P.E.
Regional Senior Engineer

MJD:ANS:pm

Enclosures: Report (5 Bound)

KLEINFELDER 4875 Longley Lane, Suite 100, Reno, NV 89502-5953 (775) 689-7800 (775) 689-7810 fax

TABLE OF CONTENTS

PAGE

APPENDICES

A Site Plan, Boring Logs and Laboratory Test Results

B Application for Authorization to Use.

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED REPLACEMENT BRIDGE
LOVELOCK, NEVADA

1 INTRODUCTION AND SCOPE

1.1 Project Description

This report presents the results of our geotechnical investigation for the proposed replacement of Bridge G-29 located on State Route 396, northeast of Lovelock, Nevada. The approximate project location is shown on the vicinity map (Plate 1).

The existing structure spans a single track of the Union Pacific Railroad line. The existing bridge is approximately 53 m (174 ft) long with 3 intermediate bents. The individual span lengths (going from plan west to east) are 13.7 m, 12.8 m, 13.7 m and 12.8 m. The bridge is constructed of reinforced concrete with steel beams to support the bridge deck at the center spans. Nevada Department of Transportation (NDOT) records indicate that the bridge was retrofitted in 1941. Each abutment is supported on four 250 mm (10 inch) H piles driven to 12.2 m (40 feet) below existing grade at a batter of 1:12 (H:V). Each pile was load tested with approximately 285 kN (32 tons) at a depth of 11.9 m (39 ft). Settlement in each case was less than 25 mm (1 inch). The intermediate bents are supported on shallow spread foundations, 1.67 m x 12.56 m (5 ft 6 in x 41 ft 2.5 in) in plan with a minimum embedment of 0.61 m (2 ft). The retrofitted structure was designed for H15 truck live loading. The bridge approaches are approximately 9.1 to 10.7 m (30 to 35 ft) high constructed out of granular fill.

The replacement bridge will be constructed along the same alignment as the existing structure. Preliminary analysis by BRG Engineering indicates two bridge alternatives are being considered 1) a 2-span cast-in-place box (CIP) girder (52.5 m long) and 2) a simple span post-tension structure (28 m long) with MSE walls. Both bridges will be 11.72 m wide and have a minimum clearance of 7 m from the top of rail. Estimated structural loads for each alternative are provided below.

Alternative 1

2-span CIP Box Girder

Location	Dead Load	Live Load (P-13)		Live Load (HS-20)	
West Abutment	1557 kN (350 kips)	1156 kN (260 kips)	-	667 kN (150 kips)	-
Center Bent	8007 kN (1800 kips)	2891 kN (650 kips)	20427 kN/m (1400 k-f)	1112 kN (250 kips)	8755 kN/m (600 k-f)
East Abutment	2224 kN (500 kips)	1557 kN (350 kips)	-	756 kN (170 kips)	-

Alternative 2

Post-tension Box Girder

Location	Dead Load	Live Load (P-13)		Live Load (HS-20)	
West Abutment	3781 kN (850 kips)	1868 kN (420 kips)	-	800 kN (180 kips)	-
East Abutment	3781 kN (850 kips)	1868 kN (420 kips)	-	800 kN (180 kips)	-

1.2 Purpose and Scope of Work

The purpose of this study is to evaluate the proposed bridge replacement with respect to the observed subsurface conditions, and to provide our geotechnical recommendations and opinions as outlined in our geotechnical proposal, dated September 21, 1999 and summarized below.

- Geologic setting, seismicity, geologic hazards, including expected horizontal accelerations;
- General soil and groundwater conditions at the project site, with emphasis on how the conditions are expected to affect the proposed construction;
- Suggested specifications for earthwork construction, including site preparation recommendations, a discussion of reuse of existing near surface soils as structural or non-structural fill, and a discussion of remedial earthwork recommendations, if warranted;

- Recommendations for temporary excavations and trench backfill;
- Recommendations for permanent fill slopes, including slope protection;
- Alternate foundation types and design values, including soil bearing values, minimum footing depth, resistance to lateral loads, estimated settlements, AASHTO seismic site criteria for use in structural design for the bridge:
- Lateral earth pressures and drainage recommendations for low height retaining structures;
- Potential for site soils to corrode and adversely react with concrete; and
- Structural section for asphalt concrete pavement, to match existing pavement section encountered in the borings.

In response to NDOT comments, we have provided the following additional information:

- Wave equations analysis relating expected driving resistance for deep foundations at the bridge abutments and central pier;
- A Site Response Spectra using an estimated acceleration of 0.13g by USGS and Section 4.6.2 (Codes and Standards) of *Geotechnical Earthquake Engineering*; and
- Recommendations for MSE retaining structures.

Our scope of services consisted of background review, site reconnaissance, field exploration, laboratory testing, engineering analyses, and preparation of a final report.

1.3 Authorization

Authorization to proceed with our work on this project was provided by Mr. Noel J. Suan, P.E. on November 15, 1999 in the form of a signed standard Kleinfelder Inc. contract.

1.4 References

The following information was provided to Kleinfelder in the course of this study and serves as the basis of our understanding of the project type and scope.

- *Topographic Site Plan*, BRG Engineering, undated. This sheet is the basis for the site plan shown on Plate 2.

- *Plans and Profile of Proposed State Highway, Pershing County, Lovelock to Zola,* (1940), Sheets 1, 5 and 7, State of Nevada Department of Highways, September 26.
- Drake, C. C., *G-29 Pershing County, US 95 Facility, UPRR Grade Separation,* (1999), Sheets 1and 2, J. Muller International, December 14, faxed copy received December 21, 1999.
- Preliminary Analysis Results (Structural Loads), BRG Engineering, faxed copy received December 21, 1999.

In addition, the following published references were reviewed during preparation of this report.

- American Association of State Highway and Transportation Officials (AASHTO) (1996), *Standard Specifications for Highway Bridges*, 16th Edition.
- Boore, D.M., W.B. Joyner and T.E. Fumal (1994), Estimation of Response Spectra and Peak Acceleration from Western North American Earthquakes: An Interim Report, Part 2, U.S. Geological Survey, Open-File Report 94-127.
- Boore, D.M., Joyner W.B., and Fumal T.E. (1997), Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes, *Seismological Research Letters*, Vol. 68, No. 1, January/February.
- Cornell, C.A. (1968), Engineering Seismic Risk Analysis, Bulletin of the Seismological Society of America, Vol. 58, No. 5.
- dePolo, C.M., Anderson, J.G., dePolo, D.M., and Price, J.G. (1997), Earthquake Occurrence in Reno-Carson City Urban Corridor, *Seismological Research Letters*, Vol. 68, No. 3, May/June, pp 401-412.
- Frankel, A.D., Mueller, C.S., Barnhard, T., Perkins, D.M., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., (1996), National Seismic Hazard Maps, June 1996 Documentation, USGS Open File Report 96-532, Denver, CO.: available at web site: http://geohazards.cr.usgs.gov/eq.
- Merz, H.L. and Cornell, C.A. (1973), Seismic Risk Analysis Based on a Quadratic Magnitude-Frequency Law, Bulletin of the Seismological Society of America, Vol. 63, No. 6.

- National Highway Institute (1996) *Design and Construction of Driven Pile Foundations*, U.S. Department of Transportation, Federal Highway Administration (FHWA) Publication No. FHWA-HI-97-013, December.
- National Highway Institute (1998) *Earth Retaining Structures*, U.S. Department of Transportation, Federal Highway Administration (FHWA) NHI Course No. 13236, Module 6, May.
- National Highway Institute (1998) *Geotechnical Earthquake Engineering*, U.S. Department of Transportation, Federal Highway Administration (FHWA) Publication No. FHWA-HI-99-012, December.
- National Highway Institute (1998) *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines*, U.S. Department of Transportation, Federal Highway Administration (FHWA) Publication No. FHWA-SA-96-071, September.
- Nevada Bureau of Mines and Geology (1977), *Geologic Map of Pershing County, Nevada*, U.S. Geologic Survey, Bulletin 89, Plate 1
- Pruss, D. E. (1957), *Southern Pacific Geologic Map*, Unpublished, T. 28 N. R. 31 & 32 E. M. D. M., Drawing R3132-28, April – July.
- Siddharthan, R., Anderson, J.G., Bell, J.W. and dePolo C.M. (1993), Peak Bedrock Acceleration For State of Nevada, Final Report to Nevada Department of Transportation, University of Nevada, Reno, Sept.
- Sibbett, B. S. (1980), *Geology Map of Colado Area, Pershing County, Nevada*, Earth Science Laboratory, University of Utah Research Institute, Plate 1.
- *Steel H-Piles* (1971), United States Steel Corporation, September.

2 METHODS OF STUDY

2.1 Field Exploration

Our selection of field exploration locations was based on the anticipated bridge alignment and site access. The subsurface exploration consisted of drilling three borings using a CME55 truck mounted drill rig with hollow stem augers. A boring was advanced at each of the existing abutments to 21.2 m (76 ft) below existing grade. One boring was drilled beneath the bridge, east of the railway line to a depth of 15.5 m (51 ft). Locations of borings are shown on the site plan in Appendix A, Plate 2. These locations were approximated by measuring from features shown on the site plans. Elevations shown on the boring logs were obtained from the project profile provided on Sheet 7 from the State of Nevada, Department of Highways plans, dated September 26, 1940. These locations and elevations should be considered accurate only to the degree implied by the method used.

A description of the Unified Soil Classification System used and Boring Log Key are presented on Plates 3 (Appendix A). Soil conditions encountered are presented on the boring logs, which are included as Plates 4 through 6.

A field geologist logged the soil conditions exposed in the borings and collected relatively undisturbed driven samples for laboratory testing. Soil samples were obtained by driving a 50.8 mm ID, 63.5 mm OD Modified California (MC) Sampler containing thin brass liners, into the bottom of the boring. The number of blows required to drive the last 300 mm of an 450 mm drive with a 64 kg. hammer dropping 760 mm is recorded as the blows per 300 mm (12 inches) (Blow Count) on the boring logs. The blow counts presented on the boring logs have not been corrected for sampler type, overburden, hammer type, rod length, etc. Based on our field experience, the Standard Penetration Test (SPT) Blow Count can be approximated from the MC Sampler Blow Count by multiplying the field count by 0.85. When the sampler was withdrawn from the boring, the brass liners containing the samples were removed, examined for logging, labeled and sealed to preserve the natural moisture content for laboratory testing.

After borings were completed, they were backfilled with excavated soil using the equipment at hand. Borings in existing pavement were backfilled with drill cuttings to the bottom of the pavement structural section. The final few millimeters were backfilled with quick setting low shrinkage grout.

2.2 Laboratory Testing

Laboratory testing is useful for evaluating both index and engineering properties of soils. Typical index tests evaluate soil moisture content, unit weight, soil particle gradation, and plasticity characteristics. Tests for engineering properties can assess soil strength, compressibility, swell potential, and potential for steel corrosion or adverse reactivity with Portland Cement Concrete. We performed laboratory testing on selected soil samples to assess the following:

- Soil Classification (AASHTO T11, T27 and T90)
- Unit Weight and Moisture Content (ASSHTO T204 and T265)
- Consolidation (AASHTO T216)
- Direct Shear Strength (AASHTO T216)

In addition, the following analytical tests were performed by Acculabs, Inc.:

- Soluble Sulfate Content
- Resistivity and pH

Individual laboratory test results can be found on Plates 7 through 19 in Appendix A at the end of this report.

3 DISCUSSION

3.1 Site Conditions

The site is located approximately 12 km (7.5 miles) northeast of Lovelock, Nevada in Pershing County. The surrounding countryside is high mountain desert with a moderate cover of sagebrush and annual grasses. The site is relatively level with no developed drainage observed.

The bridge structure is elevated approximately 9.1 to 10.6 m (30 to 35 ft) above the surrounding landscape and crosses over a single Union Pacific railroad line and a dirt road running parallel to the track. The bridge approaches are constructed out of granular fill with an estimated side slope inclination of 2:1 (H:V).

3.2 Geology

Reference to the 1:250,000 *Geologic Map of Pershing County, Nevada* indicates the site is underlain by thick sequences of Pleistocene lake sediments. The deposits were formed mainly during the inundation of large areas of western Nevada by the waters of Lake Lahotan, the largest Pleistocene pluvial lake in the Great Basin. The lake deposits consist of sand, silt, gravel, clay, tufa and saline minerals.

3.3 Subsurface Conditions

The following summarizes the results of our field exploration. The boring logs should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

The existing pavement structural section consisted of 150 mm of asphalt on approximately 130 mm of aggregate base (6 inches AC on 5 inches of AB). Subsurface conditions beneath the pavement section consisted of approximately 9.1 to 10.6 m (30 to 35 ft) of granular fill for the bridge approach embankments over very stiff to hard, moist clay lake deposits. No groundwater was encountered in any of the borings during the period of several hours for which they were open. Fluctuations in the level of the groundwater and soil moisture conditions as noted in this report may occur due to variations in precipitation, land use, and other factors.

3.4 Faulting

The bridge site is dominated by the faults of Basin and Range province. The tectonic setting of the Basin and Range province is characterized by active extension expressed by evenly spaced, sub-parallel mountain ranges and intervening alluviated basins. Basin and ranges throughout much of the province are bounded by north-trending normal faults. The study area is considered to have low to moderate seismic activity. The tectonics in the area are mainly influenced by several north-south trending fault zones.

In the vicinity of the bridge, the West Humboldt Range fault zone at about 3.4 km to the southeast is the closest fault. A major seismic event on this fault could cause significant ground shaking at the site. Other faults in the region, which can have significant impact on the site, are listed in the Table 1. The approximate location of the West Humboldt Range fault, along with other active and potentially active faults in the area relative to the project site is presented on Plate 20 in Appendix A. The fault parameters presented in Table 1 are primarily based on data provided by Siddharthan et. al. (1993) and dePolo (1997). Additional information has been obtained from Frankel et. al. (1996).

TABLE 1
SIGNIFICANT FAULTS

Fault Name	Fault Length (km)	Closest Distance to Site (km)	Magnitude of Maximum Earthquake *	Slip Rate (mm/yr)	Values of	
					a	b
West Humboldt Range	65	3.4	7.0	0.1	1.36	0.82
East Humboldt Range	20	21	6.9	0.1	1.37	0.82
Seven Troughs Range	33	38	6.9	0.1	1.36	0.80
Buena Vista Valley	49	39	7.1	0.1	1.39	0.80
Eastern Carson Sink	25	48	7.0	0.1	1.35	0.80
Dixie Valley	100	52	7.0	0.3	2.24	0.80
Eugene Mountains	11	53	6.2	0.1	1.36	0.86
The Lava Beds	26	53	6.6	0.1	1.39	0.80
Granite Springs Valley	45	59	7.0	0.5	2.12	0.80
Dunn Glen	16	59	6.5	0.1	1.33	0.80
Eastern Dixie Valley	30	66	6.9	0.1	0.34	0.80
Grass Valley	51	68	7.1	0.1	1.40	0.80
Rainbow Mountains	38	70	7.0	0.1	1.35	0.80
Jersey Valley 1	20	70	6.6	0.1	1.34	0.80
Jersey Valley 2	16	77	6.5	0.1	1.32	0.80
Selenite Range	57	78	7.2	0.1	1.38	0.80
Black Rock	102	83	7.5	0.1	1.42	0.80
Fallon	18	86	6.8	0.1	0.40	0.80
San Emidio	32	87	6.8	0.1	1.41	0.80
Western Edwards Creek Valley	34	88	6.9	0.16	1.57	0.80
Clan Alpine Mountains	38	89	7.0	0.1	1.35	0.86
Fairview Peak	48	89	7.2	0.1	1.30	0.80

* moment magnitude
** not available

The “a” and “b” values listed in this table are a measure of the frequency of occurrence of earthquakes of various magnitudes. The general form of this recurrence model is based on the Gutenberg-Richter (Gutenberg and Richter, 1956) exponential frequency-magnitude relationship:

$$\log N(M) = a - bM$$

where N(M) is the cumulative number of earthquakes of magnitude "M" or greater per year, and "a" and "b" are constants based on recurrence analyses.

3.5 Seismicity

Historically, the area has been subject to low to moderate seismic activity. Most of the historic seismic activity is associated with the Dixie Valley fault system and the Rainbow Mountain fault. Some of the significant nearby events include the 1915 (M7.8 and M6.1) earthquakes, about 75 km to the northeast and possibly associated with the northern segment of the Dixie Valley fault; the 1954 (M6.9) earthquake, about 60 km to the southeast and associated with the Dixie Valley fault; the 1954 (M6.8) Rainbow Mountain earthquake, about 100 km to the south; and the 1959 (M6.3) the Fairview Peak earthquake, about 83 km to the southeast. Epicenters of significant earthquakes (M>4.0) within the vicinity of the site are shown on Plate 20. The project site is not within any part of Earthquake Fault Zone and no active shear zones are known to exist at the site. However, the site will experience strong ground shaking in case of a seismic event at one of the nearby faults.

We have also searched our earthquake data base for the historical seismicity of the site and its vicinity. The earthquake data base used contains in excess of 5,500 seismic events and covers the period from 1800 through December 1999. The earthquake data base is principally comprised of an earthquake catalog for the State of California prepared by the Division of Mines and Geology (CDMG) and the Seismic Network of University of Nevada, Reno. The original CDMG catalog (Real, et. al, 1978) is a merger of the University of California at Berkeley and the California Institute of Technology instrumental catalogs (Hileman, et. al, 1973). The combined catalog contains earthquake records from January 1, 1900 through December 31, 1974. Updates prepared by CDMG in 1979 and 1982 extend the coverage through 1982. In addition to the CDMG updates, the data for earthquakes for period between 1910 and December 1999 have been obtained from a composite catalog by Council of the National Seismic System (CNSS). The CNSS catalog is a world-wide earthquake catalog, which is created by merging the master earthquake catalogs from contributing CNSS member networks and then removing duplicate events, or non-unique solutions from the same event. The CNSS network includes Northern and Southern California Seismic Networks, Pacific Northwest Seismic Network, University of Nevada, Reno Seismic Network, University of Utah Seismographic Stations and US National Earthquake Information Service. The earthquake data base also consists of earthquake records between 1800 and 1900. This subset of the earthquake data base was derived from Seeburger and Bolt (1976) and Toppozada, et. al (1978, 1981).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through December 1999), and magnitudes (M>4). A summary of the results of the historical search is presented below.

Time Period (1800 to December 1999)	200 years
Maximum Magnitude	7.8
Approximate distance to nearest M>4 historical earthquake	3.5 km
Number of events exceeding magnitude 4 within search area	196

3.6 Ground Motion

3.6.1 General

The ground motions in terms of peak ground accelerations at the site were estimated using both deterministic as well as probabilistic methods. Both of these methods require selection of an attenuation relationship for the attenuation of the ground motions from the fault to the site. The type of faulting, magnitude of earthquake, distance from the site, and the soil conditions are accounted in the attenuation relationship.

3.6.2 Attenuation Relationship

Many attenuation relationships have been developed to estimate the variation of peak ground surface acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Of these relationships, we have selected a relationship presented by Boore et. al. (1994,1997) because of its wide acceptance by seismologists. This relationship has also been used in developing recent National Seismic Hazard Maps (Frankel et. al., 1996) for the State of California and Nevada. This relationship uses an estimate of site shear wave velocity in the analyses. Therefore, an average site shear wave velocity of 310 m/s, recommended by the authors for typical soil deposits, was used in our analyses. This predictive relationship was developed from statistical analyses of recorded earthquakes from Western North America, including the records from the 1989 Loma Prieta earthquake and 1992 Landers earthquake. The attenuation relationships provide mean values of ground motions associated with one set of parameters: magnitude, distance, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis.

3.6.3 Deterministic Method

The deterministic method utilizes the maximum earthquake magnitude associated with the faults considered in the analysis. This method does not account for the relative activity of the fault and assumes that the site will be subjected to the ground shaking from a maximum earthquake associated with that fault. Table 2 shows the mean and mean + 1 standard deviation values of peak ground accelerations at the site due to the maximum earthquake associated with each fault.

TABLE 2

DETERMINISTIC PEAK GROUND ACCELERATIONS

Fault Name	Closest Distance to Site (km)	Magnitude of Maximum Earthquake *	Peak Ground Accelerations (g)	
			Mean	Mean + 1σ
West Humboldt Range	3.4	7.0	0.54	0.91
East Humboldt Range	21	6.9	0.20	0.34
Seven Troughs Range	38	6.9	0.13	0.22
Buena Vista Valley	39	7.1	0.14	0.24
Eastern Carson Sink	48	7.0	0.11	0.19
Dixie Valley	52	7.0	0.11	0.18
Eugene Mountains	53	6.2	0.07	0.12
The Lava Beds	53	6.6	0.09	0.14
Granite Springs Valley	59	7.0	0.10	0.16
Dunn Glen	59	6.5	0.07	0.13
Eastern Dixie Valley	66	6.9	0.08	0.14
Grass Valley	68	7.1	0.09	0.15
Rainbow Mountains	70	7.0	0.09	0.14
Jersey Valley 1	70	6.6	0.07	0.12
Jersey Valley 2	77	6.5	0.06	0.10
Selenite Range	78	7.2	0.09	0.15
Black Rock	83	7.5	0.10	0.16
Fallon	86	6.8	0.07	0.11
San Emidio	87	6.8	0.06	0.11
Western Edwards Creek Valley	88	6.9	0.07	0.11
Clan Alpine Mountains	89	7.0	0.07	0.12
Fairview Peak	89	7.2	0.08	0.13

* moment magnitude

3.6.4 Probabilistic Analysis

The probabilistic method accounts for the relative activity of the fault and historical seismicity of the region. A probabilistic modeling procedure was used to estimate the peak ground motion for the Design Basis Earthquake (DBE). According to the 1997 UBC, the DBE is defined as a ground motion having 10 percent probability of exceedance in 50 years (a return period of about 475 years). The probabilistic analysis approach is based on the characteristics of the earthquake and of the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake, distance from the site to the causative fault, maximum earthquake magnitude, length, and activity of the fault. The effects of site soil conditions and mechanism of faulting are accounted for in the attenuation relationships.

The theory behind the seismic risk analysis has been developed over many years (Cornell, 1968, 1971; Merz and Cornell, 1973) and is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another. According to this approach, the probability of exceeding PE(Z) at a given level of ground motion, Z, at the site within a specified time period, T, is given by

$$PE(Z) = 1 - e^{-\vartheta(Z)T}$$

where $\vartheta(Z)$ is the mean annual rate of exceedance of ground motion level Z. Different probabilities of exceedance may be selected, depending on the level of performance required.

3.6.5 Peak Ground Acceleration

Based on the results of our probabilistic analyses, the calculated peak ground horizontal acceleration (in units of gravity) for the DBE is presented in Table 3 below. The corresponding return period and annual probabilities of occurrence are also shown in Table 3.

TABLE 3

PEAK GROUND ACCELERATION

Event	Return Period	Probability of Occurrence	Annual Probability of Exceedance	Peak Horizontal Acceleration (g)
DBE	475	10% in 50 years	0.0021	0.17

Our estimated peak ground acceleration for the DBE compares well with the value of 0.13g estimated by USGS (Frankel et. al., 1996) for a rock like conditions at the subject site. Based on our experience with similar projects, it is our opinion that the probabilistic peak ground acceleration of 0.17g associated with the DBE may be used in the design of the bridge.

3.7 Elastic Response Spectra

As requested by the NDOT, the elastic horizontal response spectra for this project were developed based on the method presented in Figure 4-18 of Section 4.6.2 of FHWA publication Geotechnical Earthquake Engineering. We have used a peak ground acceleration value of 0.13g as estimated from the USGS probabilistic seismic hazard map for a probability of exceedance of 10 percent in 50 years for this site. This PGA represents an acceleration value for competent rock site. This approach incorporates the soil profile at the site in developing the design spectra by using a soil factor. However, it should be noted that these spectra do not replace a site specific spectra which utilizes the appropriate attenuation relationship reflecting the site soil profile type.

