FOUNDATIONS OF THE SKYBRIDGE AND NO. 2 ROAD BRIDGE CROSSINGS OF THE FRASER RIVER, VANCOUVER, B.C.

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Two new bridges have been built across the Fraser River in the Vancouver area since the Annacis cable-stayed bridge (now known as the Alex Fraser Bridge) was completed in 1986. SkyBridge is a 616 m long, high-level, cable-stayed bridge with a 340 m main span across the main shipping channel some 5 km upstream of the Annacis Bridge. The bridge was completed in 1989 to extend the Vancouver SkyTrain system into Surrey. It remains the world's only cable-stayed bridge carrying rapid transit only. The No. 2 Road Bridge across the Middle Arm of the Fraser River is 15 km downstream of SkyBridge and a key link in the road system connecting Richmond to Vancouver. It was completed in 1993. This paper describes the geotechnical investigations carried out for the two bridges and the very different foundations resulting from the soil conditions and design loads.

INTRODUCTION

SkyBridge is a high-level cable-stayed bridge across the Fraser River between New Westminster and Surrey, about 5 km upstream of the Annacis highway bridge (now the Alex Fraser) described by Bazett and McCammon (1986) (see Figure 1). The two main towers rise about 140 m above river level and support a main span of 340 m. Three piers are designed to withstand ship impact without significant structural damage. Ship impact forces and seismic design of the foundations required a comprehensive geotechnical investigation and foundation design program to be carried out.

The No. 2 Road Bridge is a low-level, 566 m long, 4 lane highway bridge across the Middle Arm of the Fraser River between Richmond and Sea Island, 15 km downstream of SkyBridge (see Figure 1). It supplements the existing Dunsmuir and Middle Arm Bridges connecting Richmond to Sea Island, on which the Vancouver International Airport is situated. Ten spans between 36 m and 64 m in length form a continuous structure across the channel. Restrictions on differential settlement of the piers and the potential for liquefaction of the foundation soils required a geotechnical investigation involving drilling and piezo-cone penetration tests (CPTs) on the river banks and in the river.

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This paper describes the geotechnical investigations carried out, the foundation design considerations, construction observations and post-construction movements recorded for the two bridges.

SKYBRIDGE

Topography

The topography differs substantially on the two banks of the river. For a considerable distance back from the river bank on the south (Surrey) side, the ground is low-lying and less than 1 m above extreme high river level. On the north side, the ground quickly rises from the river's edge to above El. 50 m (Geodetic) in developed areas of New Westminster.

Geotechnical Investigation

The locations of all drill holes completed in the period 1982 to 1985 for geotechnical investigation of the bridge are shown on Figure 2. The 1982-1983 holes were completed on an upstream (subsequently superseded) alignment. The 1984-1985 holes were completed on the selected alignment. The work was carried out onshore and offshore, using conventional drilling and sampling techniques and CPTs. A sub-bottom acoustical survey of the river on the upstream alignment was carried out in 1983. Downhole shear wave testing of overburden was carried out in one test hole on each bank of the river.

The results of these investigations were supplemented by reference to published reports on foundations of the Pattullo and Canadian National Railway bridges immediately upstream of SkyBridge (see Figure 2) and the Annacis Bridge. Reference was also made to previous investigations for land development projects on both banks of the river in the vicinity of the proposed crossing.

Geology

An inferred geological profile along the bridge centreline is shown in Figure 3. The site is underlain at depth by bedrock, varying in elevation from about El. -50 m at the New Westminster bank to about El. -80 m at the Surrey bank.

In the Surrey half of the river, bedrock is overlain by a coarse gravel deposit. Marine deltaic deposits, consisting mainly of clayey silt and an occasional layer of silty sand, overlie the coarse gravels.

On the New Westminster side of the river, deposits consisting of interbedded silt, sand and gravel overlie the bedrock and partially underlie the marine deltaic deposits.

Overlying the marine deltaic deposits and, within the river, the interbedded deposits, is Fraser River Sand. This deposit, which contains an occasional thin seam of silt and gravel, is more than 40 m thick on the Surrey side of the river but pinches out at the New Westminster bank.

Fraser River Sand is overlain by sand and gravel bedload within the river and by overbank and levee deposits consisting of mixtures of silt, clay and sand on the Surrey side.