Table 4 lists the seismic design parameters used in developing the UBC based elastic design spectra.

TABLE 4

SEISMIC DESIGN PARAMETERS

Items	Values	Reference
Seismic Zone	4	Figure 16-2 of 1997 UBC
Soil Profile Type	S_D	Table 4-4 of HI-99-012
Closest Fault	West Humboldt Range	Plate 20 & Table 1
Fault Type	B	Table 4-6 of HI-99-012
Fault Distance	3.4 km	Plate 20 & Table 1
Bedrock PGA	0.13	USGS
Na	1.16	Table 4-7 of HI-99-012
Nv	1.41	Table 4-8 of HI-99-012

Response spectral values for the DBE were calculated using the approach described above. Estimated response spectral values calculated are for damping of 5 percent of critical. The resulting design spectra for the DBE is presented on Plate 20a. We have also presented the spectral acceleration values in Table 5.

TABLE 5

SPECTRAL ACCELERATION VALUES (g)

Period (sec)	5% Damping
0.01	0.166
0.14	0.415
0.71	0.415
0.80	0.367
1.00	0.294
1.20	0.245
1.50	0.196
2.00	0.147
2.50	0.118
3.00	0.098
3.50	0.084
4.00	0.073

4 CONCLUSIONS

The following conclusions are based on the data collected during this assessment and are subject to the limitations stated in this report. These conclusions may change if additional information becomes available. Based on the results of our study, no severe soil or groundwater constraints were observed which would preclude the planned construction. The following is a summary of our conclusions.

- Subsurface conditions consist of 9.1 to 10.6 m (30 to 35 ft) of granular fill for the bridge approaches over fine grained Pleistocene lake sediments. The approach fill consists of a shallow surface layer of dry, dense, sandy gravel (possibly aggregate base) over dry, loose to medium dense silty sand. The lake deposits are made up of dry, hard, high plasticity clays. No groundwater was encountered during our investigation.

- Our stability analysis performed using the XSTABL5.2 computer program indicates the existing embankment is marginally stable under static conditions. Additional fill will be required to buttress the existing embankment. Alternatively, portions of the fill can be reconstructed with select fill to improve the embankment stability.

- We recommend that the proposed bridge abutments be supported on deep foundations extending below loose approach fill soils into the hard Pleistocene lake deposits. Recommendations for driven pipe pile foundations, sizes PP305, PP406 and PP457, and H piles, sizes HP250 x 85 and HP310 x 125 are provided.

- If a two span bridge alternative is selected, the central bridge pier can be supported on a conventional shallow spread foundation or driven pile foundations. Recommendations for shallow and deep foundations are provided.

- Resistivity testing indicates potentially corrosive soil conditions. Carbon steel corrosion rates are anticipated to be on the order of 12 μm/yr. Assuming a design life of 100 years for the proposed bridge structure and a uniform loss model, steel pipe piles will have a sacrificial wall thickness loss of less than 1.5 mm. We recommend a sacrificial wall thickness of 1.5 mm be used in determining the required pipe pile wall thickness.

Specific recommendations for project design and construction including mitigation of potential problems described above are presented in Section 5.0.

5 RECOMMENDATIONS

5.1 Standard Specifications

Specifications or Standard Specifications as referenced to in this report, mean the "Standard Specifications for Road and Bridge Construction, State of Nevada, Department of Transportation," 1996 Edition.

5.2 Site Clearing and Preparation

Prior to construction, all man-made debris such as foundations, dump fills and trash should be removed from the alignment. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment.

Surface vegetation and organic soils should be stripped and removed from the alignment or stockpiled for use in non-structural areas. Clearing and grubbing should be performed in accordance with Section 201 of NDOT Standard Specifications for Road and Bridge Construction. It appears 150 mm (6 inches) can be used as a reasonable estimate for average depth of stripping. Deeper stripping/grubbing may be required in localized areas. The resulting voids should be backfilled with adequately compacted backfill soil.

The geotechnical engineer should be present during site preparation to observe stripping and grubbing depths, and to evaluate whether buried obstacles are present. Special care should be exercised in evaluating whether loose utility backfills exist which could adversely affect the planned pavements and structure.

Dust control will be the responsibility of the contractor. A dust control plan should be prepared by NDOT or the contractor prior to the start of grading.

5.3 Earthwork

5.3.1 General Site Grading

Site preparation and grading should conform to the requirements contained in this report and in the standard specifications. We anticipate that site grading can be performed with conventional earthmoving equipment.

Where fill is necessary, materials should meet the requirements listed for "Select Borrow" in Section 203 of NDOT Standard Specifications, which requires a minimum R-value of 45 and 100% of the material passing the 75 mm sieve size. The native clay soils do not meet recommended requirements for structural fill. Imported granular fill will be required. Fill placement and compaction requirements should be in accordance with Section 208 of NDOT Standard Specifications.

Prior to fill placement, the exposed native soils should be scarified to a minimum depth of 150 mm (6 inches), moisture conditioned within 2% of optimum moisture, and compacted to a minimum of 90% relative compaction in accordance with the Nev. T101 compaction test method.

5.3.2 Recommended Permanent Slope Angles

The stability analyses were performed using the XSTABL5.2 computer program, which analyzes circular and non-circular failure surfaces and their attendant factors of safety using the modified Bishop method. The program analyzes two-dimensional cross-sections of the slope using available information on the subsurface structure, strength of the various earth materials, surcharge or seismic loading conditions. The computed factor of safety is the ratio of forces tending to resist movement to the forces tending to drive movement. A factor of safety of less than 1 suggests the slope is unstable. From a practical perspective, a factor of safety of at least 1.5 is used as an acceptance criteria for the static load case, and a minimum factor of 1.1 is used for the seismic case. The horizontal acceleration used in our seismic stability analyses was based on 50% of the peak ground acceleration (0.17g) for 10% probability of exceedance in 50 years based on our probabilistic analyses. In addition we performed a second stability analyses using 50% of the design peak acceleration of 0.4g as outlined in Figure 1-5 of the AASHTO Bridge Design Manual. The results of these analyses are shown on Plates 21 and 29 for static and seismic states and are also presented in Table 6. Each analysis included a surcharge load of 9.6 kPa (200 psf) to represent live loading conditions due to traffic.

As shown below in Table 6, under static conditions the existing slope is marginally stable. Either additional fill is necessary to buttress the existing embankment or portions of the slope will need to be reconstructed with select fill to force the critical slope failure beyond the existing embankment face. If additional fill is used to buttress the slope, we recommend that the permanent slope be constructed at a maximum slope of 2.5:1 (H:V). Alternatively, the portions of existing embankment can be reconstructed using select fill to maintain the existing 2:1 (H:V) slope inclination.

Our pseudo static analyses using 50% of the AASHTO peak accelerations indicates the slope and proposed alternatives fail to meet a minimum safety factor of 1.1. It is our opinion that the pseudo seismic analyses using 50% of the AASHTO peak acceleration is very conservative.

TABLE 6

RESULTS OF EMBANKMENT STABILITY ANALYSES

Soil Type	Properties	Embankment Height & Slope	Seismic Coefficients	Factor of Safety
Existing Fill	14.9 kN/m^3 (95 pcf)	9.1 m (30 ft)		
(average measured properties)	C= 2.4 kPa (50 psf)			
	Ø = 26°			
Native	17.3 kN/m^3 (110 pcf)			
(average measured properties)	C= 43.1 kPa (900 psf)			
	Ø = 10°			
		*2:1 (H:V)	0V,0H	1.3
			0V,0.085H	1.1
			0V,0.2H	0.9
		**2.5:1 (H:V)	0V,0H	1.6

Soil Type	Properties	Embankment Height & Slope	Seismic Coefficients	Factor of Safety
			0V,0.085H	1.3
			0V,0.2H	1.0
Select Engineered Fill for Partial Embankment Reconstruction	18.1 kN/m^3 (115 pcf)			
(assumed average properties)	C=4.8 kPa (100 psf)			
	Ø = 34°			
		***2:1 (H:V)	0V,0H	1.6
			0V,0.075H	1.3
			0V,0.2H	1.0

* Existing fill conditions
** Buttressed fill conditions
*** Reconstructed slope with select fill

If portions of the embankment are to be reconstructed, we recommend that select fill extend a minimum horizontal distance of 6 m (20 feet) into the base and 3 m (10 feet) into the top of the embankment on each side. The new fill should be continuously benched into the existing embankment as work is brought up in layers. The embankment reconstruction should be performed in accordance with Section 203 of NDOT Standard Specifications for Road and Bridge Construction.

Recommended average material properties for select fill are provided in Table 6 above. These properties may be achieved by importing material or through mechanical or physical means, such as a reinforced soil slope with metallic or polymeric reinforcing elements or ground improvement with cement or fly ash mixing. An analyses mechanical or physical stabilization is beyond our current scope of work.

Satisfactory slope performance is primarily affected by drainage and runoff. Care must be taken that drainage is not directed to flow over slope faces. Slope faces should be protected against erosion resulting from direct rain impact and melting snow. Consideration should be given to permanent measures such as geosynthetics and vegetation.

5.3.3 Temporary Trench Excavation and Backfill

It appears that excavations for footings and utility trenches can be readily made with either a conventional backhoe or excavator in either native soil or compacted imported fill. We expect excavations in the native soils to stand near vertically to a depth of approximately 1.5 m (5 ft) without significant sloughing provided that proper moisture contents are maintained and there are sufficient cohesive fines within the imported fill. Excavations in the existing embankment fill will likely be unstable. Construction personnel should evaluate all excavations to verify their stability prior to occupation. Shoring or sloping of trench walls may be necessary to protect personnel and provide temporary stability. All excavations in the native soils should comply with current OSHA safety requirements for Type A soils (Federal Register 29 CFR, Part 1926). Excavations in granular fill should comply with OSHA requirements for Type C soils.

During wet weather, runoff water should be prevented from entering excavations. Water should be collected and disposed of outside the construction limits. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance of one-third the slope height from the top of any excavation.

Backfills for trenches or other excavations within pavement areas, beneath concrete slabs, and adjacent to foundations should be compacted in 150 mm to 200 mm (6 to 8 inch) layers with mechanical tampers. Jetting and flooding should not be permitted. We recommend all backfill be compacted to a minimum compaction of 90% of the maximum dry density as determined by Nev. T101. The moisture content of compacted backfill soils should be within 2% of the optimum. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement structural section or other overlying improvements.

5.4 Foundations

Due to the loose to medium dense nature of the existing fill soils, we recommend that the proposed bridge abutments be supported on deep foundations. Recommendations for driven pipe pile foundations, sizes PP305, PP406 and PP457, and H piles, sizes HP250 x 85 and HP310 x 125 are provided. In the case of a two span structure the proposed center pier can be supported on a conventional shallow spread foundation or a deep foundation. Recommendations for both foundation alternatives are provided.

5.4.1 Conventional Shallow Foundation Design Parameters

Our recommendations for ultimate and allowable bearing pressures and anticipated settlements for the bridge central pier are provided below in Tables 7A and 7B, respectively. Ultimate bearing pressures are provided as a function of footing width and embedment depth. Bearing Capacities were calculated in accordance with AASHTO *Standard Specifications for Highway Bridges, 16th edition, 1996, Section 4.4.7.*

TABLE 7A

ULTIMATE BEARING CAPACITY (kPa)
AND ESTIMATED SETTLEMENT (mm)

Depth (m)	Footing Width (m)							
	1.5		2.0		2.5		3.0	
	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)
1.0	507	12	513	14	519	16	525	16
1.5	531	18	538	20	543	22	549	22
2.0	554	22	561	26	566	28	572	28
2.5	573	26	580	30	585	32	591	32

TABLE 7B

ALLOWABLE BEARING CAPACITY (kPa)
AND ESTIMATED SETTLEMENT (mm)

Depth (m)	Footing Width (m)							
	1.5		2.0		2.5		3.0	
	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)	Bearing Capacity	Settlement (mm)
1.0	169	10	171	12	173	12	175	12
1.5	177	12	179	14	181	16	183	16
2.0	185	16	187	18	189	20	191	20
2.5	191	18	193	20	195	22	197	22

Foundations should bottom on hard native soil or compacted structural fill. Any loose soil in the bottom of the footing excavation should be recompacted to at least 95% relative compaction or removed to expose firm unyielding material.

The allowable bearing pressures were calculated using a factor of safety of 3.0. The allowable bearing pressure value may be increased by one-third for total loading conditions, including wind and seismic forces. The allowable bearing pressure is a net value; therefore, the weight of the foundation and backfill may be neglected when computing dead loads.

If seismic loading is evaluated in accordance with AASHTO *Standard Specifications for Highway Bridges 16th edition, 1996, Section 3*, we recommend using a peak acceleration coefficient of 0.4g and a Site Coefficient (S) of 1.2 which is applicable to a Type II Soil Profile. It should be noted that our probabilistic analysis indicates a peak acceleration of 0.17g for the site.

5.4.2 Deep Foundation Design Parameters

Included in Plates 30 through 39 (Appendix A) are design charts for ultimate capacities with depth for single driven closed end pipe piles, designations PP305, PP406, and PP457 and H-piles, designations HP250 x 85 and HP310 x 125. Plates 30 through 34 are for piles driven through the embankment. Plates 35 through 39 are for piles driven for the central bridge pier. A group efficiency value of 1.0 should be used for piles with a minimum center-to-center spacing of three times the pile diameter (3B). An efficiency factor of 0.7 should be used for piles with center-to-center spacings of less than 3B but greater than 2.5B. Center-to-center spacings of less than 2.5B are not recommended. All piles driven through embankment fills should extend a minimum of three meters into the original ground surface.

In accordance with AASHTO *Standard Specifications for Highway Bridges, 16th edition, 1996, Table 4.5.6.2A,* we recommend that allowable pile capacities be calculated using a factor of safety of 2.75. This factor of safety value is consistent with ultimate bearing capacities being determined from static calculations evaluated with a WEAP analysis. For any construction control, which includes a static load test, a factor of safety of 2.0 may be used.

NDOT

We recommend a minimum of Grade 3 and A-36 steel for pipe and H-piles. Piles should be designed for a maximum allowable design stress of 0.25 f_y or 0.33 f_y if damage is unlikely and confirming load tests are performed. Driving stress should be limited to a maximum compression and tension driving stresses of 0.9 f_y.

Wave equations relating expected driving resistance with depth for pipelines at each bridge location are provided below in Table 8. Driving resistances were calculated assuming various Delmag diesel hammers and wood block cushions.

TABLE 8

SUMMARY OF WEAP ANALYSIS

Boring Location	Pile Length (m)	Pile Type	Ultimate Pile Capacity (kN)	Required Yield Strength (MPa)	Hammer Description	Blows per Meter	Driving Stress (MPa)	Allowable Driving Stress (MPa)
B-1	12.5	PP305 7.9mm wall	612	310	Delmag D 36-32	28	279.0	279
B-1	12.5	PP406 9.5mm wall	1080	310	Delmag D 62-22	36	278.9	279
B-1	12.5	PP457 9.5mm wall	1102	310	Delmag D 80-23	26	278.9	279
B-2	12.5	PP305 7.9mm wall	568	310	Delmag D 36-32	25	278.9	279
B-2	12.5	PP406 9.5mm wall	919	310	Delmag D 62-22	39	277.7	279
B-2	12.5	PP457 9.5mm wall	937	310	Delmag D 80-23	21	272.6	279
B-3	10	PP305 7.9mm wall	612	310	Delmag D 36-32	28	279.0	279
B-3	10	PP406 9.5mm wall	1080	310	Delmag D 62-22	36	278.9	279
B-3	10	PP457 9.5mm wall	1102	310	Delmag D 80-23	25	278.9	279

Notes: Hammer efficiency is assumed to be 80 percent. A factor of safety of 2.75 is used for ultimate pile capacities.

The recommended driving shoe to be used for pipe piles is presented on Plate 40 in Appendix A.

The design uplift capacity of a single pile should be taken as 1/3 of the calculated ultimate shaft resistance plus the weight of the deep foundation. The ultimate shaft resistance with depth for driven closed end pipe piles, designations PP305, PP406, and PP457 and H-piles, designations HP250 x 85 and HP310 x 125 are provided on Plates 41 through 50. Plates 41 through 35 are for piles driven through the embankment. Plates 36 through 50 are for piles driven for the central bridge pier.

Soil parameters for laterally loaded pile analysis are provided in Table 9. Design parameters were estimated based on corrected blow counts, laboratory index test results and recommendations provided in Tables 9-12 and 9-13 in FHWA *Design and Construction of*

Driven Pile Foundations, Workshop Manual, Volume 1, December 1996. These values are based on the use of the COM624P Program for design.

TABLE 9

PILE LATERAL LOAD DESIGN PARAMETERS

Material Type	Elevation (m)	Total Density (kN/m^3)	Soil Parameters	k (kN/m^3)
Granular Fill	Above 1243	15	Ø = 26°	15,610 (static and cyclic)
Hard Native Clay	Below 1243	17.3	Cu = 300 kPa	543,000 (static)
			$_{50}$ = 0.004	217,000 (cyclic)

5.5 Retaining Structures

Lateral earth pressures will be imposed on all subterranean structures, including retaining walls and foundations. Table 10 presents a list of soil parameters, which we recommend for design of these structures.

TABLE 10

ULTIMATE LATERAL EARTH PRESSURES

Backfill Angle	Earth Pressure	Earth Pressure Coefficient	Equivalent Fluid Density
0°	Active	0.35	5.3 kN/m^3
	At-rest	0.50	7.5 kN/m^3
	*Passive	5.0	76.5 kN/m^3
	Active (K_{AE})	0.75	11.3 kN/m^3
	Passive (K_{PE})	3.4	51.0 kN/m^3
26° (2:1)	**Active	0.4	6.0 kN/m^3
	**Active (K_{AE})	1.1	16.5 kN/m^3
22° (2.5:1)	Active	0.45	7.0 kN/m^3
	Active (K_{AE})	1.0	15 kN/m^3
	Friction Coefficient (tan δ)		0.3
	Angle of Internal Friction (ϕ)		30°
	Wall Friction Angle (δ)		18°
	Unit Weight (γ)		15 kN/m^3

*The passive coefficient assumes that the ground surface in front of the retaining wall is level.
** An angle of internal friction of 34 degrees was used in calculating the active and passive cases for back slope of 2:1 (H:V) since portions of the slope will need to be rebuilt in order to provide a stable embankment.

Earth pressures provided above were determined in accordance with recommendations provided in the Federal Highway Administration *Earth Retaining Structures, Reference Manual (Draft),* May 1998. Seismic lateral earth pressure design parameters were calculated using the Mononobe-Okabe analysis as outlined in the Federal Highway Administration *Geotechnical Earthquake Engineering,* Publication No. FHWA HI-99-012, December 1998. Earth pressures are ultimate values. Recommended minimum factors of safety against sliding, overturning and bearing failure are listed in Table 11, below.

TABLE 11

Recommended Minimum Factors of Safety	
Factor of safety against sliding	1.5
Factor of safety against overturning	2.0
Factor of safety against bearing failure	3.0

If both passive and frictional resistances are assumed to act concurrently, we recommend a minimum safety factor of 2.0 be used for design against sliding.

The at-rest case is applicable for braced walls where rotational movement is confined to less than 0.001 H. If greater movement is possible, the active case applies. A wall movement of about 0.01H is required to develop the full pressure. The passive pressure resistance should also not be considered effective in the upper 24 inches of the subsurface soil profile. These values do not include hydrostatic pressures that might be caused by groundwater or surface water trapped behind a structure. Where backfill is placed against structures such as retaining walls, we recommend that non-expansive, free-draining materials meeting NDOT filter criteria be used in the zone immediately adjacent to the structure to reduce hydrostatic forces. The free-draining material should have a minimum lateral thickness of 600 mm. Alternately, the use of pre-manufactured drainage panels should be considered. Furthermore, adequate drainage of the backfill in the form of subdrains and/or weepholes should be provided at the base of the wall. If weepholes are constructed, they should be on a maximum of three-meter centers vertical spacing, five-meter centers horizontal spacing, and have a minimum diameter of 102 mm. All weepholes should be backed with a minimum of 0.06 cubic meters of Type 2 drain backfill encased in geofabric, Mirafi 160N, 180N, or equal.

General backfill should be non-expansive, such as NDOT Granular Backfill. The lateral loads computed using the values in Table 4 assume that the non-expansive backfill will extend laterally at least one-half of the wall height. If this condition does not apply, the design values may require revision. This backfill should be compacted to 95% of maximum dry density and within 2% of the optimum moisture content as determined by Nev. T101. Over-compaction should be avoided as the increased compactive effort will result in lateral pressures higher than those recommended above. Heavy equipment or other loads should not be allowed in within one-third of the wall height unless planned for in the structural design.

If seismic loading is evaluated in accordance with AASHTO *Standard Specifications for Highway Bridges 16th edition, 1996, Section 3*, we recommend using an acceleration coefficient of 0.4g and a Site Coefficient (S) of 1.2 which is applicable to a Type II Soil Profile.

Additional earth pressures resulting from loads applied at the ground surface must also be included in the design of an earth retaining structure. These earth pressures may be generated by surface surcharge loads, point loads, line loads and strip loads.

5.6 Mechanically Stabilized Earth Walls

Mechanically stabilized earth (MSE) walls should have a minimum reinforcement length of 1.1H and embedment depth of 0.5 m. It is anticipated that soil reinforcement will consist of inextensible strips for ease of construction in order to work around driven pile foundations at the bridge abutments. A minimum distance of 1.5 m should be maintain between the front of the finished MSE wall and the nearest pile foundation to provide a workable space for the installation of soil reinforcement.

The external stability of the earth wall was evaluated assuming a maximum wall height of 6.4 m, a reinforcement length of 7.0 m, and a broken backfill slope at 2:1 (H:V) extending from the back of the MSE wall to a height of 8.5 m. The broken backslope was modeled as an infinite upslope with a 12 degree inclination. For external stability, the wall facing and reinforced backfill were considered to act as a coherent block structure with active forces acting on a vertical plane, 7.9 m high, intersecting the backfill slope. Resistance to sliding was calculated a coefficient of resistance to sliding of 0.40. The external wall stability was evaluated for the potential failure mechanisms of sliding, overturning bearing capacity, and deep seated stability (rotational). The external stability was calculated for static conditions and for seismic conditions assuming a horizontal ground acceleration of 0.2g, and a retained fill unit weight of 19.6 kN/m^3.

The computed factors of safety are included in Table 12 below. Resistance to sliding under seismic conditions dictates the required minimum reinforcement length. Recommended minimum design factors of safety are also provided in Table 12. . The results of our external stability analyses for rotation are presented on Plates 51 and 52 for static and seismic states

TABLE 12

MSE External Factors of Safety (FOS)		
Failure Mechanism	**Calculated Factor of Safety**	**Minimum Factor of Safety**
Sliding	1.9	1.5
Seismic Loading	1.1	1.1
Overturning	7.1	2.0
Seismic Loading	2.8	1.5
Bearing Capacity	5.0	2.5
Seismic Loading	4.4	1.9
Rotational	2.4	1.3
Seismic Loading	1.7	1.0

The internal stability will need to be evaluated by the manufacturer.

All backfill used in the structure volume of the MSE structures shall be free of organic and deleterious matter and conform to the gradation limits as determined by AASHTO T-27.

Sieve Size	Percent Passing
102 mm	100
0.425 mm	0-60
0.075 mm	0-15

The material Plastic Index should not exceed 6.

The material should have the following minimum or maximum electrochemical index properties to limit the corrosion potential of steel reinforcement.

Property	Criteria	Test Method
Resistivity	>3000 ohm-cm	AASHO T-288-91
pH	>5<10	AASHO T-289-91
Chlorides	<100 PPM	AASHO T-291-91
Sulfates	<200 PPM	AASHO T-290-91
Organic Content	1% maximum	AASHO T-267-86

The existing embankment fill does not meet the resistivity requirements listed above. Material resistivity test results are discussed further in Section 5.7. The need for select imported granular fill should be anticipated.

5.7 Steel and Concrete Reactivity

Analytical testing of selected soil samples from structure borings was performed to assess the potential for adverse reactivity with concrete and corrosivity with steel. Soluble sulfate tests were performed to evaluate potential sulfate attack against Portland Cement Concrete. Soluble sulfate contents were observed to be less than 0.15 percent. Therefore, the potential for sulfate attack appears to be minor and conventional Type II cement may be used according to data furnished by Cement Industry Technical Committee of California and Acculabs Inc.

Resistivity tests are used as an indication of possible steel corrosion activity. Generally, the lower the native resistivity of the soils, the more likely galvanic currents may occur and corrosion result. Resistivity values for the site soils are on the order of 790 to 940 ohm-cm; therefore, appear to be severely corrosive when in contact with metal. Carbon steel corrosion rates are anticipated to be on the order of 12 μm/yr. per exposed side. Assuming a design life of 100 years for the proposed bridge structure and a uniform loss model, steel pipe and H-piles will have a sacrificial wall thickness of less than 1.5 mm where driven foundations are in contact with existing materials. We recommend a sacrificial wall thickness of 1.5 mm be used in determining the required pipe pile wall thickness. Consideration should be given to epoxy coating steel reinforcing, since this material could be exposed to additional corrosive potential from de-icing salts during the winter months. The coating should be a minimum of 450 μm thick.

6 ADDITIONAL SERVICES

6.1 Project Bid Documents

It has been our experience during the bidding process, that contractors often contact us to discuss the geotechnical aspects of the project. Informal contacts between Kleinfelder and an individual contractor could result in incorrect or incomplete information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to BRG Engineering. After consultation with Kleinfelder, BRG Engineering (or representative) should provide clarifications or additional information to all contractors bidding the job.

6.2 Construction Observation/Testing and Plan Review

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation and earthwork.
- Observation of footing excavations.
- Observation and testing of construction materials.
- Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

The review of plans and specifications and the field observation and testing by Kleinfelder are an integral part of the conclusions and recommendations made in this report. If we are not retained for these services, the Client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction.

7 LIMITATIONS

Recommendations contained in this report are based on our field explorations, laboratory tests, and our understanding of the proposed construction. The study was performed using a mutually agreed upon scope of work. It is our opinion that this study was a cost-effective method to evaluate the subject site and evaluate some of the potential geotechnical concerns. More detailed, focused, and/or thorough investigations can be conducted. Further studies will tend to increase the level of assurance; however, such efforts will result in increased costs. If the Client wishes to reduce the uncertainties beyond the level associated with this study, Kleinfelder should be contacted for additional consultation.

The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in soils exist between the points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site, which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to our recommendations. In addition, if the scope of the proposed project, locations of structures, or structural loads change from the description given in this report, our firm should be notified.

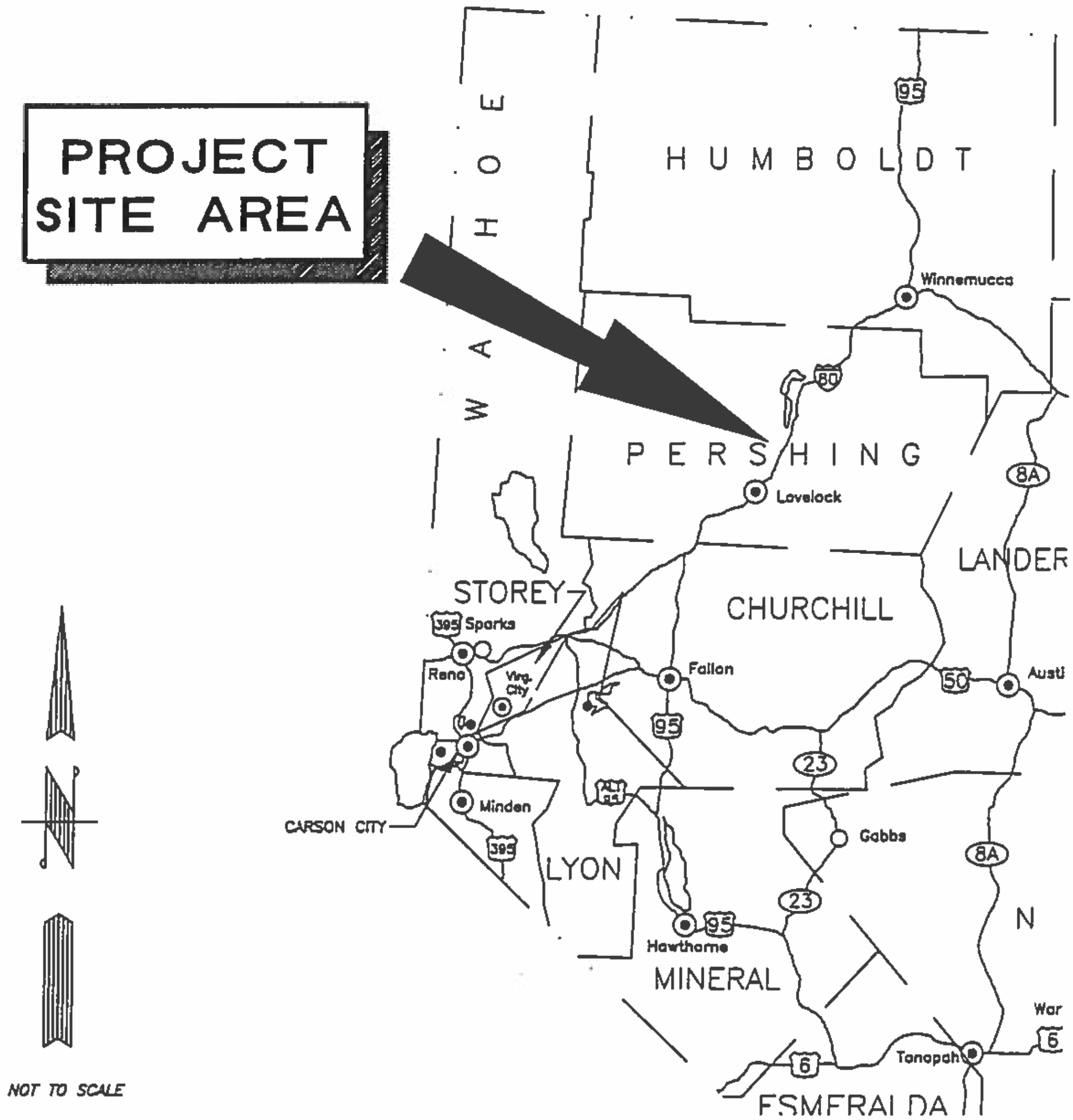
This report has been prepared for design purposes for specific application to the Bridge G-29 Replacement Project in accordance with the generally accepted standards of practice at the time the report was written. No warranty, express or implied, is made.

This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site), or other factors including advances in man's understanding of applied science may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issue. Kleinfelder should be notified if the project is delayed by more than 24 months from the date of this report so that a review of site conditions can be made, and recommendations revised if appropriate.

It is the CLIENT'S responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk. Any party other than the Client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

APPENDIX A

Site Plans
Boring Logs
Laboratory Test Results



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

VICINITY MAP

B-29 BRIDGE REPLACEMENT

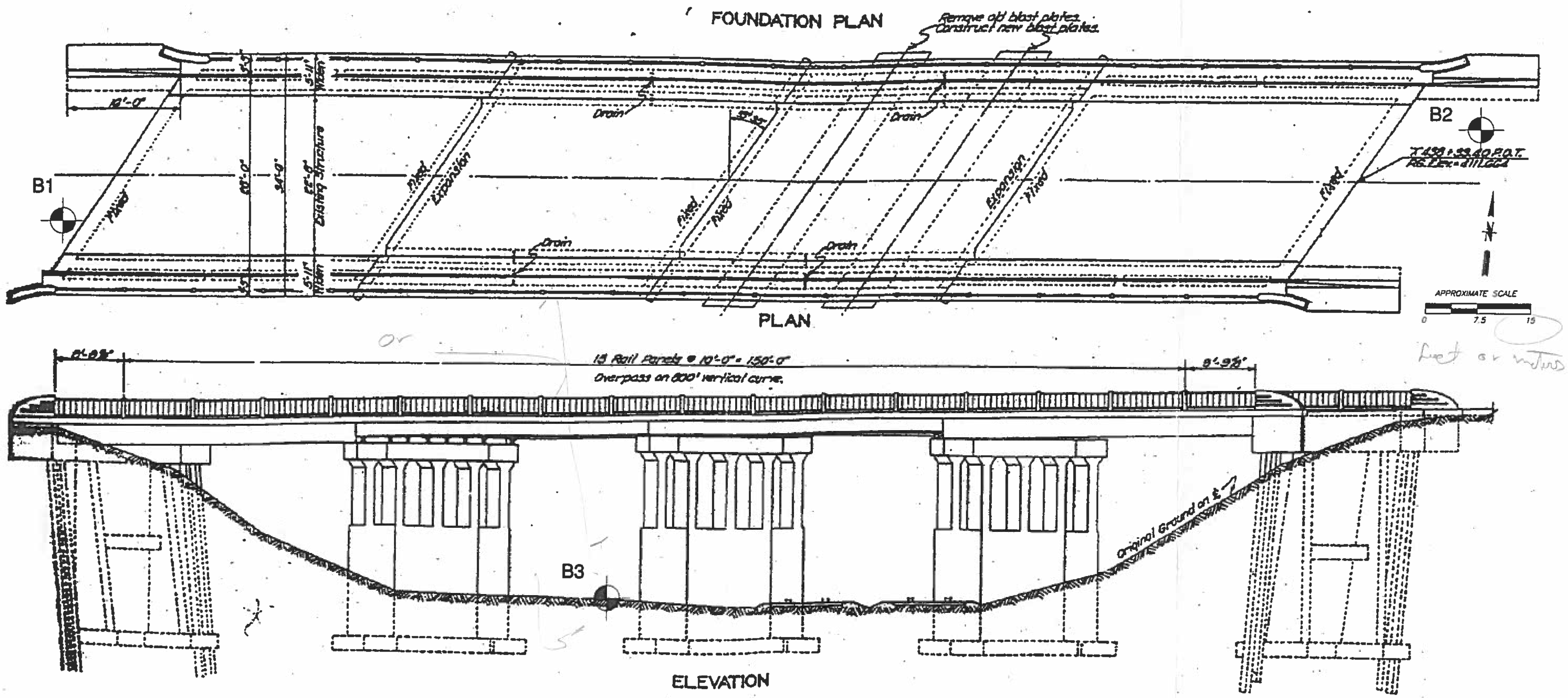
LOVELOCK, NEVADA

PLATE

1

CAD FILE: L:\1999\DRAFTING\30275901\30275910-vicinity.dwg

CAD FILE: L:\2000\DRAFTING\30275901\30275901-site.dwg



LEGEND

B3 — APPROXIMATE SAMPLE / BORING LOCATIONS

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01.001

SITE PLAN
B-29 BRIDGE REPLACEMENT
LOVELOCK, NEVADA

PLATE 2

Hammer Drop ...

Depth in Feet	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Soil Description
0					6" Asphalt
					5" Aggregate Base
2					**DARK BROWN SANDY GRAVEL (GP)** fill, dry, dense
5			42		**OLIVE SILTY SAND (SP-SM)** fill, dry, loose, no cementation, fine grained sand with some gravel up to 1", non-plastic fines
10	86	6.6	3	9	
15	87	7.3	6	13	Slightly more silt
20	90	6.2	11	7	
25			9	9	Slightly moist
30	102	5.4	19		
31					**OLIVE CLAY (CH)** moist, very stiff, no cementation, fine grained with some coarse gravel up to 1" in diameter, approximately 75% high plasticity fines, clay cube crumbles easily
35			19	100	Increasing fines
40			50/3"		Becoming hard

DATE DRILLED: 12-2-99
TOTAL DEPTH: 76.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4112 feet

KLEINFELDER

PROJECT NO. 30-2759-01 January 2000

B-29 Bridge Replacement
Lovelock, Nevada

LOG OF BORING B1

PLATE 3

Depth (ft)	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
46	82	35.2	23	99			Very stiff, mottling, calcium veins, clay cube breaks easily
48							
50	81	38.2	41				Becoming hard
52							
54			52/10"				
56							
58							
60			45/9"				
62							
64			50+				
66							
68							
70			50+				
72							
74			50+				
76							No free water encountered
78							
80							
82							
84							
86							
88							
90							

DEPTH IN FEET

DATE DRILLED: 12-2-99
TOTAL DEPTH: 76.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4112 feet

KLEINFELDER

PROJECT NO. 30-2759-01 January 2000

B-29 Bridge Replacement

Lovelock, Nevada

LOG OF BORING B1

PLATE

3

Hammer Drop sys

Electric

Depth (ft)	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
0							6" Asphalt
							5" Aggregate Base
2							**DARK BROWN SANDY GRAVEL (GP)** fill, dry, dense
5			16				**OLIVE SILTY SAND (SM-SP)** fill, dry, medium dense, no cementation, fine grained sand with some gravel up to 1"
10			15				
15			12				Slightly more silt
20	96	6.0	25				Slightly moist
25	91	5.1	32	1			Decreasing fines
30			50+				Increasing fines
35			50+				**OLIVE CLAY (CH)** moist, hard, no cementation, trace fine grained sand with some coarse gravel up to 1" in diameter
40	85	30.2	50+	76			Mottling (minor) and calcium veins

DEPTH IN FEET

DATE DRILLED: 12-2-99
TOTAL DEPTH: 76.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4112 feet

KLEINFELDER

B-29 Bridge Replacement
Lovelock, Nevada

PLATE 4

PROJECT NO. 30-2759-01 January 2000

LOG OF BORING B2

DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
46			30				No gravel
48							
50			50+				Increasing mottling
52							
54							
56			50+				
58							
60	71	46	50+				
62							
64							
66			50+				
68							
70			50+				
72							
74							
76			50+				No free water encountered
78							
80							
82							
84							
86							
88							
90							

DATE DRILLED: 12-2-99
TOTAL DEPTH: 76.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4112 feet

KLEINFELDER

PROJECT NO. 30-2759-01 January 2000

B-29 Bridge Replacement
Lovelock, Nevada

LOG OF BORING B2

PLATE
4

metric

Hammer Drop 54??

1 - 2 - 3
1 4

Depth (feet)	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
0							**OLIVE SANDY CLAY (CH)** dry, stiff, no cementation, fine grained sand, approximately 75% fines
5			13				
10	69	50.4	17	100			**OLIVE CLAY (CH)** moist, very stiff, no cementation
15			54				Becoming hard, some gravel up to approximately 1" in diameter.
20	79	42.4	38	99			Increasing fines
25	90	31.8	59				
30			26				Mottling
35			49				Clay cube does not break apart easily
40	78	42.5	45/10"	100			

DEPTH IN FEET

DATE DRILLED: 11-30-99
TOTAL DEPTH: 51.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4081 feet

KLEINFELDER

PROJECT NO. 30-2759-01 January 2000

B-29 Bridge Replacement
Lovelock, Nevada

LOG OF BORING B3

PLATE
5

42 - 50/3 → 92/9"→ 50/2

Depth (ft)	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
46			42/9"				
48							
50			50+				Calcium veins and mottling
52							No free water encountered
54							
56							
58							
60							
62							
64							
66							
68							
70							
72							
74							
76							
78							
80							
82							
84							
86							
88							
90							

DEPTH IN FEET

DATE DRILLED: 11-30-99
TOTAL DEPTH: 51.0 feet
DIAMETER: 6 inch

LOGGED BY: K. PETERSON
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 4081 feet

KLEINFELDER

PROJECT NO. 30-2759-01 January 2000

B-29 Bridge Replacement
Lovelock, Nevada

LOG OF BORING B3

PLATE
5

THE UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOIL More than 50% of the material is LARGER than the No. 200 sieve.	GRAVELS More than 50% of coarse part is LARGER than the No. 4 Sieve.	CLEAN GRAVELS Less than 5% finer than No. 200 Sieve. PI<4	GW	Well graded gravels, gravel – sand mixtures, little or no fines, Cu>4 & 1<Cc>3
		PI>7	GP	Poorly graded gravels or gravel – sand mixtures, little or no fines Cu<4 or 1>Cc<3
		GRAVEL More than 12% finer than No. 200 Sieve.	GM	Silty gravels, gravel – sand – silt mixtures
			GC	Clayey gravels, gravel – sand – clay mixtures
	SANDS More than 50% of coarse part is SMALLER than the No. 4 Sieve.	CLEAN SANDS Less than 5% finer than No. 200 Sieve.	SW	Well graded sands, gravelly sands, little or no or no fines, Cu>6 & 1<Cc>3
			SP	Poorly graded sands or gravelly sands, little or no fines Cu<6 or 1>Cc<3
		SAND More than 12% finer than No. 200 Sieve. PI<4	SM	Silty sands, sand – silt mixtures
		PI>7	SC	Clayey sands, sand – clay mixtures
FINE GRAINED SOIL More than 50% of the material is SMALLER than the No. 200 sieve.	SILTS AND CLAYS Liquid limit LESS than 50	PI–Below A–Line	ML	Inorganic silts, rock flour, or clayey silts of low plasticity
		PI–Above A–Line	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts & organic clays of low plasticity
	SILTS AND CLAYS Liquid limit GREATER than 50	PI–Below A–Line	MH	Inorganic silts, clayey silts, or silts of high plasticity
		PI–Above A–Line	CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat & other highly organic soils

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		Coarse	Fine	Coarse	Medium	Fine		

12" 3" 3/4" #4 #10 #40 #200 0.002 mm

DESCRIPTIVE TERMS USED WITH SOILS

CONSISTENCY & APPARENT DENSITY		
	SILTS & CLAYS	SANDS & GRAVELS
Strongest	Hard	Very Dense
	Very Stiff	Dense
	Stiff	Medium Dense
	Medium Stiff	Loose
Weakest	Soft	Very Loose
	Very Soft	

MOISTURE CONTENT	
Wettest	Wet
	Very Moist
	Moist
	Slightly Moist
Driest	Dry

▽ – Water Level Observed During Exploration

▼ – Water Level Observed After Exploration

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (702) 689–7800

KEY TO SOIL CLASSIFICATION AND TERMS

B–29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 6

PROJECT NO. 30–2759–01

CAD FILE: L:\1999\DRAFTING\30275901\30275901–P6_uscs.dwg

SYMBOLS

Disturbed Bag or Bulk Sample

Standard Penetration Sample
(1-3/8" I.D.)

Modified California (Porter) Sample
(2" I.D.)

Shelby Tube Sample (3" I.D.)

* No Sample Recovery

Water Level Observed During Drilling

Water Level Observed After Drilling

COMMENTS

NOTE: Blow count represents the number of blows required to drive a sampler through the last 12 inches of an 18-inch penetration. A standard 140-pound hammer with a 30-inch free fall is used to drive the sampler.

NOTE: The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings.

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

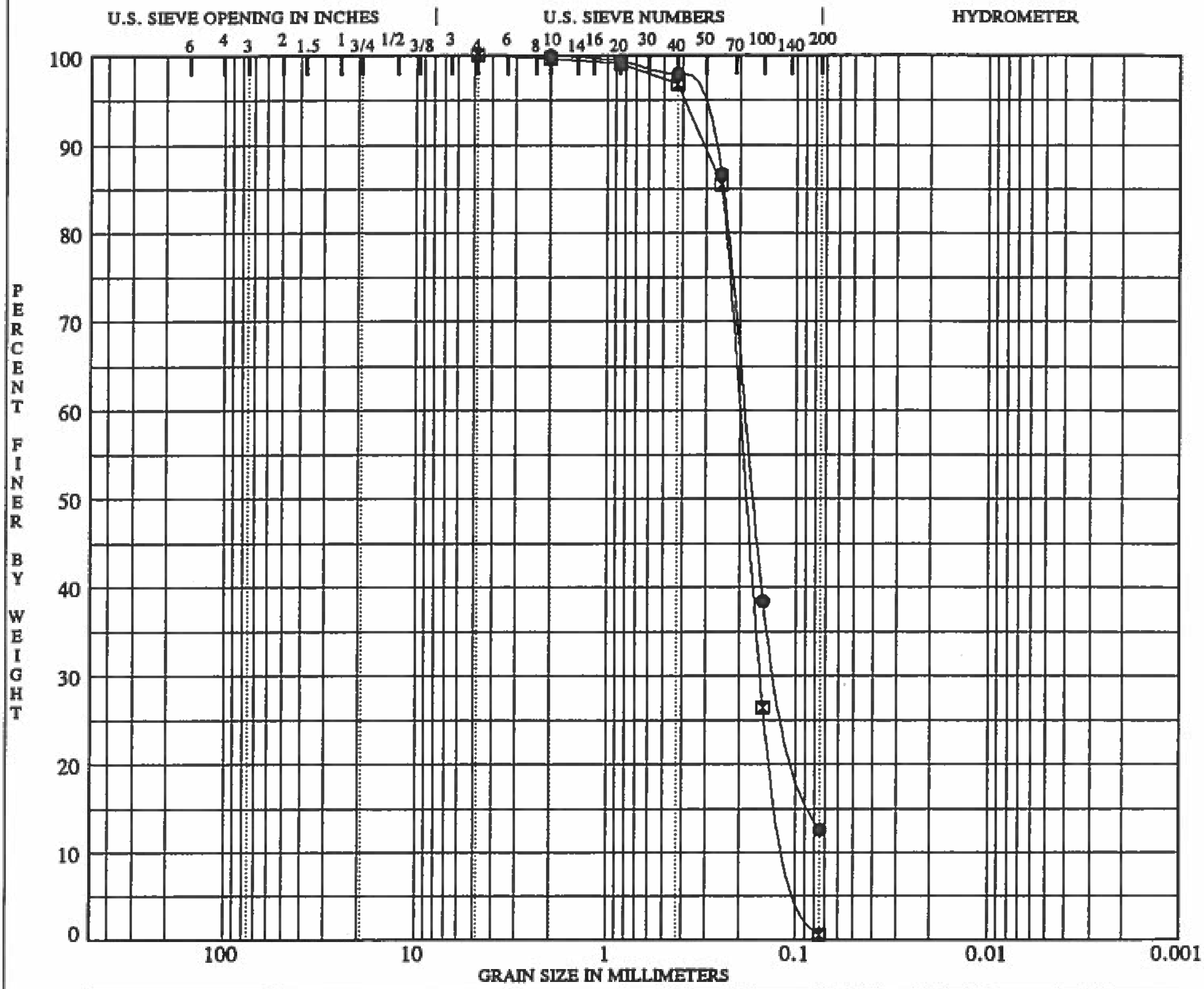
BORING KEY

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 7

CAD FILE: L:\1999\DRAFTING\30275901\30275901-P7.bor.dwg



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring		Depth (ft.)	Description - ASTM Classification	MC%	LL	PL	PI	Cc	Cu
●	B-1	at	16.0	Olive Silty Sand (SM/SP)						
⊠	B-2	at	26.0	Olive Sand (SP)					1.24	2.1

	Boring		Depth (ft.)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	at	16.0	2.00	0.19	0.120		0.0	87.4	12.6	
⊠	B-2	at	26.0	4.75	0.20	0.155	0.0965	0.0	99.4	0.6	