Further description of the principal stratigraphic units on the bridge alignment is as follows, in generally ascending order:

Bedrock

Bedrock cores were obtained in six boreholes. Bedrock consists of very weak to strong, interbedded layers of sandstone, siltstone and shale with thin coal seams and some pebble conglomerate. Breccia zones and some slickensides were identified in bedrock cores recovered in two holes. These features are believed to result from disruption of the bedrock by processes other than faulting. Artesian flows were observed during drilling of several holes when they reached bedrock. Additional discussion of this aspect is presented below.

Interbedded (Pre-Vashon) Deposit

An interbedded deposit of silt, sand and gravel was encountered in drill holes at and between Piers N1 and N2. The deposit also contains occasional cobbles and boulders. The deposit is dense and believed to be Pre-Vashon, and therefore overridden by glaciers during the Vashon phase of glaciation (approximately 15,000 years before present). However, no radiocarbon dating has been undertaken to confirm this classification. Similar material is reported in other test holes in New Westminster. Artesian flow similar to that encountered in the bedrock was encountered in one hole.

Lag Gravels

Coarse gravels with some cobbles and boulders overlie bedrock in the area of Piers S1 and S2. The deposit appears to thicken towards the Surrey shoreline, but is generally of the order of 10 m thick.

Marine Deltaic Deposits (referred to as Marine Basin Deposits or Marine Silt in some studies)

In the middle of the river, drilling has indicated this deposit to consist mainly of firm to very stiff, clayey silt or silty clay with some thin, loose to compact, sand layers. CPT results indicate that the sand layers tend to be concentrated near the top and bottom of the unit, with reasonably uniform clayey silt occupying the middle portion. Wood and shell fragments are present.

Fraser River Sand (referred to as Fraser River Alluvium in some studies)

This unit is generally compact to dense and reaches a maximum thickness of nearly 40 m in the middle of the River. The sand is micaceous, generally fine to medium with up to 10% silt, but contains layers of silty, fine sand and gravel. On the Surrey bank, the sand unit contains occasional layers of deltaic clay.

Bedload

Bedload is generally loose, fine to medium sand.

Overbank and Levee Deposits

These deposits have formed during post-glacial times, by deposition of fine sand grading to clayey silt in the quiet water areas behind natural levees built up by the Fraser River adjacent to the main channel. More recently, swamps formed in some of these poorly drained areas and peat deposits (up to 8 m thick) developed. The mineral, non-peaty soils are characteristically in a loose to very loose condition.

Representative gradation envelopes for the Fraser River Sand and marine silt are shown in Figure 4. The plasticity chart for the marine silt is shown in Figure 5.

Groundwater Conditions

On the south bank of the river (Pier S2), the groundwater level approximates the mean Fraser River level and is generally within 2 to 3 m of the ground surface.

Artesian conditions were detected at depth during drilling at Piers N1 and N2 by flow of water out of the top of the drill casing. A multiport piezometer (manufactured by Westbay Instruments Ltd.) was installed in TH 85-1 in Pier N2 and monitored for 8 months. A typical record is shown in Figure 6. The artesian level increases with decreasing elevation, indicating upward drainage flow. An artesian head of 3 m above river level is shown for El. -20 m, the approximate foundation level of Pier N2. Sealed standpipe piezometers installed in 2 nearby drill holes gave similar results. The New Westminster uplands to the north of the bridge provide groundwater recharge areas at high elevation.

Groundwater samples were obtained from 4 ports of the Westbay piezometer and submitted to the University of Waterloo for determination of tritium concentration to assist in identification of the age and, therefore, the origin of groundwater flow on the New Westminster bank. Below detection levels of tritium were recorded, suggesting, according to Freeze and Cherry (1979), that the groundwater is at least 30 years old.

Test Pile Program

Pile driving and pile load test programs were undertaken at Piers N1 and N2 to assist in pile design.

At Pier N1, in the river, two heavy section H-piles and a 324 mm diameter, 19 mm wall, open-ended steel pipe pile were driven to bedrock. At Pier N2, on the New Westminster bank, a heavy section H-pile and a 610 mm diameter, 19 mm wall, open-ended steel pipe pile were driven into the interbedded deposit to the expected elevation of the pier support piles. All piles were fitted with Pruyn Points and driven using a Delmag D46 diesel hammer. They were monitored by a pile analyzer during driving to allow CAPWAP analysis of pile capacity. Four lengths of sheet pile were also driven into overburden in the vicinity of Pier N2 to check drivability for cofferdam construction. The H pile at Pier N2 was loaded to 3500 kN using six pipe piles driven open-ended to about 13 m embedment for the reaction.