KLEINFELDER

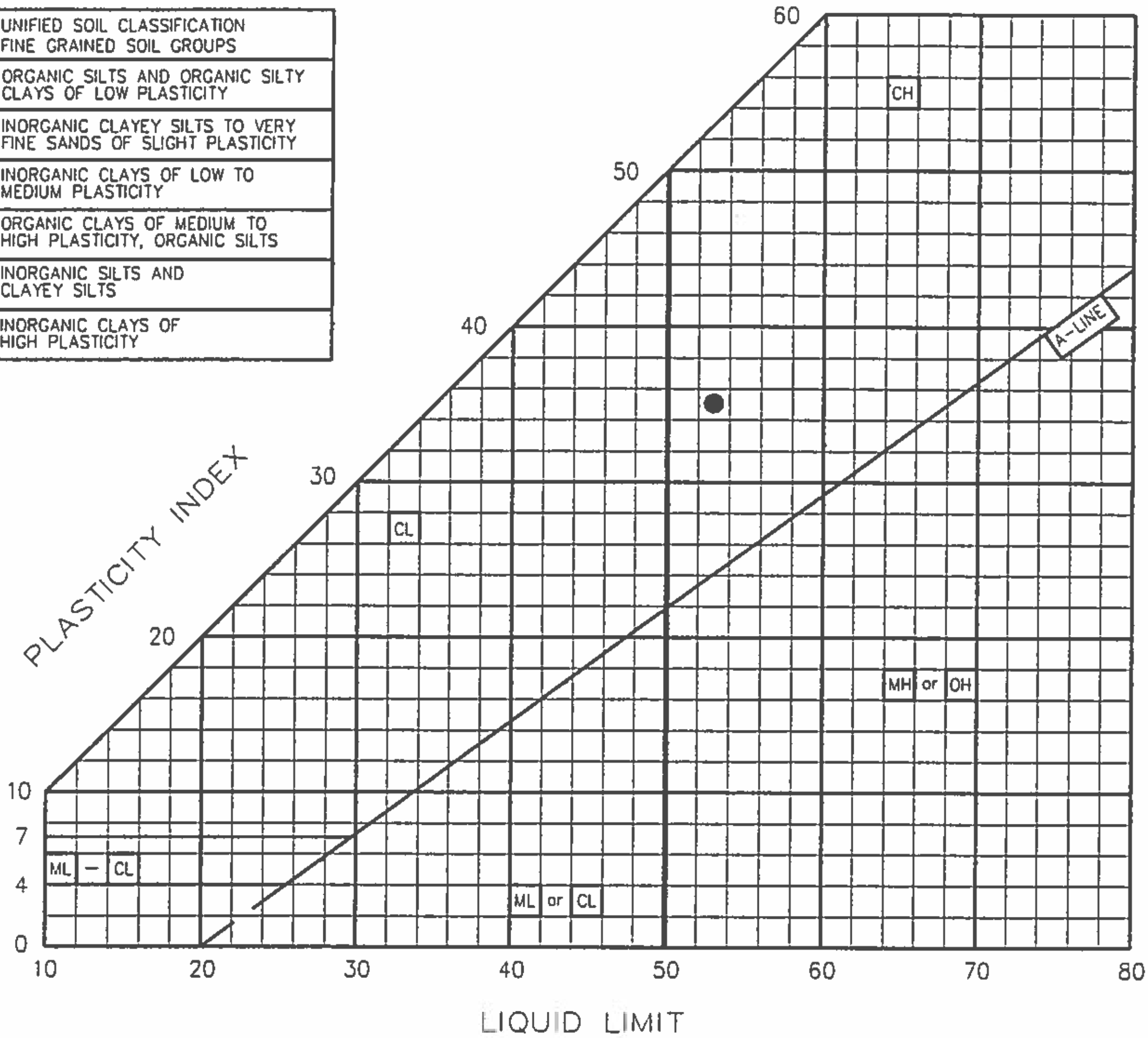
B-29 BRIDGE REPLACEMENT
LOVELOCK, NEVADA

PLATE
8

PROJECT NUMBER: 30-2759-01

GRAIN SIZE ANALYSES

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILTS
CH	INORGANIC CLAYS OF HIGH PLASTICITY



TEST SYMBOL	SAMPLE NO.	SAMPLE (DEPTH)	LIQUID LIMIT	PLASTICITY INDEX	CLASSIFICATION
★	B-1	11'	NA	NP	Olive Silty Sand (SP/SM)
⬟	B-1	36'	137	113	Olive Clay (CH) off chart
●	B-3	11'	53	35	Olive Clay (CH)

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PLASTICITY INDEX

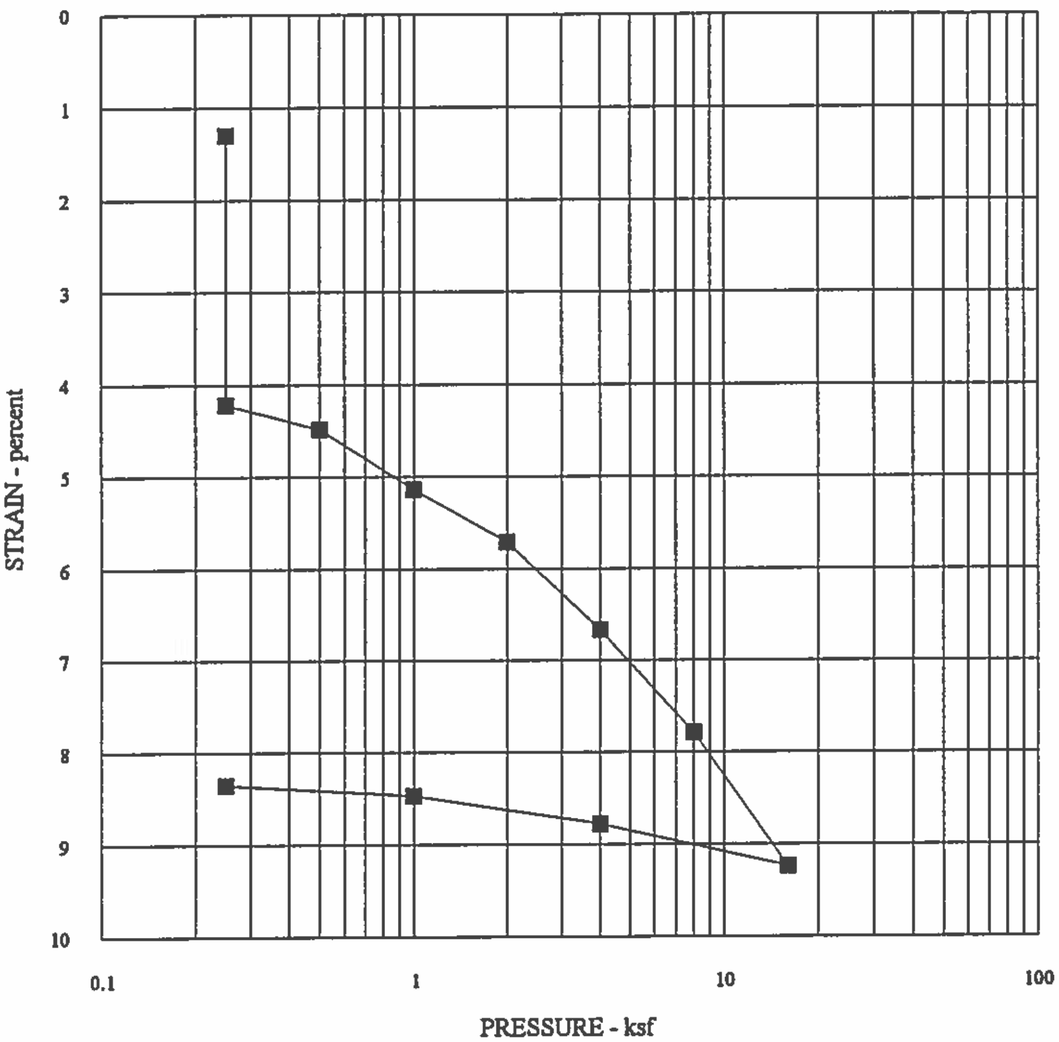
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

9

CAD FILE: L:\1999\DRAFTING\30275901\30275901-P9,pi.dwg



undisturb or remolded (see page 6.)

BORING NO.:	B-1	DEPTH:	21
SAMPLE DESCRIPTION: Olive Silty Sand			
OVERBURDEN PRESSURE, psf:			2000
PRECONSOLIDATION PRESSURE, psf:			2400

	INITIAL	FINAL
DRY DENSITY - pcf	90	98.2
WATER CONTENT - %	6.2	24.0
VOID RATIO	0.7484	0.6161
DEGREE OF SATURATION, %	21.00	99.00
SAMPLE HEIGHT - inches	1.0000	0.9165

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

CONSOLIDATION TEST

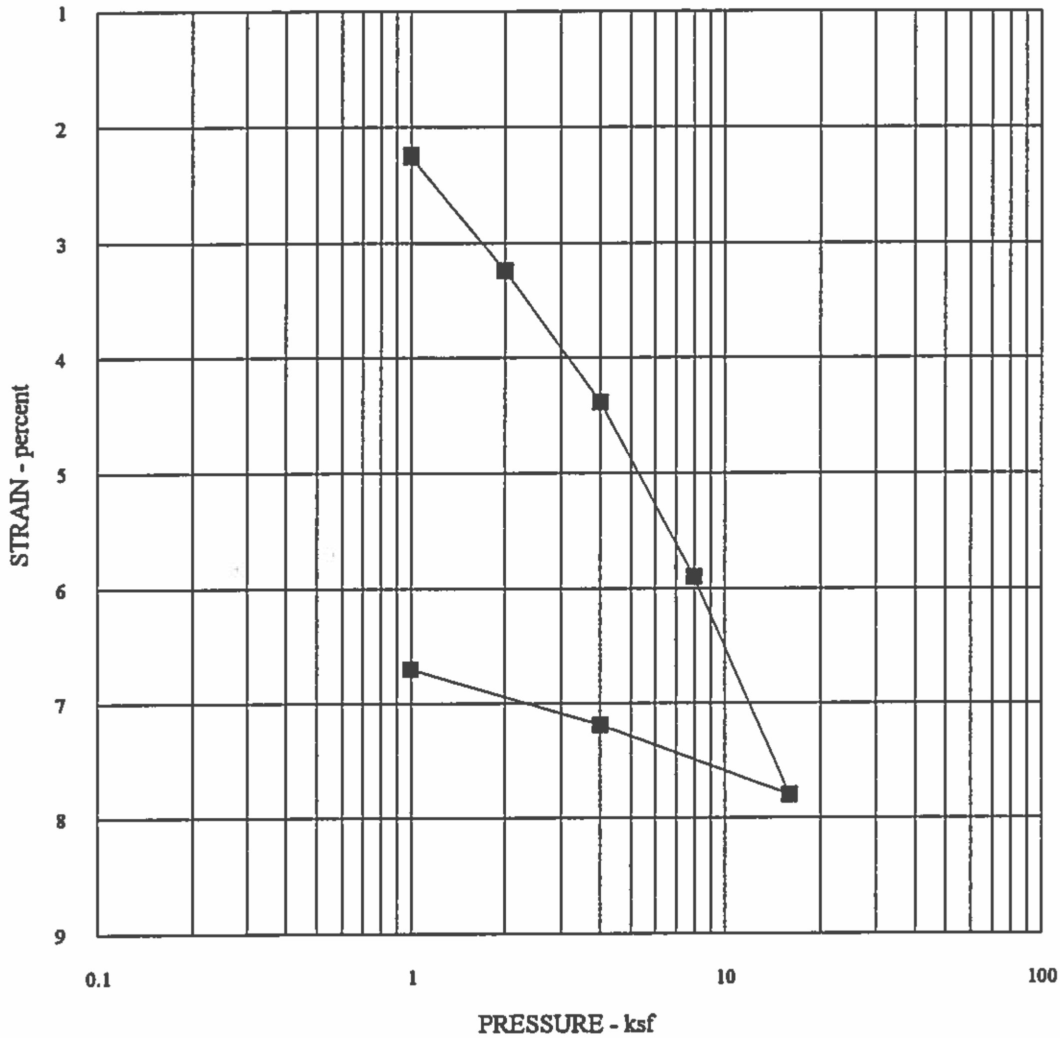
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 10

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P10,P13,consolidation.dwg



BORING NO.: B-2	DEPTH:	41	
SAMPLE DESCRIPTION: Olive Clay			
OVERBURDEN PRESSURE, psf:			4100
PRECONSOLIDATION PRESSURE, psf:			7000

	INITIAL	FINAL
DRY DENSITY - pcf	85.3	91.3
WATER CONTENT - %	30.2	29.2
VOID RATIO	0.8814	0.7572
DEGREE OF SATURATION, %	88.00	99.00
SAMPLE HEIGHT - inches	1.0000	0.933

CAD FILE: L:\1999\30275901\30275901-P10,P13,consolidation.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

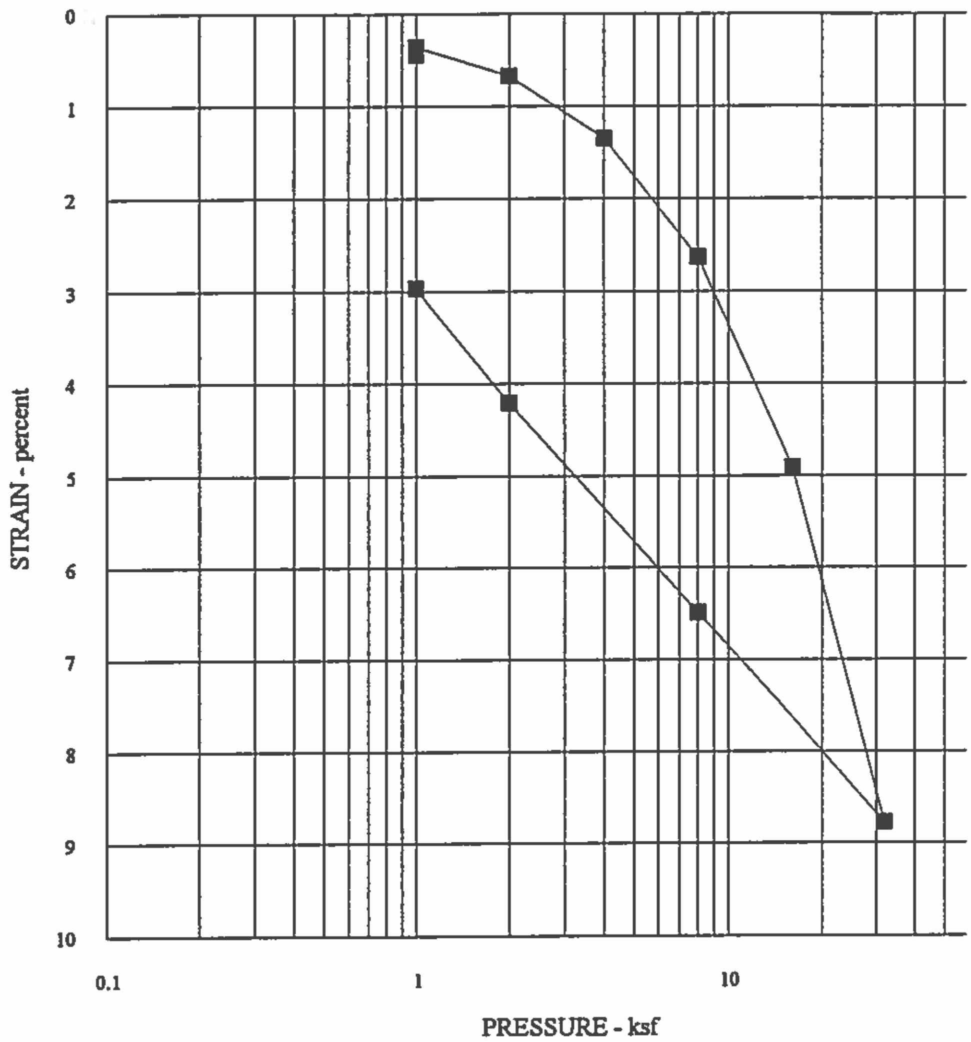
CONSOLIDATION TEST

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

11



BORING NO.: B-3	DEPTH: 20.5
SAMPLE DESCRIPTION: Olive Clay	
OVERBURDEN PRESSURE, psf:	2200
PRECONSOLIDATION PRESSURE, psf:	9000

	Initial	Final
DRY DENSITY - pcf		
WATER CONTENT - %		
VOID RATIO		
DEGREE OF SATURATION, %		
SAMPLE HEIGHT - inches		

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

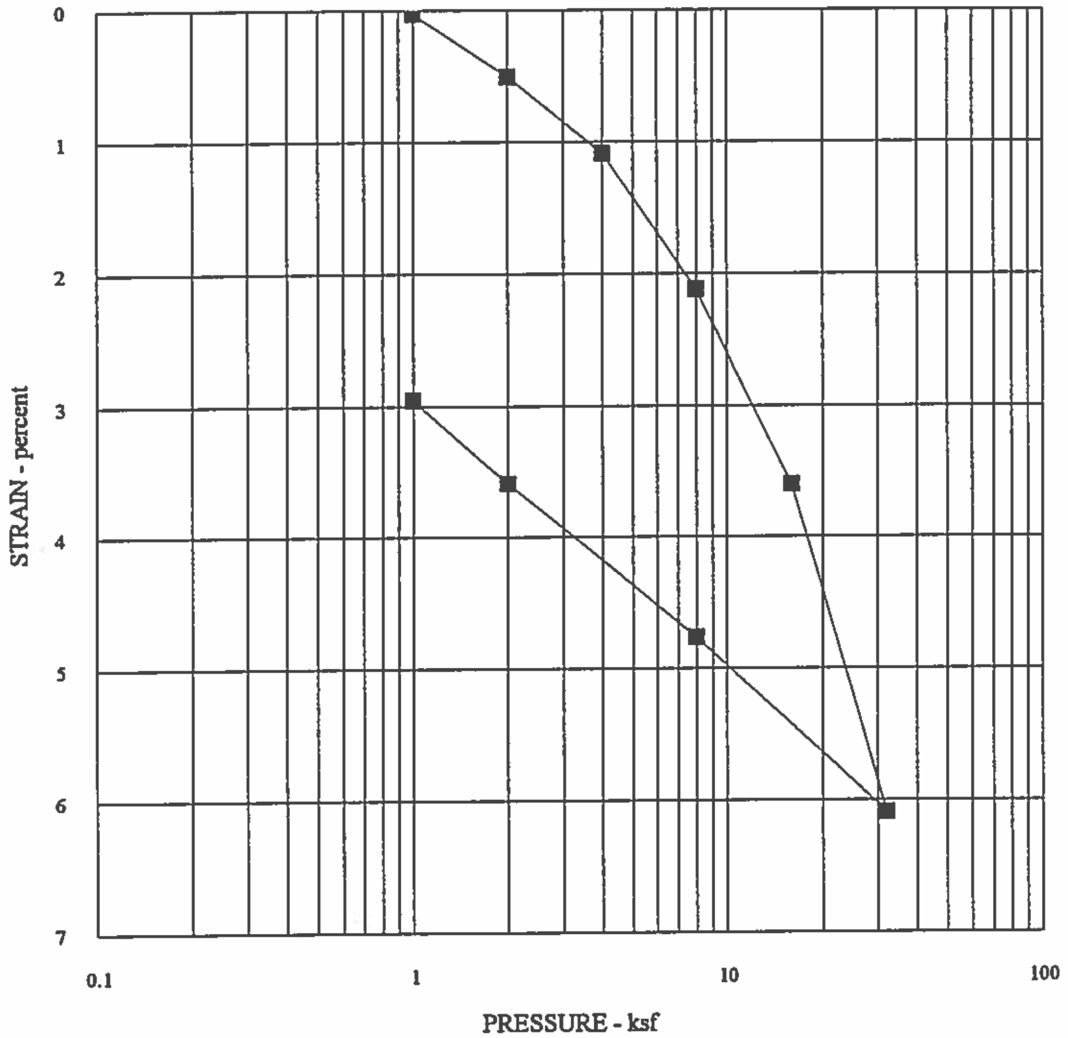
CONSOLIDATION TEST

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 12

CAD FILE: L:\1999\30275901\30275901-P10,P13,consolidation.dwg



BORING NO.: B-3	DEPTH: 40.5
SAMPLE DESCRIPTION: Olive Clay	
OVERBURDEN PRESSURE, psf:	4500
PRECONSOLIDATION PRESSURE, psf:	9500

	INITIAL	FINAL
DRY DENSITY - pcf	78.3	80.5
WATER CONTENT - %	42.5	44.2
VOID RATIO	1.4184	1.3558
DEGREE OF SATURATION, %	91.00	99.00
SAMPLE HEIGHT - inches	1.0000	0.9705

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

CONSOLIDATION TEST

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

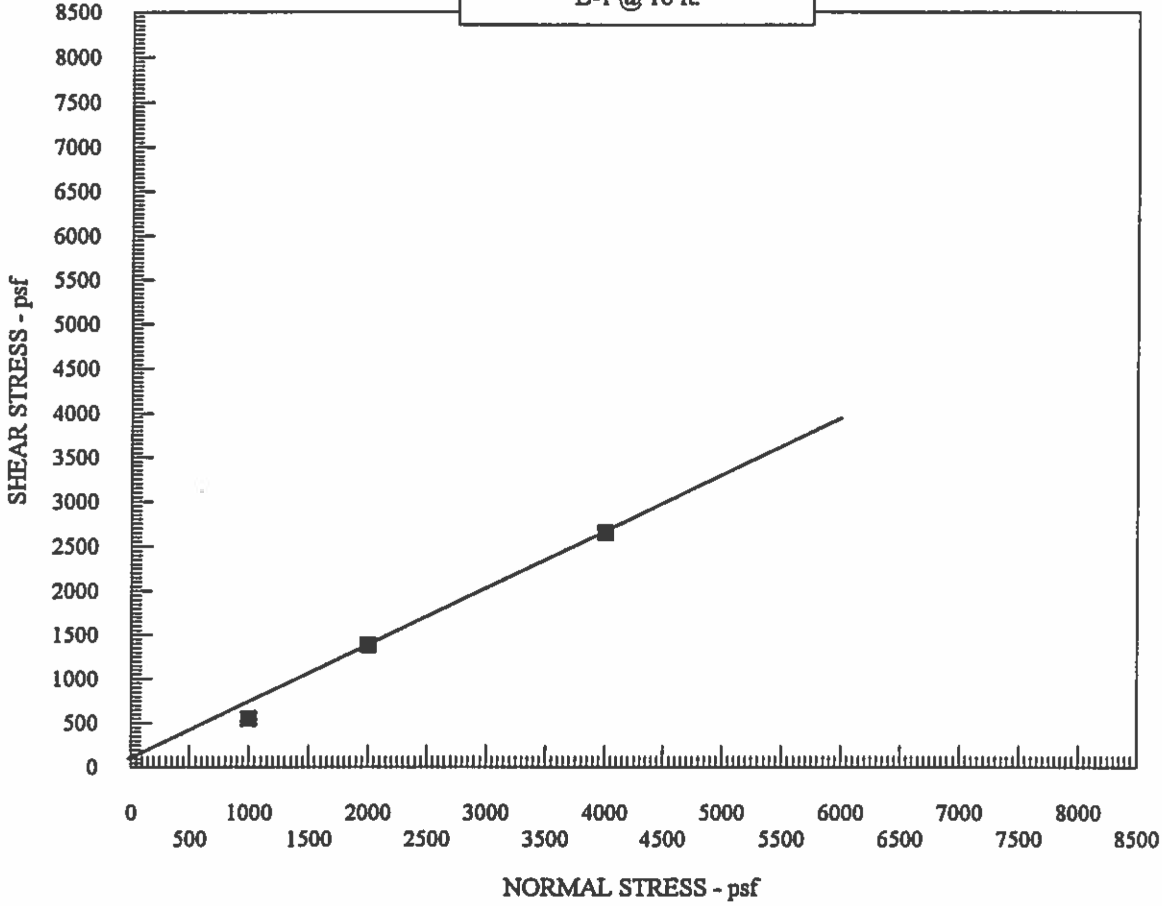
PLATE

13

CAD FILE: L:\1999\30275901\30275901-P10,P13,consolidation.dwg

DIRECT SHEAR

B-1 @ 16 ft.



BRG Engineering

TEST TYPE:	CD/WET/STAGED
BORING NO:	B-1
DEPTH:	16 ft
SOIL DESCRIPTION:	Olive Silty Sand
RATE OF SHEAR:	0.0020 in/min

FRICTION ANGLE:	26 deg.
COHESION:	50 psf

INITIAL DRY DENSITY - pcf			87.3
FINAL DRY DENSITY - pcf			88.9
INITIAL WATER CONTENT - %			7.3
FINAL WATER CONTENT - %			30.4
NORMAL STRESS - psf	1000	2000	4000
MAXIMUM STRESS - psf	553	1388	2652

Undisturbed or remolded

see page 6

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

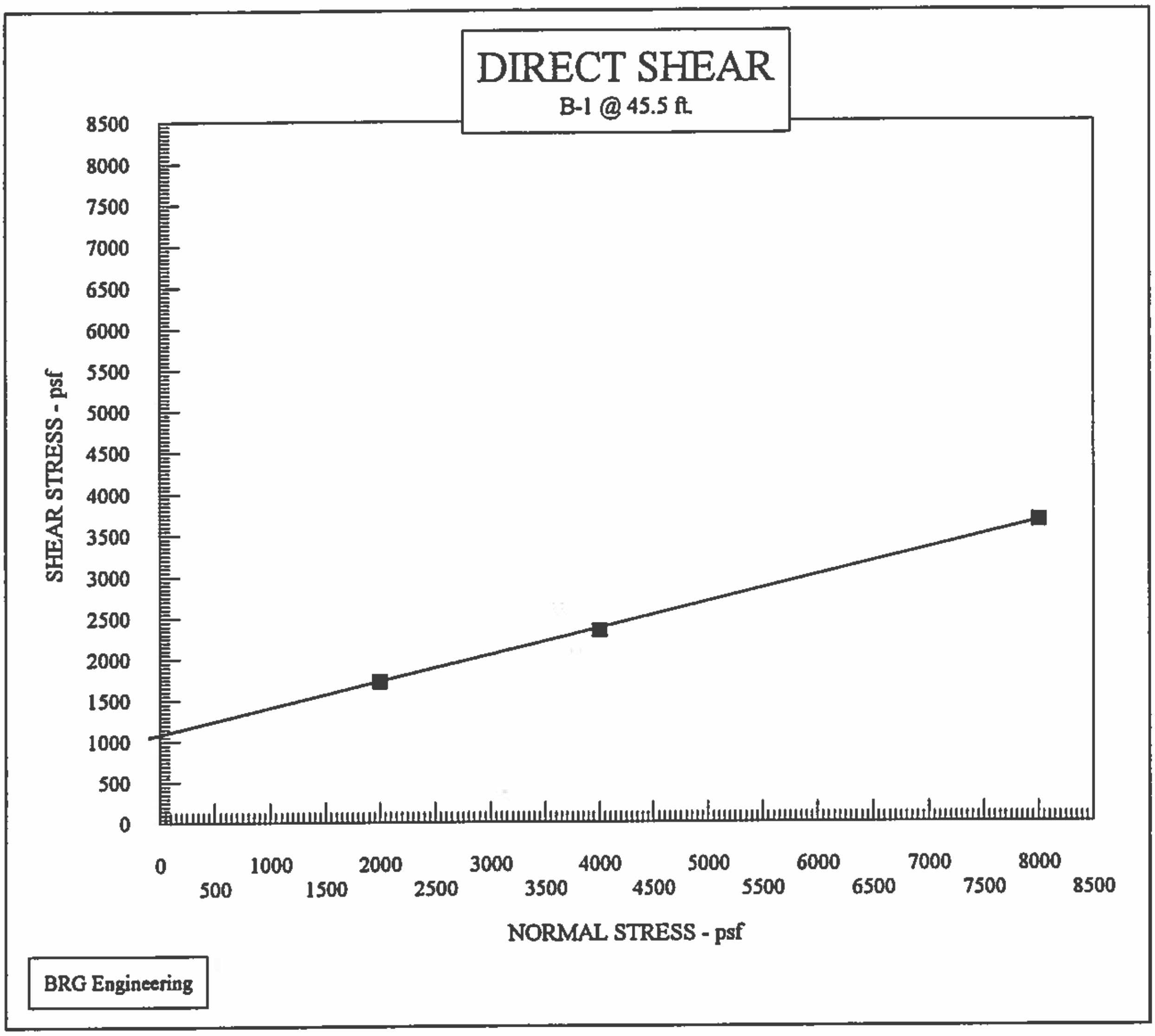
DIRECT SHEAR

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 14

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



TEST TYPE:	CD/WET/STAGED
BORING NO:	B-1
DEPTH:	45.5 ft
SOIL DESCRIPTION:	Olive Clay
RATE OF SHEAR:	0.0020 in/min

FRICTION ANGLE:	14 deg.
COHESION:	1050 psf

INITIAL DRY DENSITY - pcf			82.4
FINAL DRY DENSITY - pcf			76.2
INITIAL WATER CONTENT - %			35.2
FINAL WATER CONTENT - %			51.6
NORMAL STRESS - psf	2000	4000	8000
MAXIMUM STRESS - psf	1724	2342	3667

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

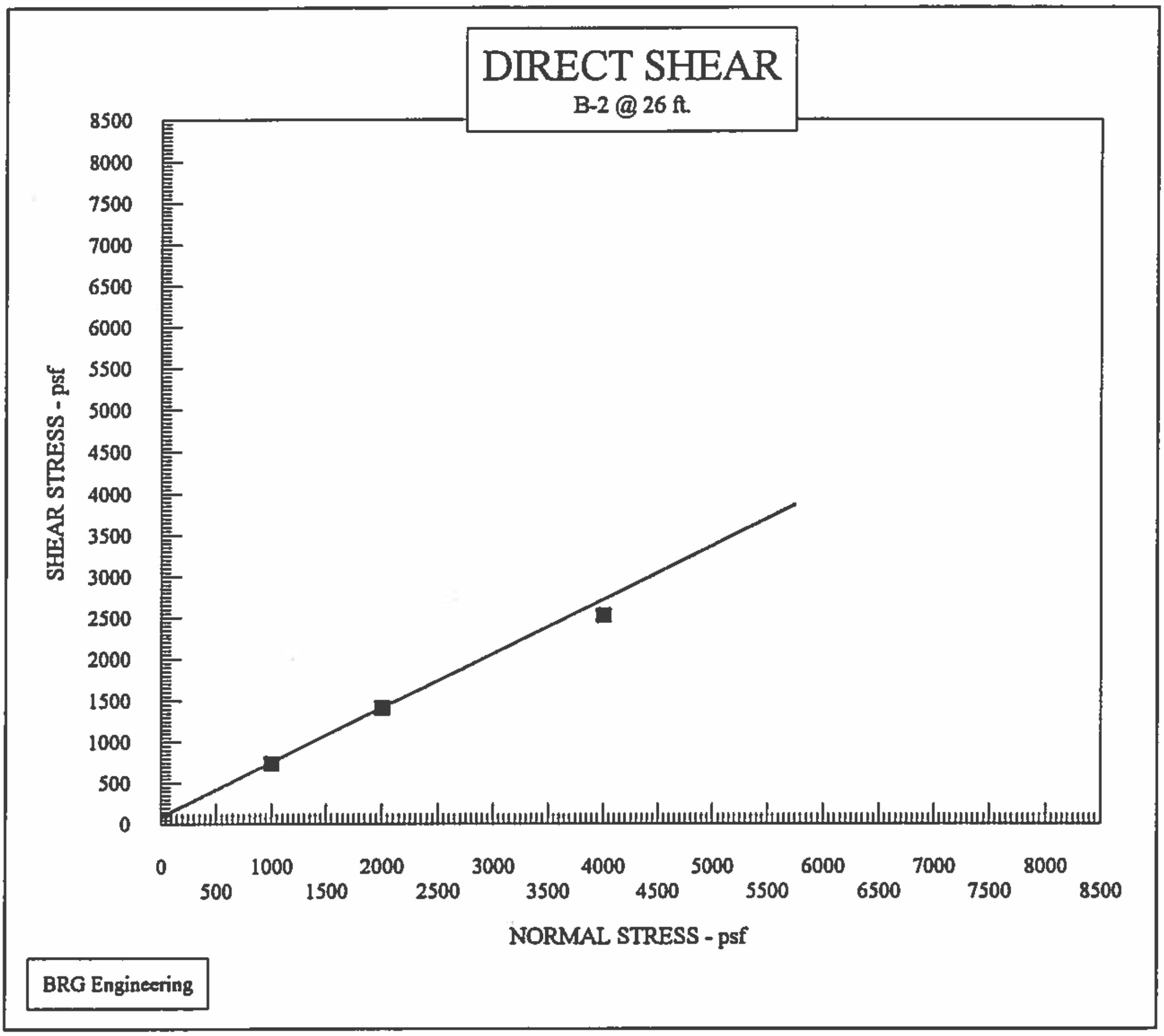
DIRECT SHEAR

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 15

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



TEST TYPE:	CD/WET/STAGED		
BORING NO:	B-2		
DEPTH:	26 ft		
SOIL DESCRIPTION:	Sand		
RATE OF SHEAR:	0.0020 in/min		
INITIAL DRY DENSITY - pcf			90.5
FINAL DRY DENSITY - pcf			94.2
INITIAL WATER CONTENT - %			5.1
FINAL WATER CONTENT - %			27.2
NORMAL STRESS - psf	1000	2000	4000
MAXIMUM STRESS - psf	743	1411	2529

FRICTION ANGLE:	24 deg.
COHESION:	50 psf

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

DIRECT SHEAR

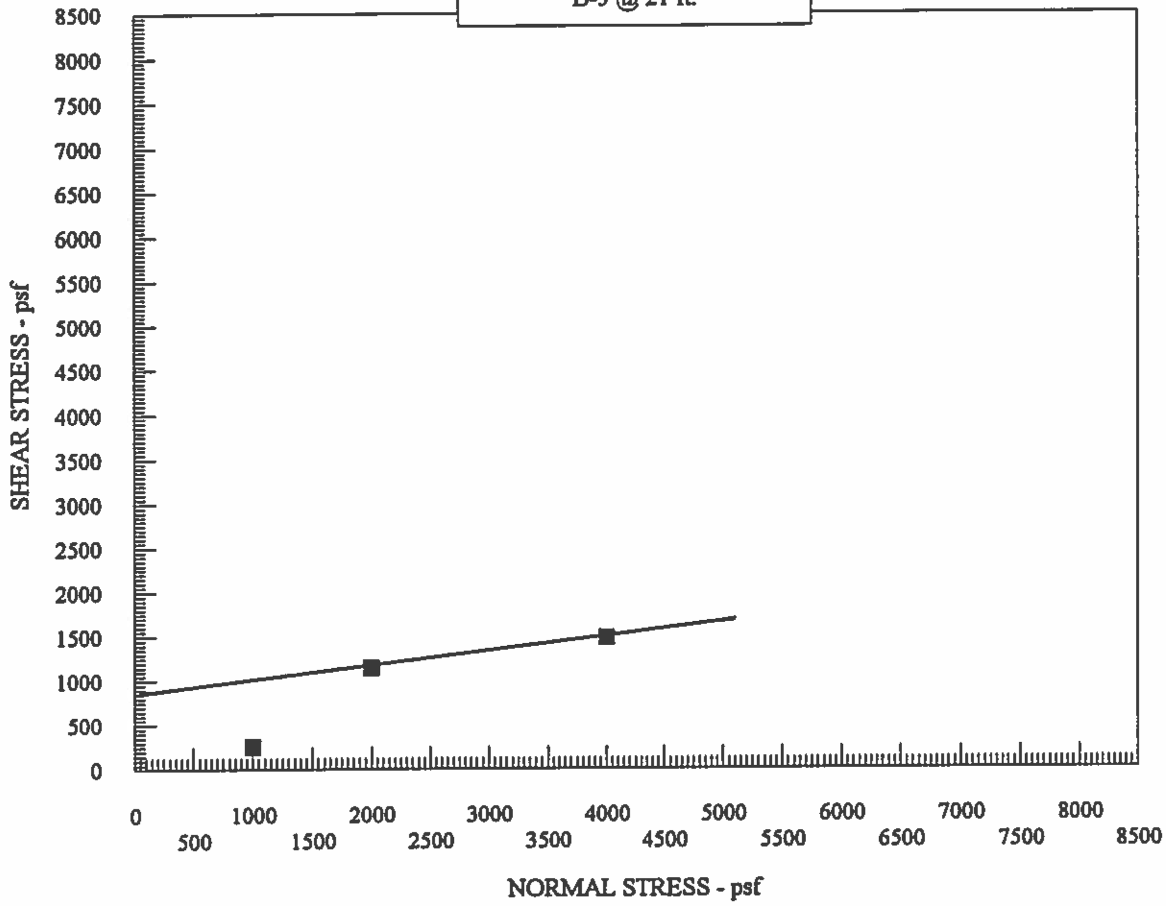
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 16

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

DIRECT SHEAR
B-3 @ 21 ft.



BRG Engineering

TEST TYPE:	CD/WET/STAGED		
BORING NO:	B-3		
DEPTH:	21 ft		
SOIL DESCRIPTION:	Olive Clay		
RATE OF SHEAR:	0.0020 in/min		
INITIAL DRY DENSITY - pcf			69.3
FINAL DRY DENSITY - pcf			75.5
INITIAL WATER CONTENT - %			46.3
FINAL WATER CONTENT - %			54.2
NORMAL STRESS - psf	1000	2000	4000
MAXIMUM STRESS - psf	258	1143	1482

FRICTION ANGLE:	7 deg.
COHESION:	800 psf

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

DIRECT SHEAR

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

17

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

STEEL CORROSION POTENTIAL OF SOILS*

Corrosion Resistance	Resistivity (ohm-cm)
Excellent	6,000 to 10,000
Good	4,500 to 6,000
Fair	2,000 to 4,500
Bad	0 to 2,000

LABORATORY TEST RESULTS

Soil Type	Source	Resistivity (ohm-cm)	pH**
SILTY SAND	B1 @ 6 FT	940	8.60
SANDY SILT	B3 @ 6 FT	790	8.26

* Reference: "Accelerated Corrosion Tests for Buried Metal Structures", by Paul Lieberman, Ph.D., in Pipeline and Gas Journal October, 1996, Pg.51

** Note: Corrosion potential of soils generally increases as pH decreases below 7.

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STEEL CORROSION POTENTIAL OF SOIL

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PROJECT NO. 30-2759-01

PLATE 18

CAD FILE: L:\1999\DRAFTING\30275901\30275901-P18,P19,react-corr.dwg

POTENTIAL REACTIVITY OF SOLUBLE SULFATES IN SOIL OR GROUNDWATER WITH PORTLAND CEMENT CONCRETE

TABLE 1: RECOMMENDATIONS FOR CONCRETE IN SULFATE ENVIRONMENTS *

Soluble Sulfates In Soil %	Sulfates In Water P.P.M.	Cement Type	Maximum Water/Cement Ratio	Minimum Cement Content – Lbs.
0–0.02	0–150	(Negligible................................	........Sulfate................................	........Reaction)
0.02–0.10	150–1000	I or II	0.55	470
0.10–0.20	1000–2000	II	0.50	560
0.20–1.50	2000–15,000	II	0.45	660
		V	0.50	560
Over 1.50	Over 15,000	V	0.45	660

* NOTE A. Concrete for piling and other concrete in sea water environments may contain Type II cement when the water–cement ratio is a maximum of 0.50 or the cement factor is a minimum of 560 pounds. The sulfate concentration in Table I should govern in all cases.

* NOTE B. Sewage treatment facilities normally are constructed using Type II cement except in areas where high sulfate soils of waters exist (See Table I). In sewage, sulfides rather than where sulfates are formed. The sulfide combining with water in the presence of oxygen, can produce sulfuric acid to which no Portland cement is time resistant. Under these conditions, plastic liners, or coatings, are generally used. Closed tanks normally contain an atmosphere of methane rather than oxygen, so acid attack would not be likely to occur. Good quality concretes containing Type II cement with a maximum water cement ratio of 0.53 have provided excellent service in Los Angeles City and County sanitary treatment facilities.

Under special conditions, a concrete materials engineer should be consulted.

Reference: "Recommended Practice to Minimuze Attack on Concrete by Sulfate Soils and Water" by Cement Industry Technical Committee of California.

SAMPLE IDENTIFICATION	B1 @ 6 FT	B3 @ 6 FT		
SAMPLE DESCRIPTION	OLIVE SILTY SAND	OLIVE SANDY SILT		
SOLUBLE SULFATE (%)	0.13	0.052		
SOLUBLE SULFATES (PPM)	--	--		
COMMENTS	LOW SULFATE REACTION	LOW SULFATE REACTION		

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

POTENTIAL REACTIVITY

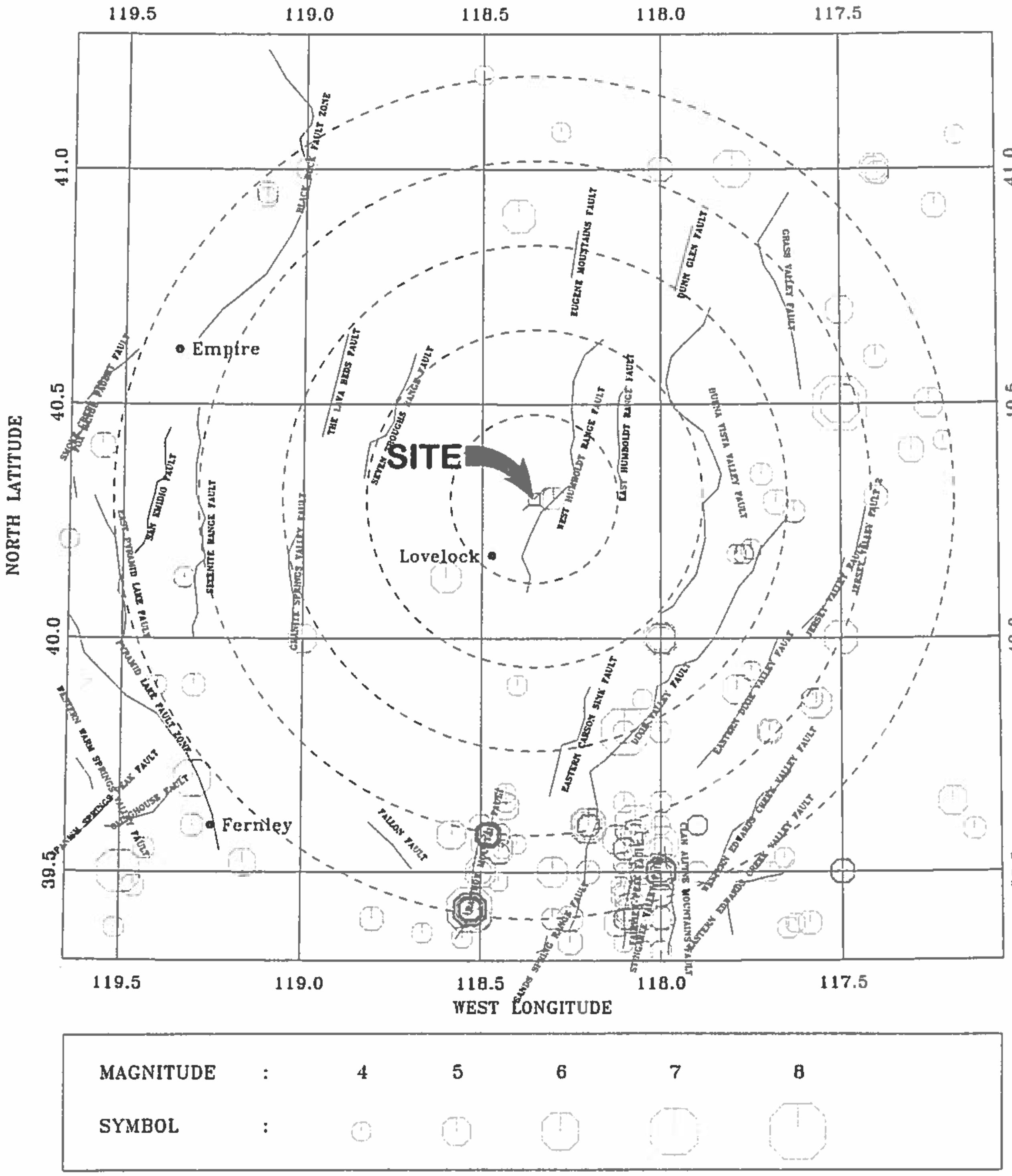
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE 19

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\DRAFTING\30275901/30275901-P18,P19,react-corr.DWG



RADIUS OF LARGEST CIRCLE IS 100 KM

KLEINFELDER

DRAFTED BY: L. Sue	DATE: 1-7-00
CHECKED BY: Z. Zafir	DATE: 1-7-00

REGIONAL FAULT AND SEISMICITY MAP (1800 - DECEMBER 1999)

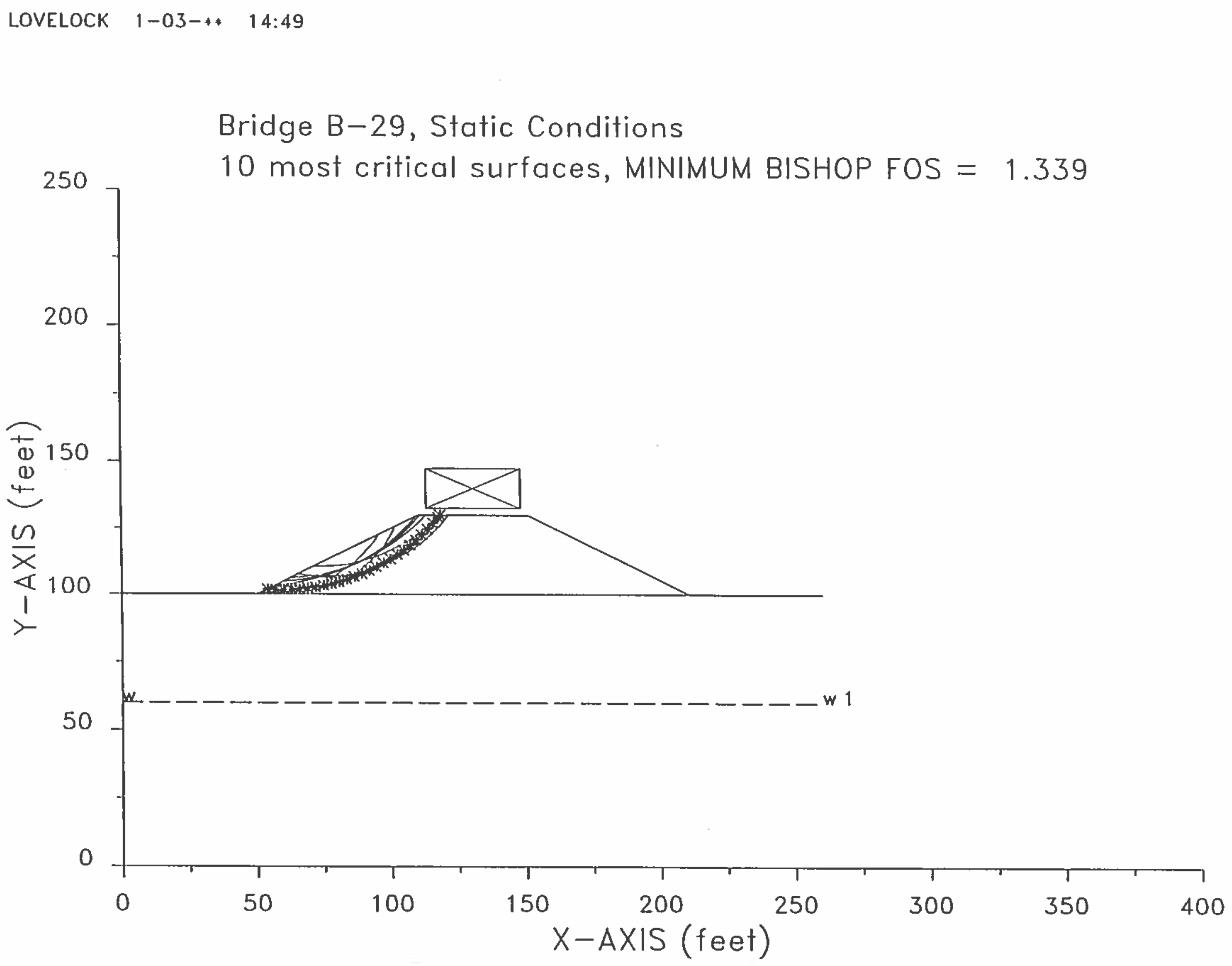
BRIDGE B-29 REPLACEMENT
LOVELOCK, NEVADA