The results of the CAPWAP analysis are summarized in Table 1. Plots of driving resistance versus depth for Piles N1-3 (a 324 mm diameter pipe pile) and N2-3 (an HP360x174 pile) are shown in Figure 7. Figure 8 shows the load versus head displacement plot for Pile N2-3 and indicates an ultimate pile capacity in excess of the 3500 kN test load, in agreement with the CAPWAP analysis.

The driving and subsequent withdrawal and inspection of the test piles and reaction piles confirmed that pile tips would be required to prevent damage and that pile clean out would be affected by cobbles and boulders in the overburden, but piles could be expected to reach bedrock at Pier N1.

Design Considerations

Some of the criteria affecting foundation design are discussed below.

Ship Collision

The two main towers (Piers S1 and N1) and the north anchor pier (N2) were designed for two levels of head-on ship collision, namely an extreme ship with a displacement of 18,000 tonnes travelling at 6 m/s and a reduced ship with weight and velocity equal to 80% of the extreme values. The extreme ship collision is permitted to cause damage of a local and repairable nature and permanent displacement, but the bridge is to remain structurally sound. During a reduced ship collision, the structure is to remain essentially elastic with no significant permanent displacement.

Design of the piers resulted in round shafts of 18 m diameter. This increased the probability that most collisions would result in a glancing blow, but head-on collision forces were used for design. This conservatism was further increased for Pier S1 because shallow water is unlikely to allow the design ship to reach the pier.

Scour

Increased river velocity due to the piers was estimated to cause a general lowering of the river bed of about 1.5 m on average, due to scour. During flood stages, this could increase to 2.5 m.

The most serious localized scour conditions occur at Pier N1 which is in a local depression in the river bed with a water depth of about 19 m. Scour conditions are aggravated by a rock weir which extends out from the river bank upstream of the pier. Hydraulic model tests were conducted to determine scour patterns and required riprap protection around Piers N1 and N2. Substantial protection was dictated around Pier N1, with stones weighing up to 1900 kg in a layer 3 m thick with the top set a few metres below the river bed to allow for general scour effects.

Scour around Pier N2 is minimal due to the dense nature of the surrounding soil. Pier S1 is in relatively shallow water. Thus, riprap aprons around these piers are nominal.

Earthquake

The bridge is designed to remain elastic during the 100 year (design level) earthquake with no significant structural damage or interruption to operation. The bridge must also undergo a survivability level earthquake with 475 year return period without collapse of the bridge, foundation or any primary structural element. Inelastic deformation and local damage are permitted providing the damage is repairable.

Peak accelerations of 0.12g and 0.24g at bedrock level were established for the design and survivability earthquakes, respectively, by BC Transit (the Owner), primarily based on seismicity studies carried out for the Annacis Bridge.

A seismic response analysis was carried out for the foundations of Piers N1 and S1 to determine the free-field response spectra, the compliance springs to connect the free-field to the bridge foundation and the liquefaction potential of the sand.

Base rock time histories for 3 earthquakes (Santiago 1965, Imperial Valley 1979 and San Fernando 1971) were scaled to peak accelerations of 0.12g and 0.24g and used with the shear wave propagation computer program SHAKE to bring the motion to the surface. Soil properties for the analyses were established from drilling results for SkyBridge and comparison with in-situ testing carried out for the Annacis Bridge. Upper and lower bound values were established for some properties.

Base rock peak accelerations of 0.12g and 0.24g were shown by the SHAKE analysis to produce upper bound values of 0.25g and 0.33g, respectively, at river bed level at Pier S1, but only 0.15g and 0.20g at 6 m below river bed, at the underside of the pier footing. For Pier N1, the peak values are 15% to 25% higher.

At Pier N1, the analysis indicates that liquefaction will occur to 10 to 16 m below river bed (depending on whether the upper or lower bound value of modulus reduction is used) for the 475 year earthquake. At Pier S1, liquefaction to a depth of 6 to 22 m is indicated.