PROJECT NO. 30-2759-01.001

PLATE
20

CAD FILE: 1999\DRAFTING\30275901\FAULTMAP.dwg

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

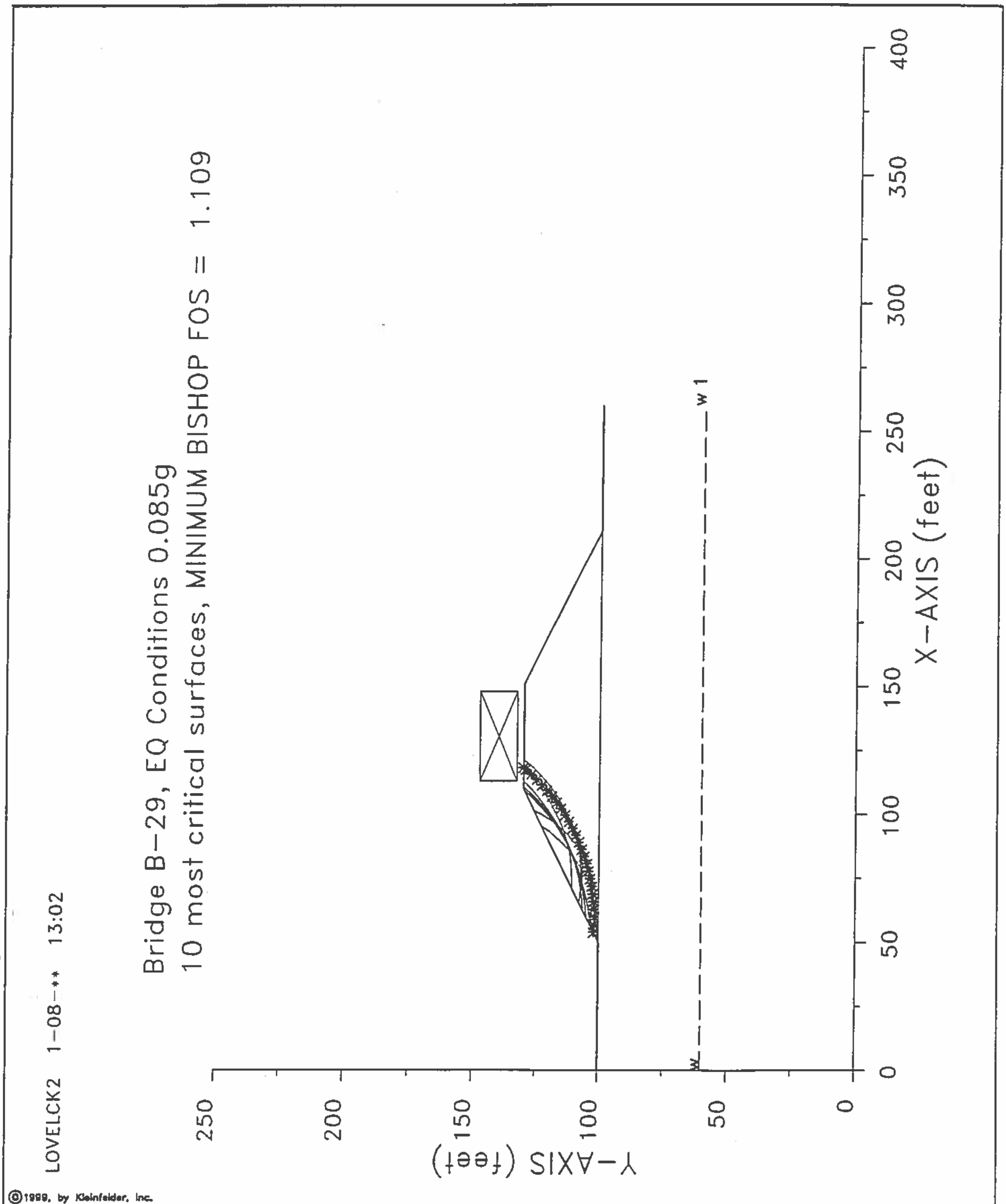
STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

21



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

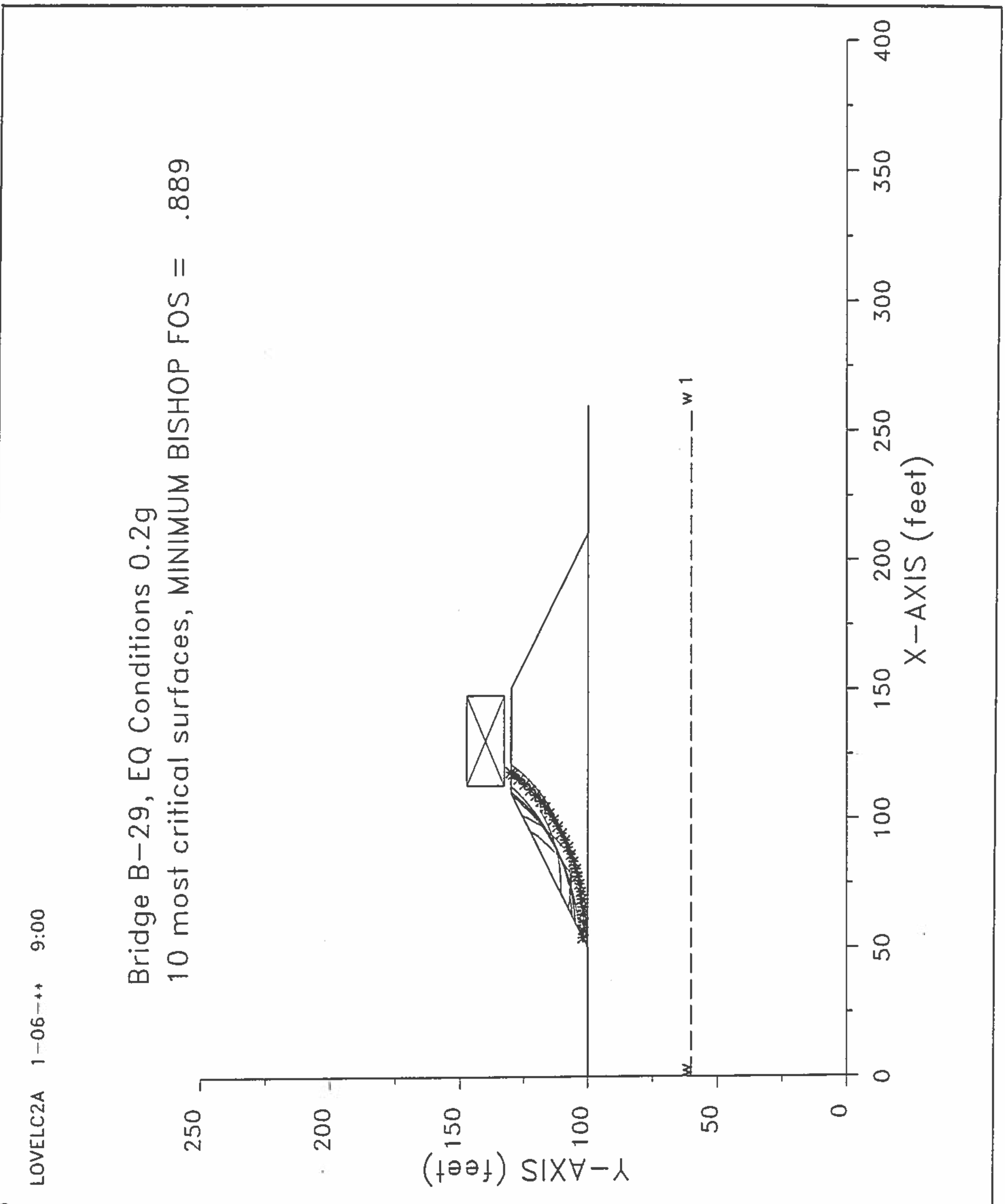
LOVELOCK, NEVADA

PLATE

22

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

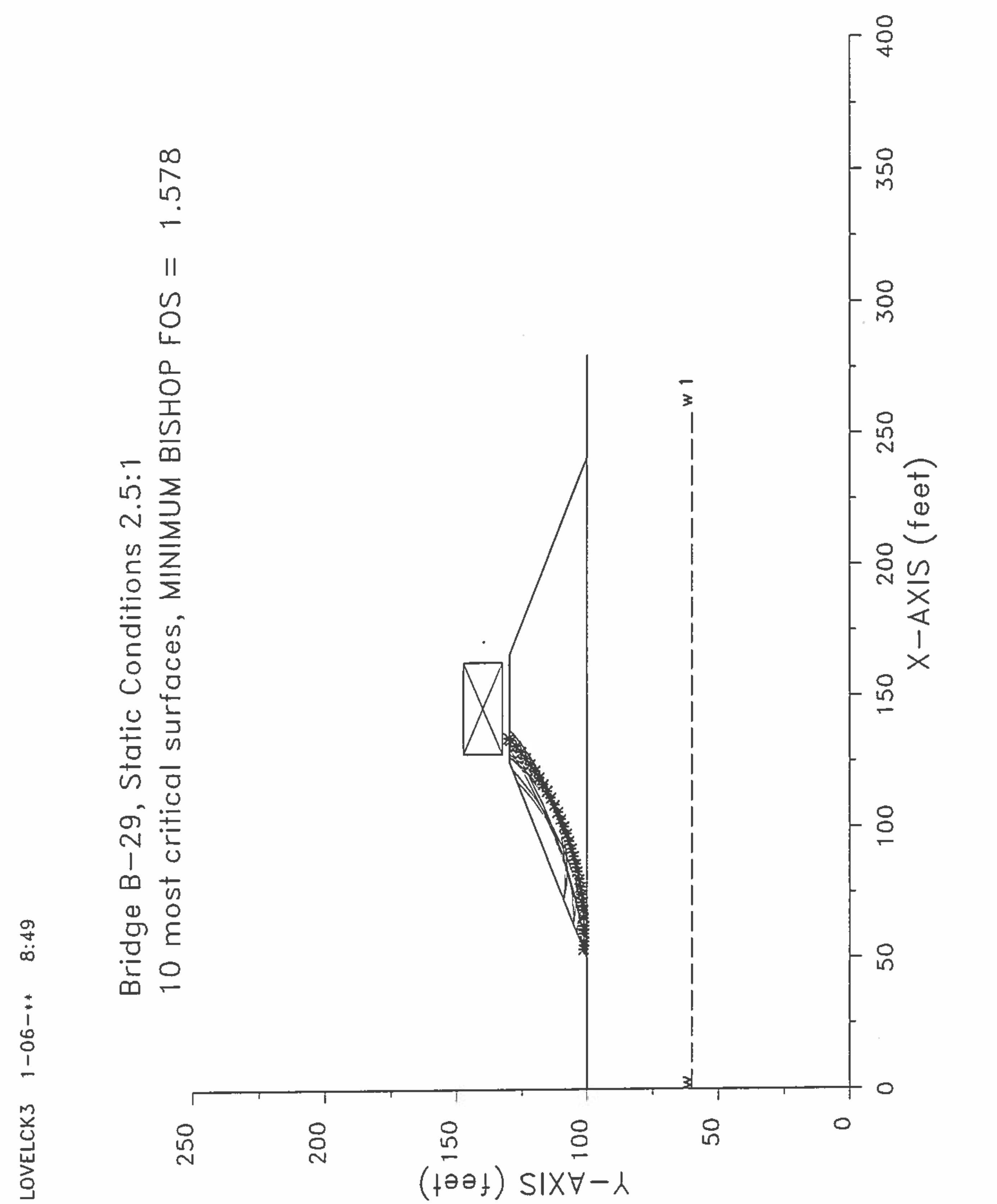
LOVELOCK, NEVADA

PLATE

23

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

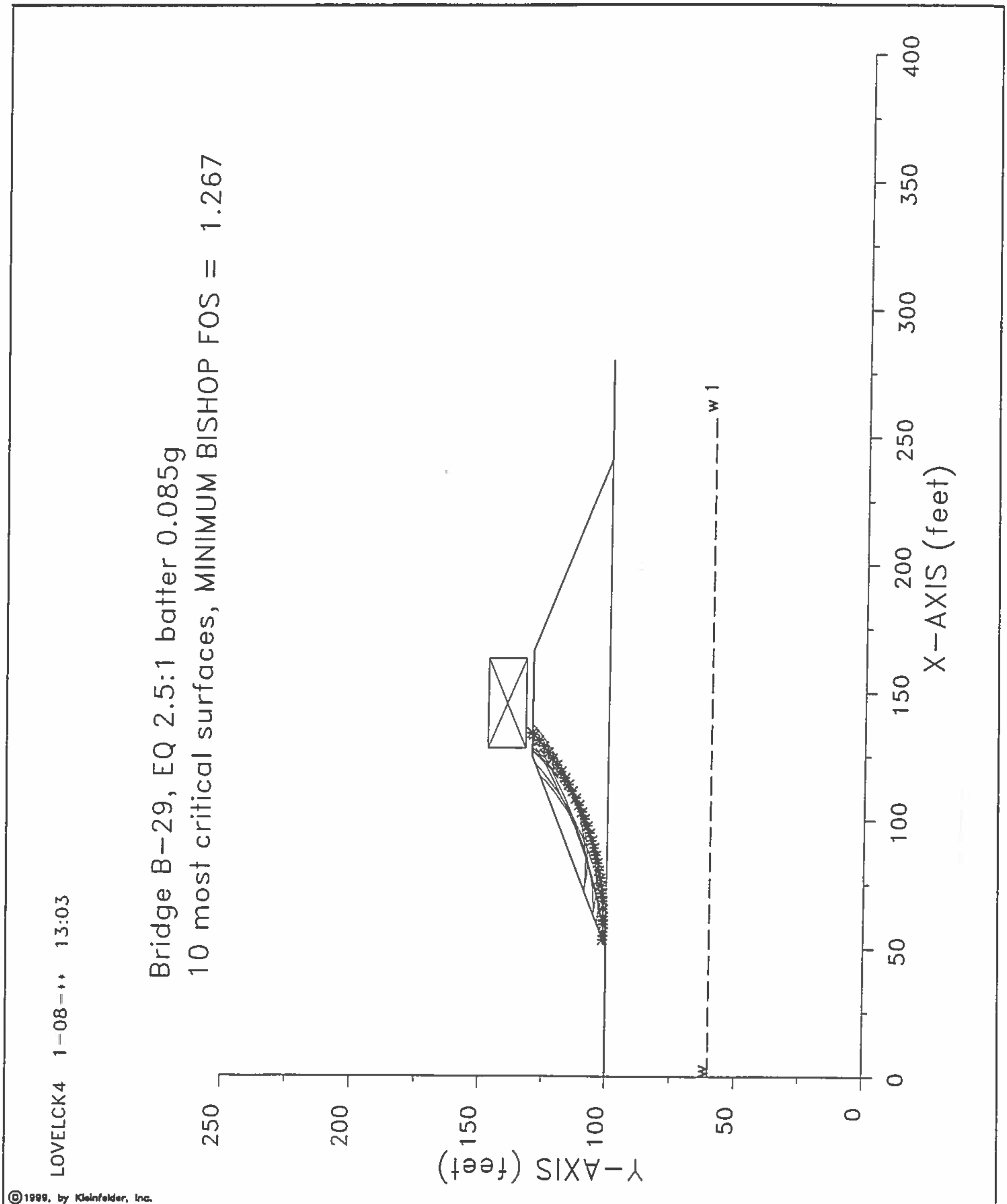
LOVELOCK, NEVADA

PLATE

24

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

STABILITY ANALYSIS

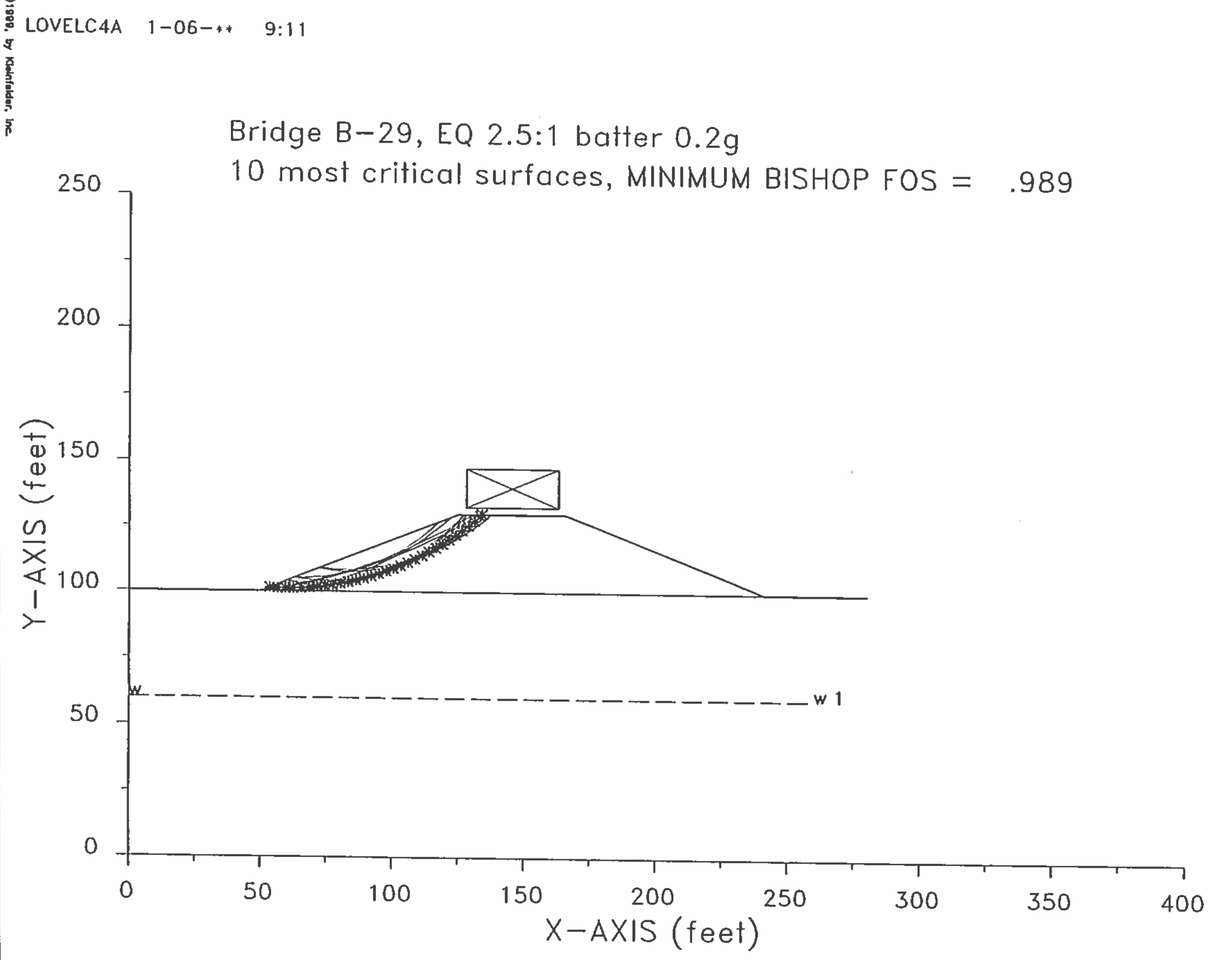
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

25

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

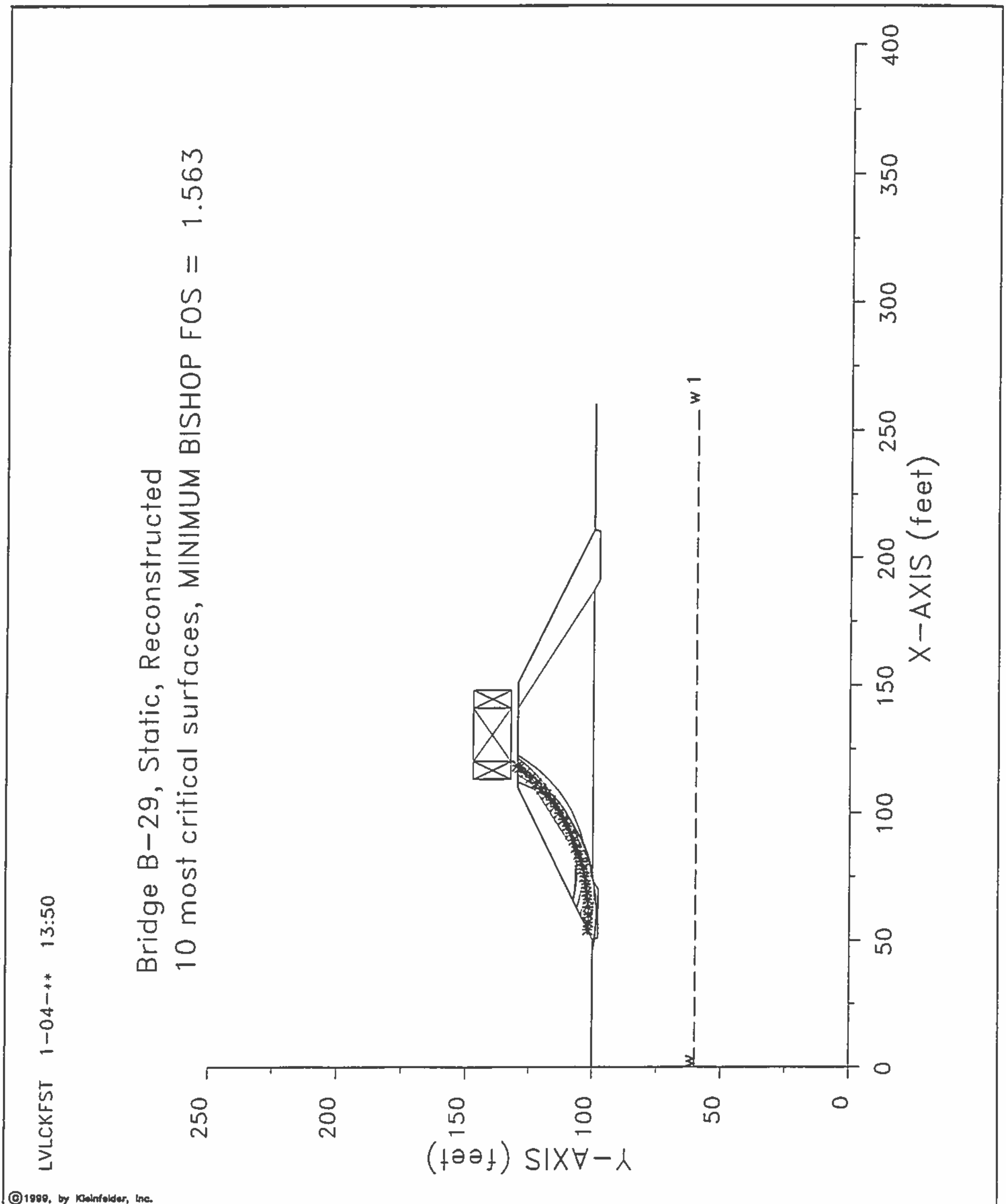
STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