Settlement

At an early stage in the design process, it was established that the foundations for Pier N1 would be taken down to bedrock, whereas for the other 3 piers, the foundations would be terminated above bedrock. Thus, foundation settlement during and after construction was a critical item, particularly for Pier S1 because of the compressibility of the Fraser River sand and consolidation of the marine deltaic deposits. Most of the compression in the sand layer would take place as the foundation load increases during pier and tower construction and can be compensated for. Consolidation of the marine deltaic deposits will continue after the bridge is completed. The maximum allowable post-construction settlement of the bridge foundations was specified to be 0.001 of the smaller, adjacent span, giving 140 mm for Piers N1 and S1 and 60 to 70 mm for anchor Piers N2 and S2.

Expected settlement at Pier S1 was calculated using compression index (C_c) values of 0.15, 0.25 and 0.35 for the marine deltaic deposit, based on consolidation test results, and an elastic modulus increasing from 20 to 30 MPa with depth for the Fraser River sand. The model was calibrated against observed settlements at the Pattullo Bridge, immediately upstream. This calibration indicated good agreement for C_c values of 0.25 to 0.35.

The results of the analysis for Pier S1 are shown in Table 2. The estimated post-construction settlements are shown to be generally within tolerable limits.

The Contract Documents for bridge construction advised the contractor that the settlement of Pier S1 was conservatively estimated to be 300 mm during construction and required him to compensate for the actual settlement that took place.

Foundation Design

Foundation Alternatives

The variation of soil conditions across the river and design forces provided the following considerations:

(a) Pier N2

Pier N2 is founded in dense glacial material up to 36 m thick overlying bedrock. A spread footing, designed for an allowable bearing pressure of 800 kPa, would be suitable if of sufficient weight to resist overturning arising from ship collision forces. With scour as an additional concern, a footing founded at El. -18 m and supported by HP310 x 174 piles was selected. The pile driving test program confirmed that driving would be difficult but was feasible with use of driving shoes to prevent pile damage. Resistance to ship collision forces, additional to the tension capacity of heel piles, was provided by passive resistance against the foundation.

(b) Pier N1

Pier N1 is founded in deep water, with about 16 m of sand and silt above bedrock. The design of the pier is governed primarily by dead and live loads from the bridge superstructure, combined with ship collision forces. These forces dictate use of a massive pier. A spread footing was not considered advisable, unless founded at considerable depth below the river bed because of scour potential. The pile driving test program confirmed that piles could be driven to bedrock. Therefore, H piles or steel pipe piles driven open-ended to bedrock were considered. The possibility of liquefaction of the surrounding sand to a depth of 16 m below river bed was the most important factor in selecting 915 mm diameter pipe piles for the foundation.

(c) Pier S1

This pier is founded in relatively shallow water and is underlain by Fraser River sand. It does not have the same construction problems as at Pier N1. Scour potential ruled out use of a shallow spread footing and steel pipe piles of 610 mm diameter, driven closed-end to El. -30 m in the sand, were selected. The pier is surrounded by timber compaction piles to densify the sand and prevent liquefaction in the event of an earthquake. The primary design loads are dead load from the superstructure and ship collision forces. Live, wind and seismic forces are of little significance.

(d) Pier S2

This pier is on land and underlain by 60 m of overbank deposits and Fraser River sand. A spread footing would have to be founded at a depth of at least 6 m and, consequently, it was concluded that a pile foundation was more economical. For consistency, the same piles were selected as for Pier S1. The pile cap is set high enough to avoid excavation below the water table. Timber compaction piles surround the pile cap to prevent liquefaction of the sand during an earthquake.

The governing design loads for Pier S2 derive from dead load and earthquake conditions. Live and wind loads are of little significance and ship collision inappropriate.

Final Design Pile Capacities

Using the results of the geotechnical investigation and pile load test program and Annacis Bridge pile data, the pile capacities shown in Table 2 were established.

The ultimate (useable) pile capacity value was established at about 80% of the conventionally defined ultimate pile capacity to safely resist the short duration force imposed by an extreme ship collision, whilst allowing permanent displacement, but not collapse, of the bridge. For the 610 mm pipe piles, the values were established by consideration of Annacis Bridge results for test pile and production pile driving and testing.

Construction Phase

Construction of cofferdams at Piers S1, N1 and N2 and foundation excavation and pile driving at all piers were carried out with relatively few problems between September 1986 and December 1987. Less than $250,000 in extras was paid to the contractor for changed foundation conditions and pile redriving carried out at the Engineer's discretion, compared to the $12 million cost of the foundation work.

Driving records were prepared for all piles, using visual observations of hammer blows for each 300 mm of pile penetration. Monitoring of selected piles was carried out with a pile driving analyzer, followed by CAPWAP analysis, to confirm design assumptions.

Settlement monitoring was carried out at all Piers, but with particular attention to Pier S1 because of the need to maintain the correct bridge geometry. Up to completion of the shaft at El. 8.9 m, about 20 mm of settlement was recorded for a net increase in load of 60,000 kN at underside of foundation level. A further 15 mm was recorded during construction of the tower, applying 50,000 kN additional load. Upon completion of the bridge in January 1989, with 180,000 kN total net increase in load, the settlement was 55 mm. An appropriate correction was made to the tower height, with the expectation that up to 100 mm post-construction could occur.

Settlements recorded at Piers N1, N2 and S2 were negligible. A horizontal movement of 27 mm in the downstream direction was recorded on the top of the shaft (El. 8.6 m) at Pier N1 between July 1987 and January 1989. The reason for this movement was not evident, but suggested the need for long term survey monitoring of the bridge.

Post Construction Foundation Movements

A precise survey control system was set up in January 1989 to allow long term monitoring of the bridge alignment. The piers were surveyed in March 1994 and indicated that Piers N1 and N2 had not moved vertically or horizontally in the previous

5 years (within the 0.5 mm accuracy of the readings) whereas Piers S1 and S2 had settled 35 mm and 15 mm.

NO. 2 ROAD BRIDGE

Topography

The topography on both banks is similar, comprising flat-lying ground near extreme high river level, protected from flooding by dykes.

Geotechnical Investigation

A drilling and CPT investigation of the crossing was carried out in 1990. The locations of all test holes are shown on Figure 9. The maximum depth of investigation was 160 m on the north bank. The bridge centreline was moved upstream subsequent to completion of the investigation, but no further drilling was deemed to be necessary.

Geology

An inferred geological profile along the bridge centreline is shown in Figure 10. The site is underlain by a surficial silt layer overlying 10 to 15 m of sand which is underlain by a silt unit shown to be more than 130 m thick by drilling on the north bank.

The sand is fine, with a gradation typical for Fraser River sand, and contains occasional silty lenses. It varies from loose to dense.

The lower silt unit is firm to stiff and contains thin laminations of silty sand and clayey silt. It has low plasticity with a Liquid Limit in the range of 27 to 33. Consolidation tests suggest that the unit is normally consolidated on the river banks and slightly overconsolidated under the river.

Groundwater Conditions

The groundwater level is within 1 to 2 m of the ground surface on both banks and approximates mean river level.

Test Pile Program

A test pile was driven on the south river bank to investigate driving resistance. The pile utilized was a 915 mm diameter, closed end steel pipe pile driven to 16 m embedment using a Kobe K45 diesel hammer. The asphalt coating on the pile could not be removed prior to driving due to time constraints.

Foundation Design Considerations

General

The site investigation indicated that, within the depth of investigation, the only firm bearing encountered was provided by the sand unit. However, to limit consolidation of the underlying silt layer, the foundation system had to be as high as possible in the sand layer whilst being deep enough to resist scour. An upper limit of 50 kPa was established for the increase in stress at the top of the silt layer, based on the consolidation test results and local experience.

Given the above considerations, spread footings or short pipe piles were considered to be feasible alternatives. Since footings require construction of cofferdams for the 7 piers in the river, it was deemed more economical to utilize short piles with a pile cap raised above low river level to allow construction in the dry.

Earthquake

The design earthquake corresponds to a Richer Magnitude 7 event and uses a peak horizontal ground acceleration on firm ground or rock of 0.21g. The deep deposits of silt will amplify this motion and a peak horizontal ground surface acceleration of 0.30g was assumed, in accordance with the City of Richmond's design procedures.

Without ground improvement, an earthquake of this magnitude would result in liquefaction of loose sand under the river banks and river bed with considerable effect on the 9 piers and 2 abutments. Therefore, consideration was given to use of timber compaction piles, vibrocompaction, vibroreplacement and dynamic compaction.