26



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

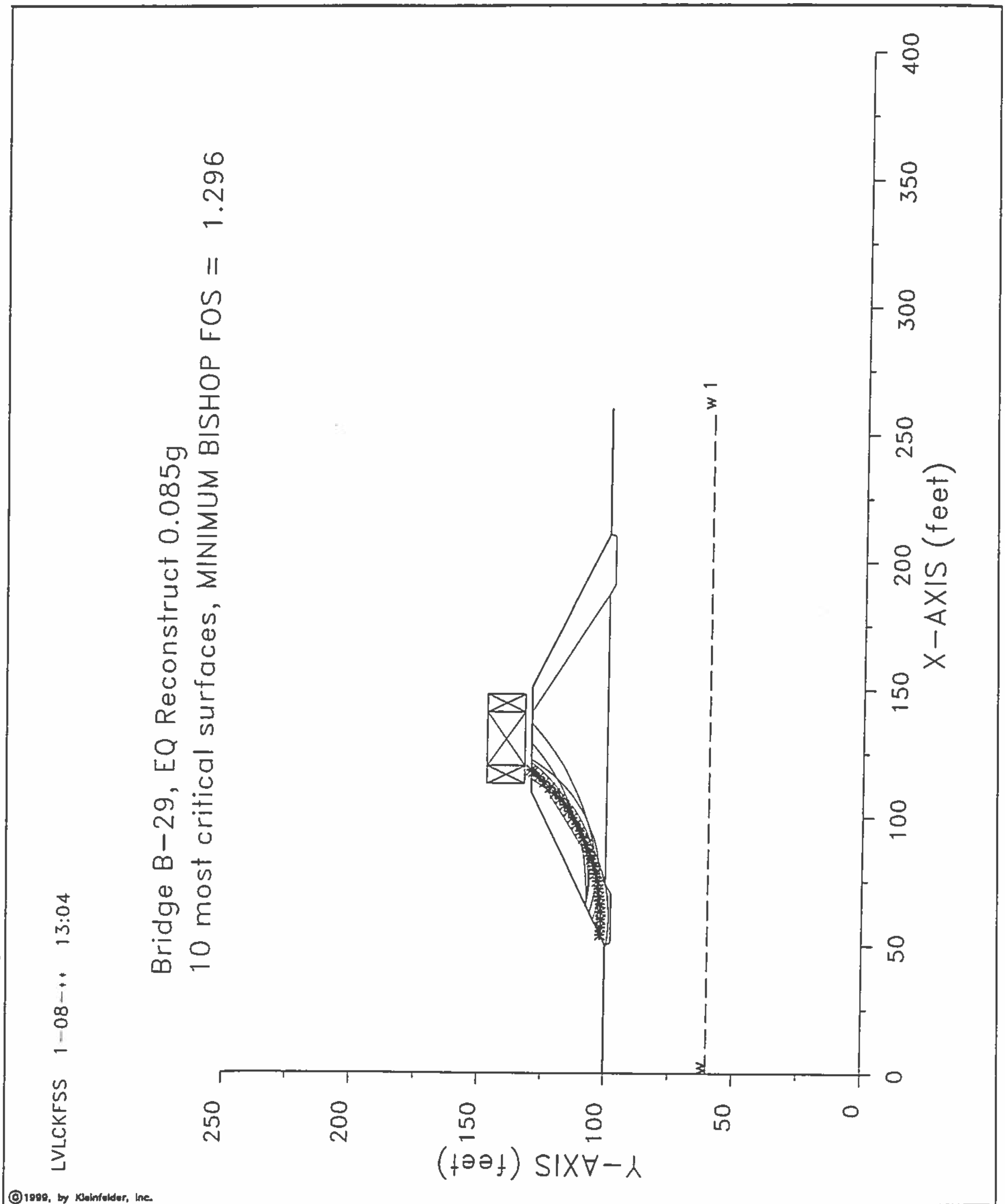
LOVELOCK, NEVADA

PLATE

27

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

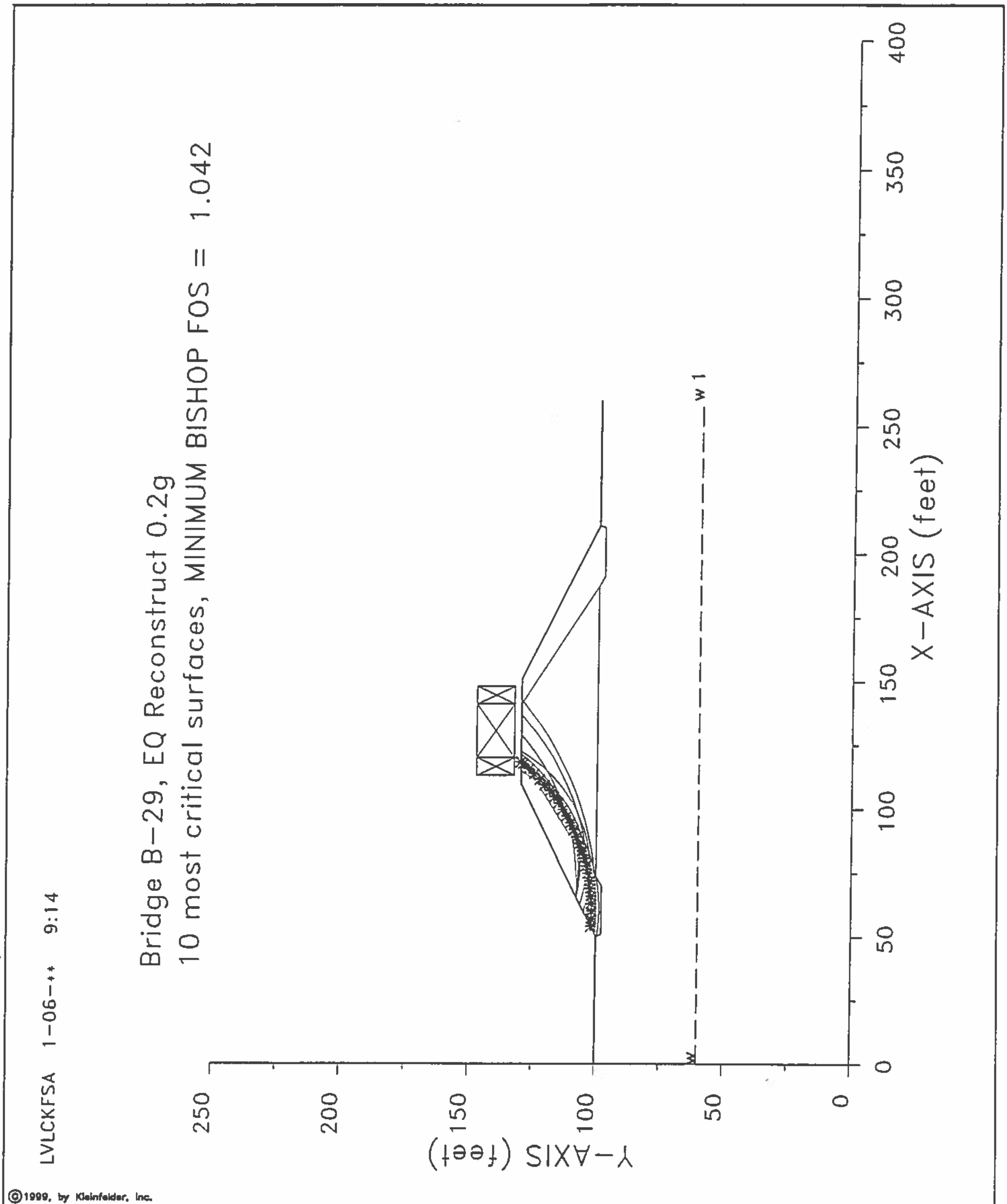
STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

28



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

STABILITY ANALYSIS

B-29 BRIDGE REPLACEMENT

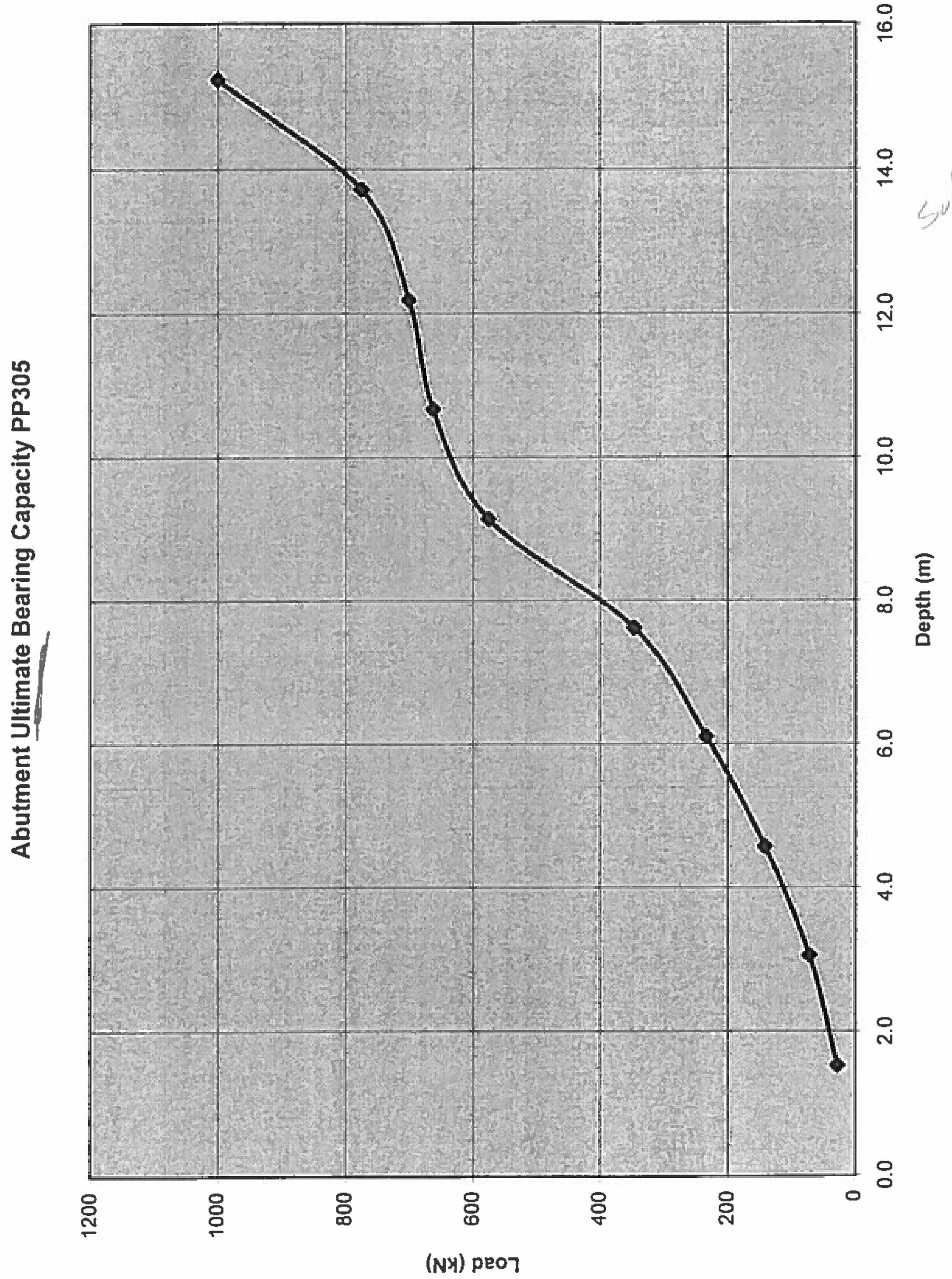
LOVELOCK, NEVADA

PROJECT NO. 30-2759-01

PLATE

29

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



Submit calcs.

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

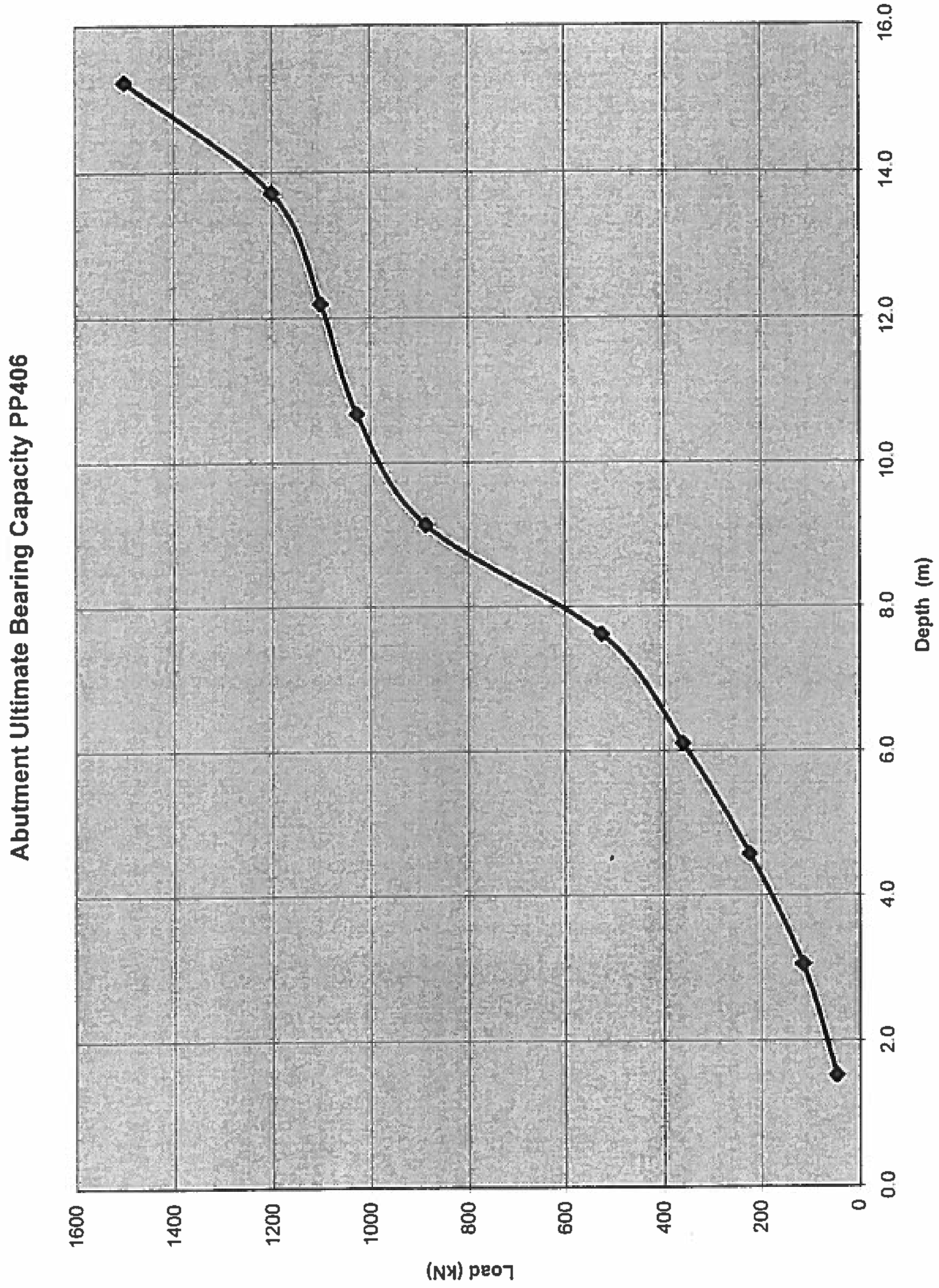
PILE ULTIMATE BEARING CAPACITY

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

30



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PILE ULTIMATE BEARING CAPACITY

B-29 BRIDGE REPLACEMENT

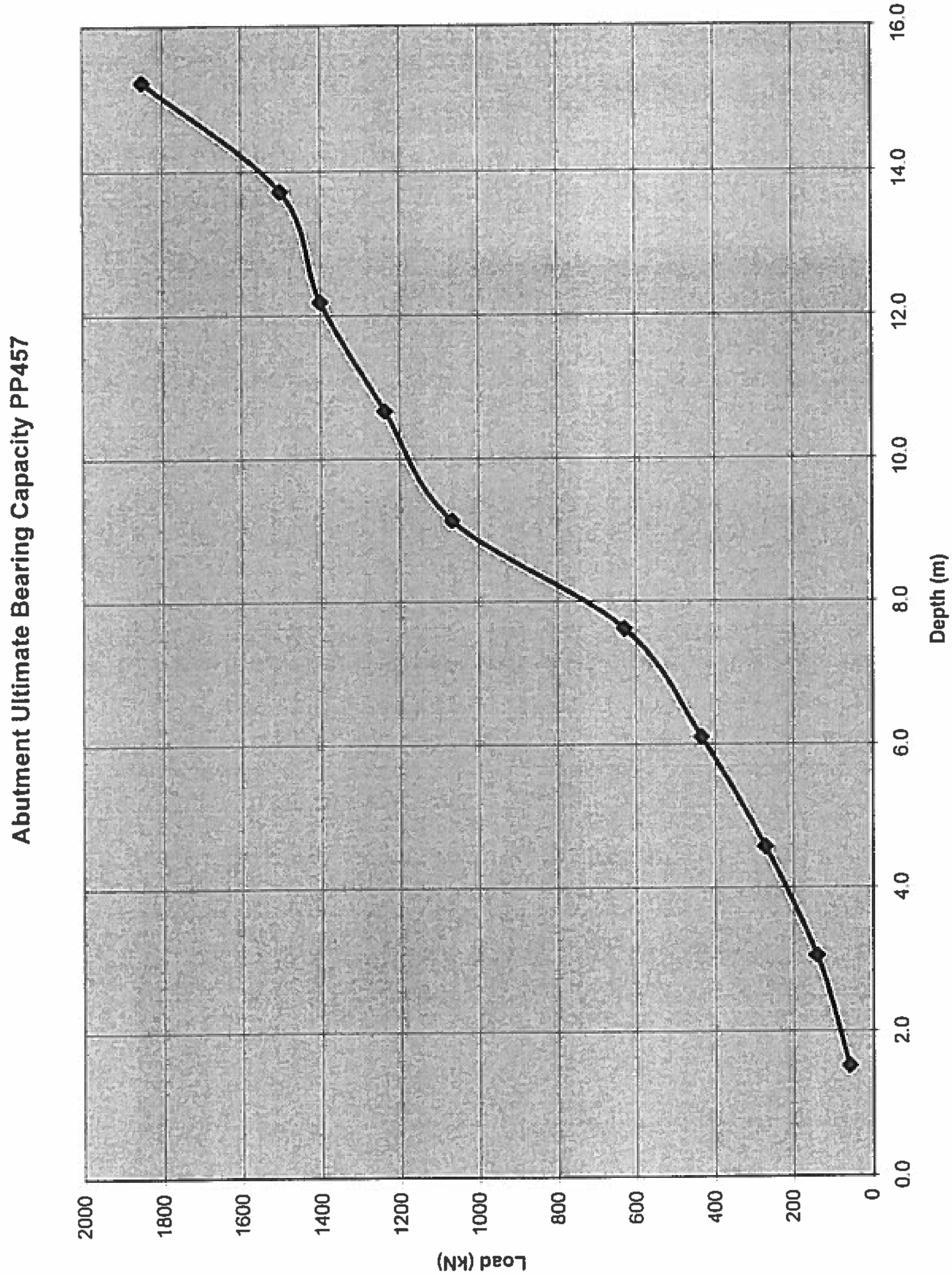
LOVELOCK, NEVADA

PLATE

31

PROJECT NO. 30-2759-01

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

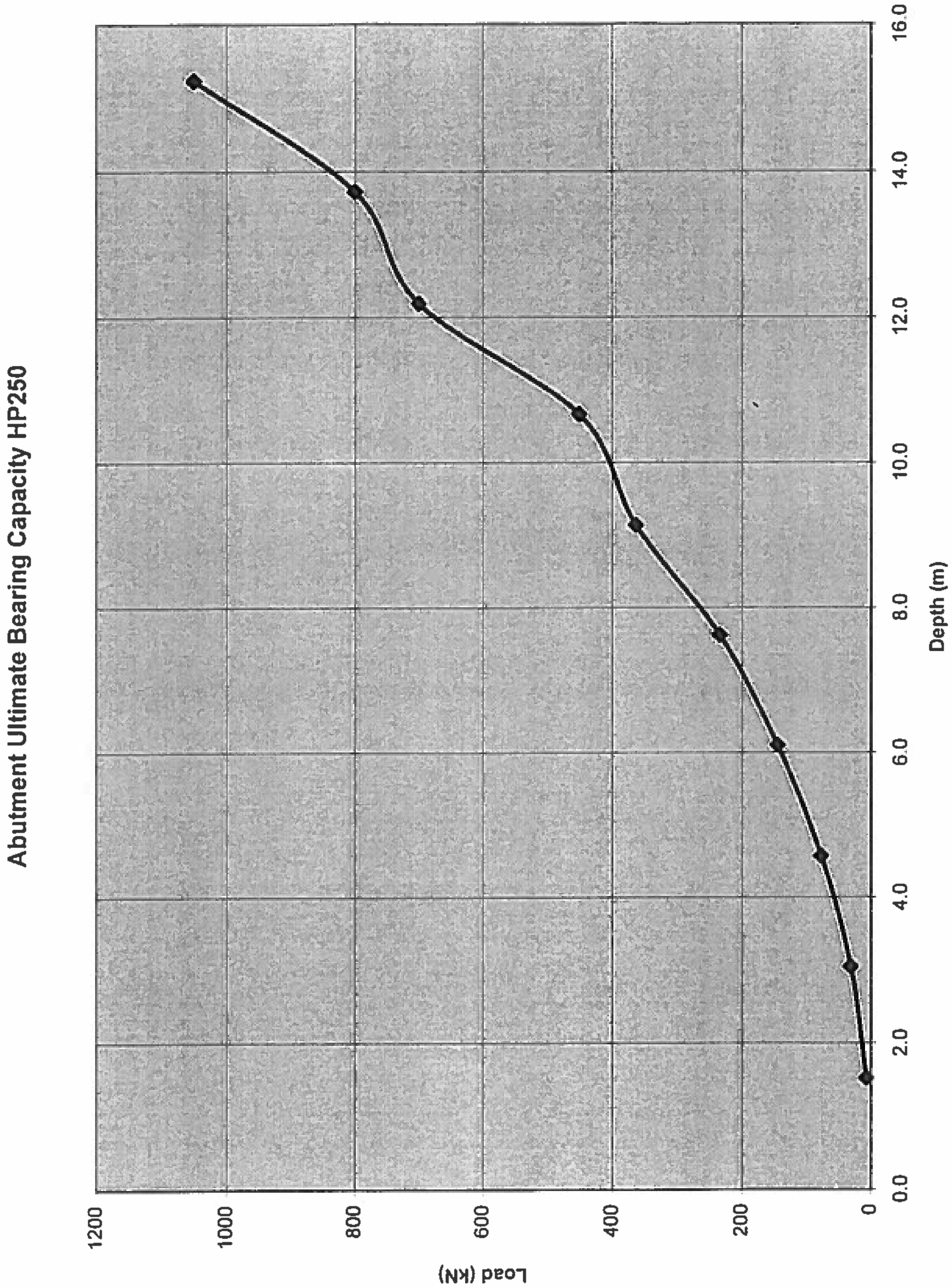
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

32

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

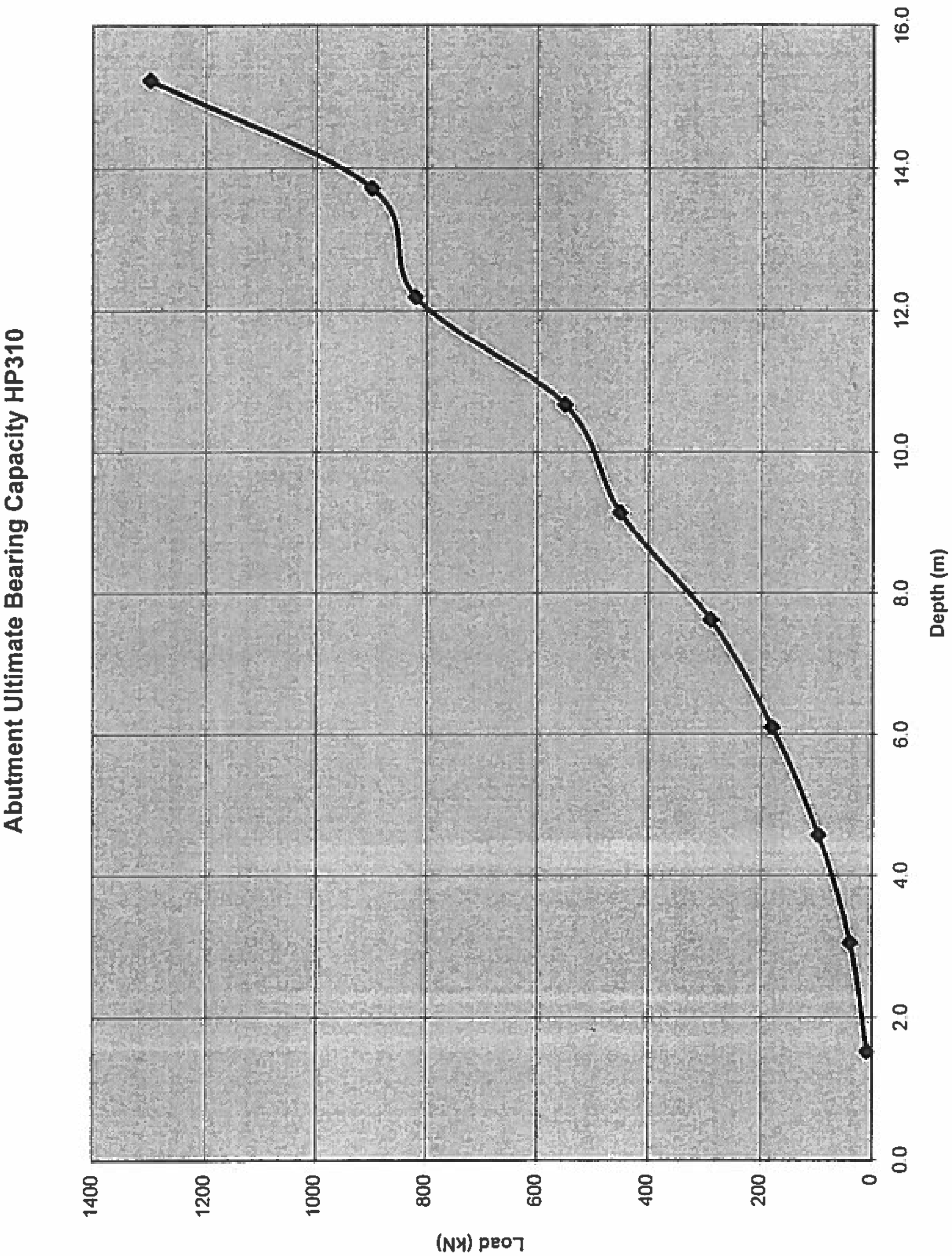
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

33

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

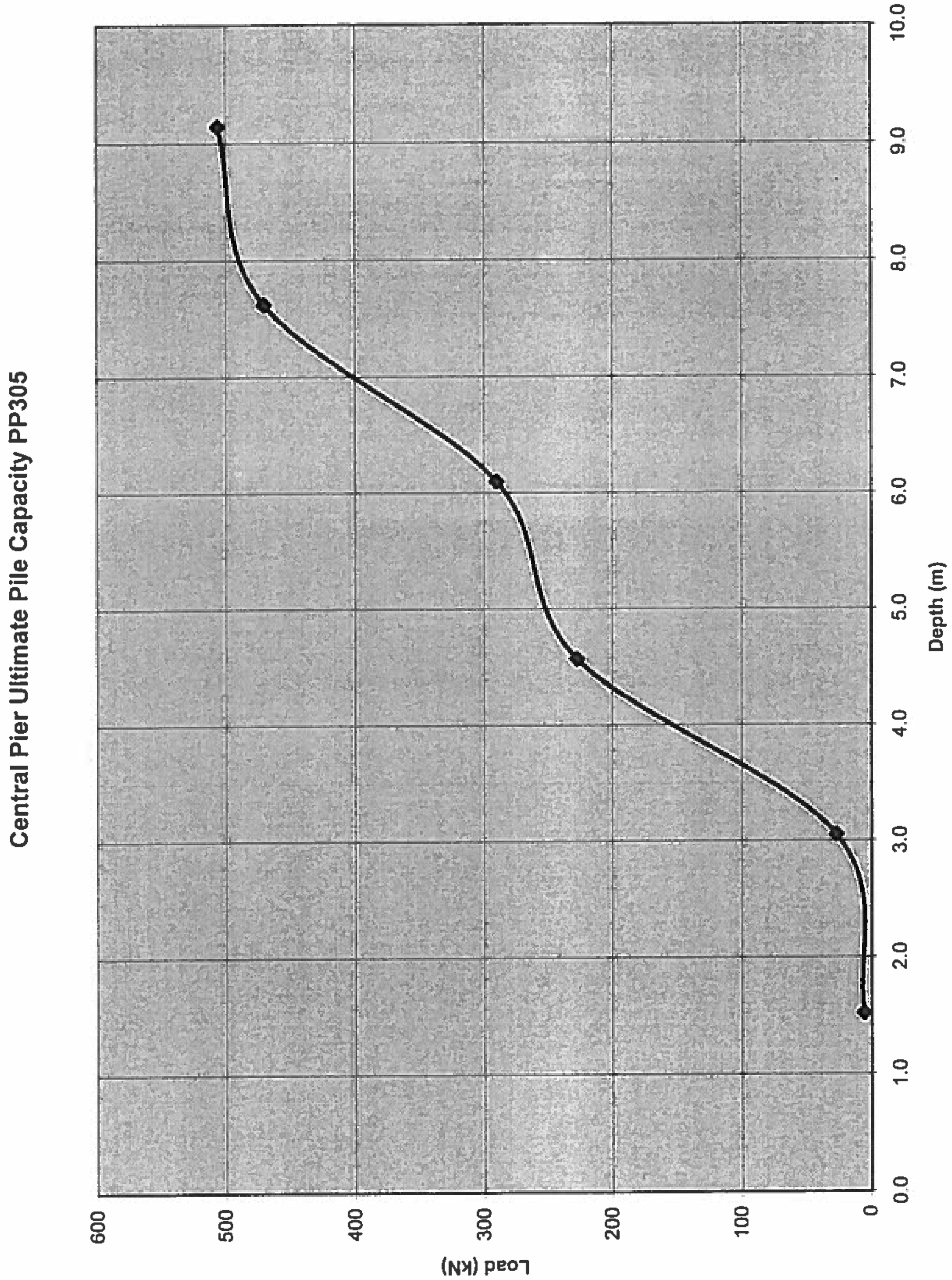
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

34

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

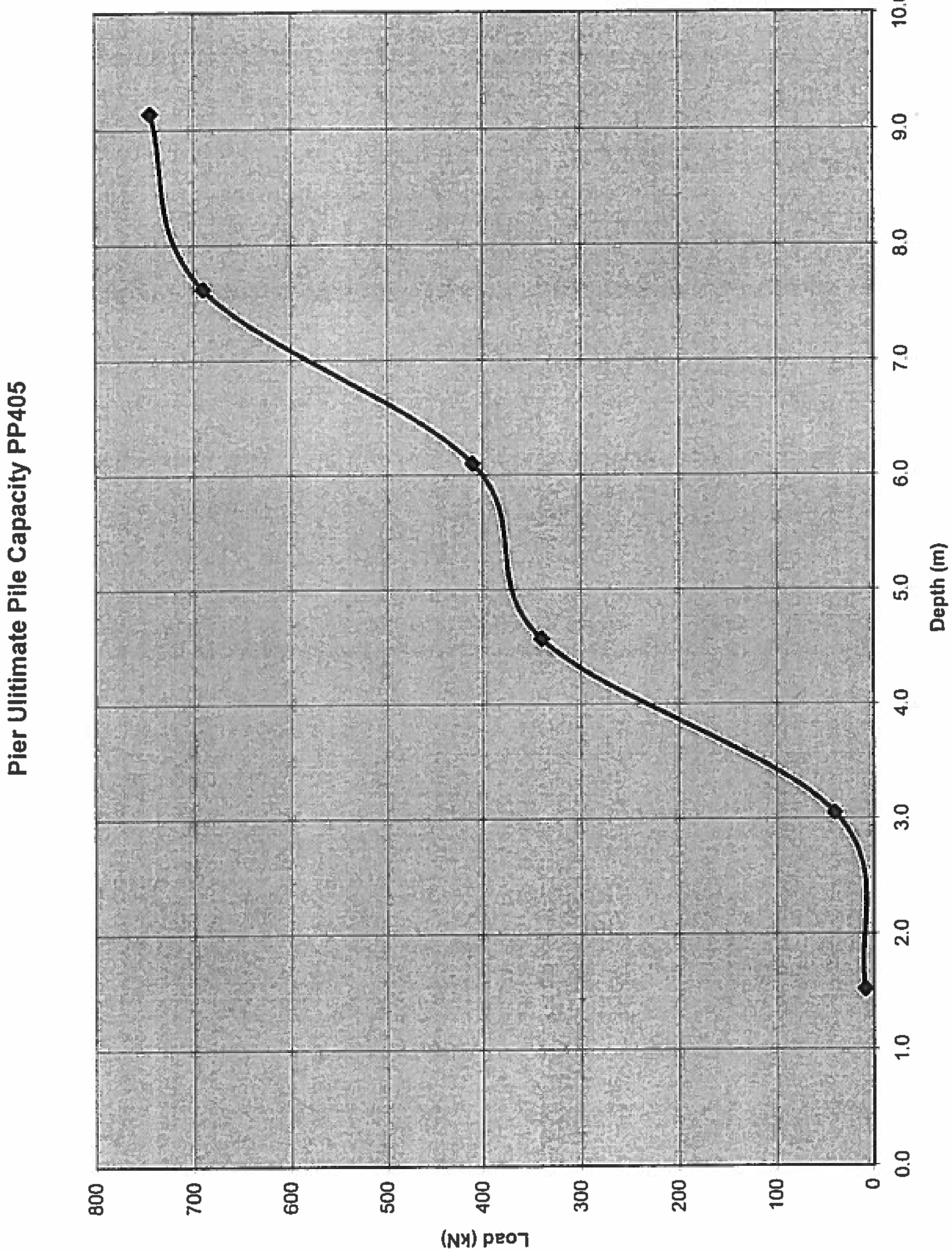
PILE ULTIMATE BEARING CAPACITY

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

35



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

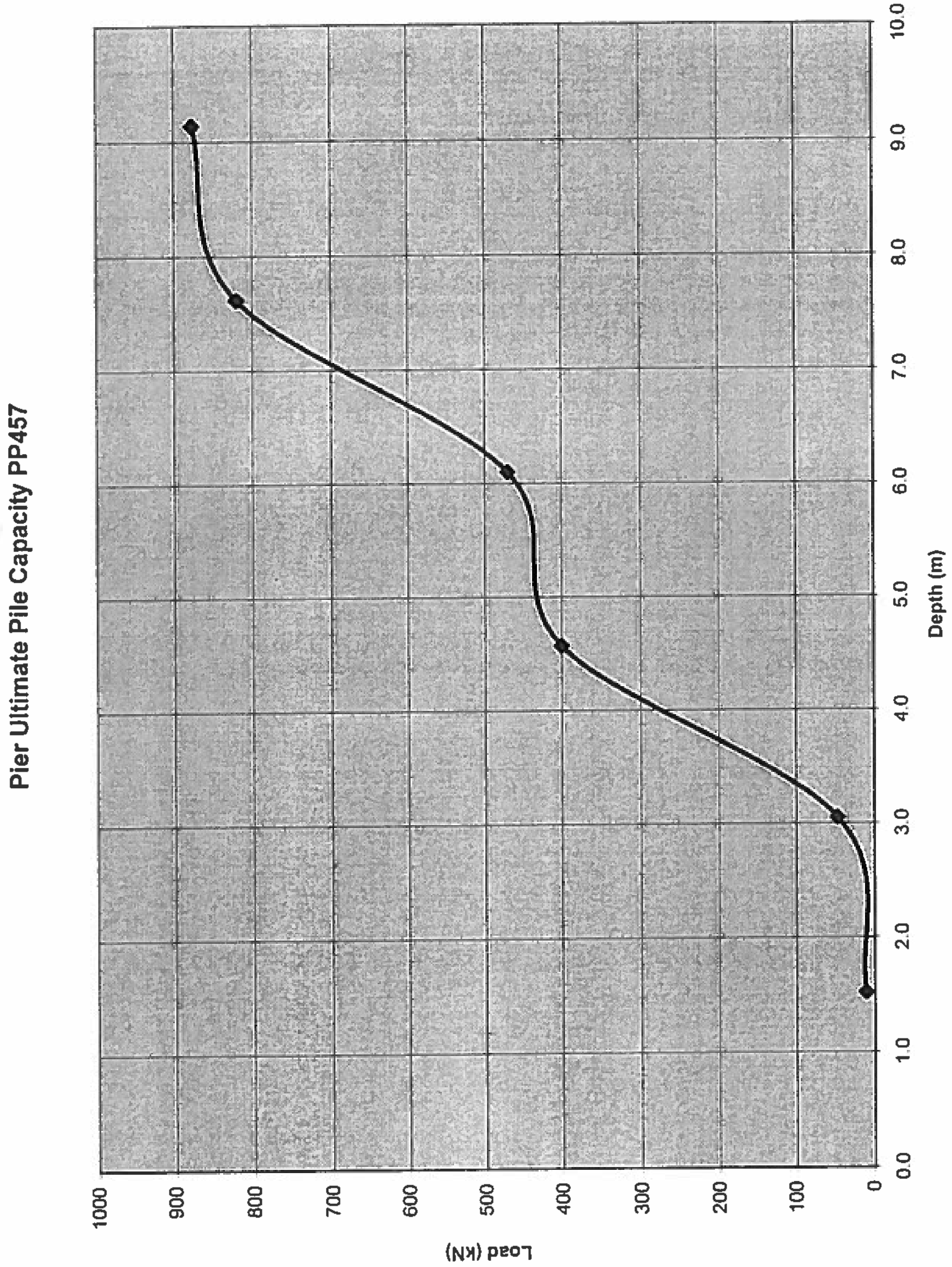
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

36

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

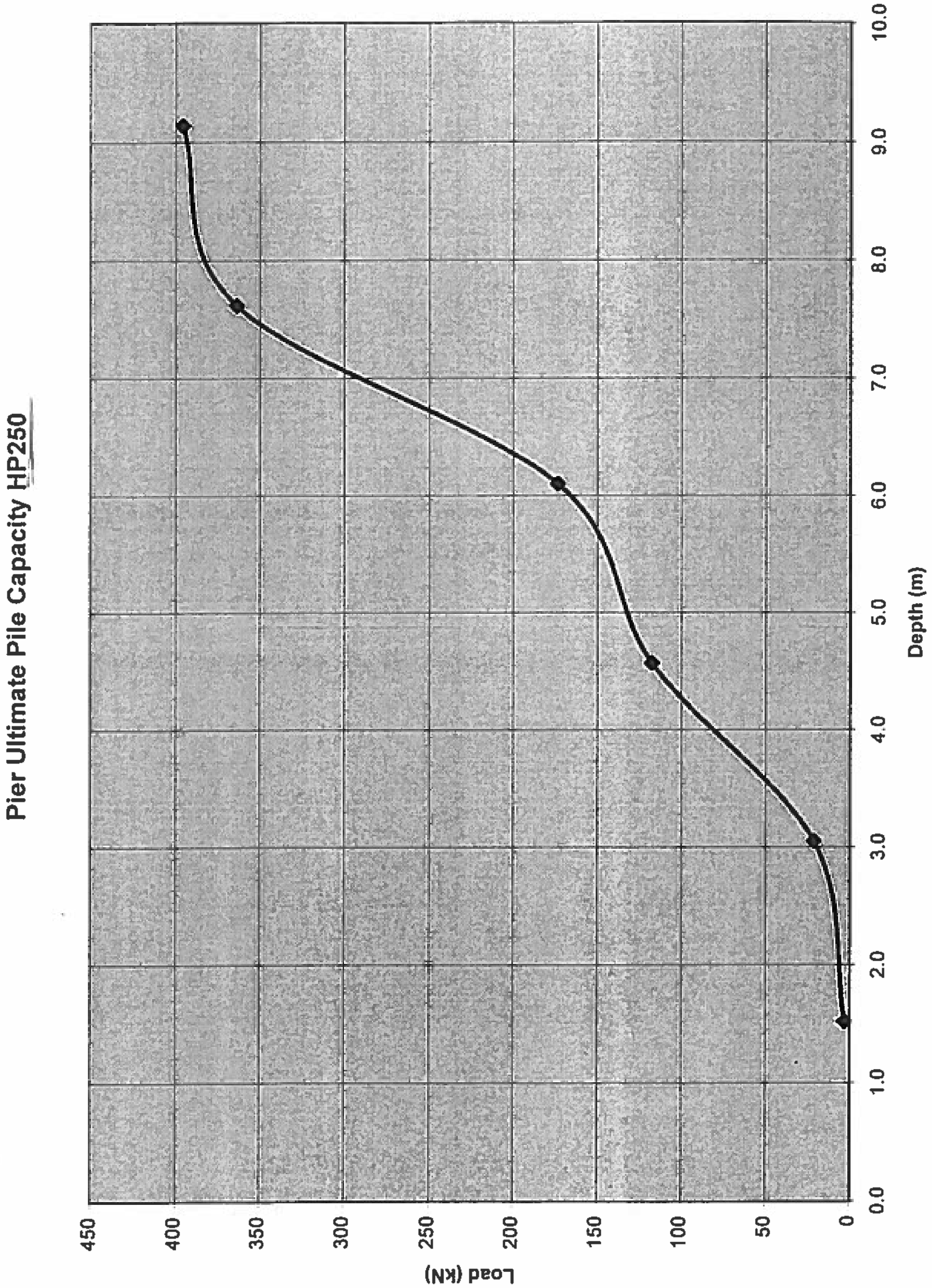
B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

37

CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg



CAD FILE: L:\1999\30275901\30275901-P14,P17,directshear.dwg

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

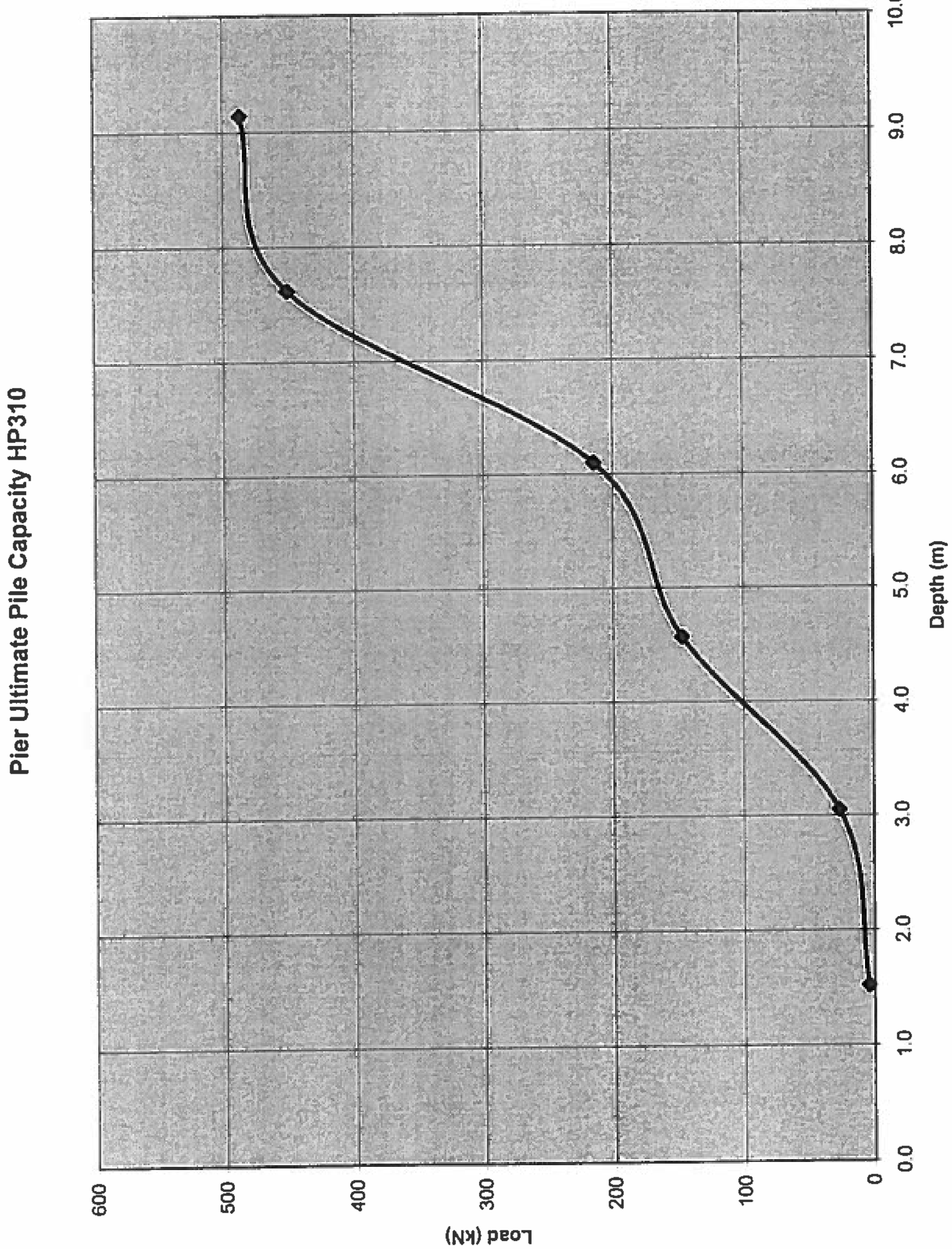
PILE ULTIMATE BEARING CAPACITY

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

38



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 30-2759-01

PILE ULTIMATE BEARING CAPACITY

B-29 BRIDGE REPLACEMENT

LOVELOCK, NEVADA

PLATE

39

CAD FILE: L:\1999\30275901-P14,P17,directshear.dwg

APPENDIX B

Application for Authorization to Use

APPENDIX B
APPLICATION FOR AUTHORIZATION TO USE

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BRIDGE B-29 REPLACEMENT
LOVELOCK, NEVADA

Kleinfelder, Inc.
4875 Longley Lane, Suite 100
Reno, Nevada 89502

To whom it may concern:

Applicant understands and agrees that the "Geotechnical Investigation Report, Proposed Bridge B-29 Replacement, Lovelock, Nevada," dated January 10, 2000, Job No. 30-2757-01, for the subject site is a copyrighted document, that Kleinfelder, Inc. is the copyright owner and that unauthorized use or copying of said document for the subject site is strictly prohibited without the express written permission of Kleinfelder, Inc. Applicant understands that Kleinfelder, Inc. may withhold such permission at its sole discretion, or grant permission upon such terms and conditions as it deems acceptable.

Applicant agrees to accept the contractual terms and conditions between Kleinfelder, Inc. and BRG Engineering originally negotiated for preparation of this document. Use of this document without permission releases Kleinfelder, Inc. from any liability that may arise from use of this report.

To be Completed by Applicant

____
(company name)

____
(address)

____
(city, state, zip)

______________ ______________
(telephone) *(FAX)*

By: ______________________________

Title: ______________________________

Date: ______________________________

For Kleinfelder, Inc.'s use only

_______ approved for re-use with additional fee of $ ____________

_______ disapproved, report needs to be updated

By: ______________________________
(Kleinfelder, Inc. project manager)

Date: ____________________