Timber piles were selected, principally on economic grounds, to densify a zone about 5 m wide around each pile cap and to a depth of about 10 m. A larger zone was utilized adjacent to the river banks where sloping ground increased the risk of liquefaction-induced ground movement.

The raised pile caps and limited pile embedment result in concern regarding the ability of the piles to safely withstand the horizontal inertia loads developed by the design earthquakes without excessive horizontal deflection. Analysis using the LATPILE computer program for horizontal earthquake levels of 500 to 850 kN at pile cap level indicated predicted deflections of 35 to 50 mm at the pile cap for a single pile. Group effect was predicted to increase the transient deflections of the pile caps to between 100 and 125 mm.

Final Foundation Design

The pile design selected for the bridge foundation comprised 1220 mm diameter steel piles, driven closed end. Pile tip elevations were specified to provide at least 6 m of pile embedment below the finished river bed and not less than 6 m between the pile tip and the top of the silt stratum. An ultimate pile capacity of 12000 kN was established, with an allowable (service) capacity of 4000 kN, allowing a factor of safety of 3.

A typical river pier foundation is shown on Figure 11.

Construction Phase

Construction of the pier and abutment foundations was carried out with few problems between June 1992 and November 1992. The Contractor started pile driving with a vibratory hammer, then progressed to Kobe K32 and Delmag D46 diesel hammers until a Delmag D62 hammer was ultimately obtained for use at Piers 4 to 9 and the North Abutment. Driving of all piles was monitored by visual counts of hammer blows. Pile driving analyzer records were obtained at 4 piers for CAPWAP analysis to confirm design assumptions. Typical pile driving records are shown on Figure 12.

Survey hubs were established on each pile cap on completion of concreting to monitor settlement during and after completion of the bridge. Readings taken in May 1993, between 161 and 293 days after the initial survey at each pier, indicated less than 20 mm settlement except at Pier 5 where 35 mm was recorded, consistent with lower blow counts at this Pier as compared to the other piers. A survey of the bridge deck carried out in March 1994 indicated that negligible additional settlement of the piers had occurred.

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No. 2 Road: PBK Engineering Ltd.

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Both Bridges: Dr. P. M. Byrne (UBC)

Pile Driving Analyzer Testing
SkyBridge: Trow Geotechnical Ltd.
No. 2 Road: Anna Geodynamics Ltd.

Contractor:
SkyBridge: Kerkhoff-Hyundai
No. 2 Road: Delta Catalytic Constructors Ltd.

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Table 1. Test Pile Program Results Ultimate Pile Capacity Estimated By CAPWAP Analysis

Test Pile	Pile Type	Tip Elevation (m)	Embedment Depth (m)	Material at Tip	Estimated Ultimate Pile Capacity (kN)		
					Skin	Toe	Total
N-1	HP360x174	-52.0	31.6	Bedrock	4700	1850	6550
N-2	HP310x174	-52.6	31.2	Bedrock	4410	1240	5650
N-3	324 mm open-pipe	-52.5	30.7	Bedrock	3620	2420	6040
N2-2	610 mm open-pipe	-38.1	27.2	Overburden	4070	1400	5470
N2-3	HP360x174	-31.8	22.8	Overburden	3500	270	3770
N2-3	HP360x174	-42.6	33.6	Overburden	4010	500	4510

Table 2. Predicted Settlement of Pier S1

Load Application Point	Compression Index of Silt Unit	Predicted Settlement (mm)				
		During Construction			Post Construction	Total
		Sand	Silt	Total		
1/3 above	0.15	290	10	300	50	350
pile tip	0.25	290	20	310	80	390
(El.-24 m)	0.35	290	25	315	115	430
Pile tip	0.15	170	15	185	65	250
(El.-30 m)	0.25	170	25	195	110	305
	0.35	170	40	210	150	360

Table 3. Pile Capacity

Pier	Pile Type	Approx. Tip Elev. (m)	Pile Capacity (kN)					
			Service		Ultimate (Useable)		Ultimate (Failure)	
			Comp.	Tens.	Comp.	Tens.	Comp.	Tens.
N2	HP310x174	-33 to -39	1800	1500	3600	3000	4250	3750
N1	915 mm pipe	-50	11500	4500	22000	6700	29000	9000
S1	610 mm pipe	-30	1800	900	3600	1800	4500	2000
S2	610 mm pipe	-20	1800	900	3600	1800	4500	2000

April 29, 1994

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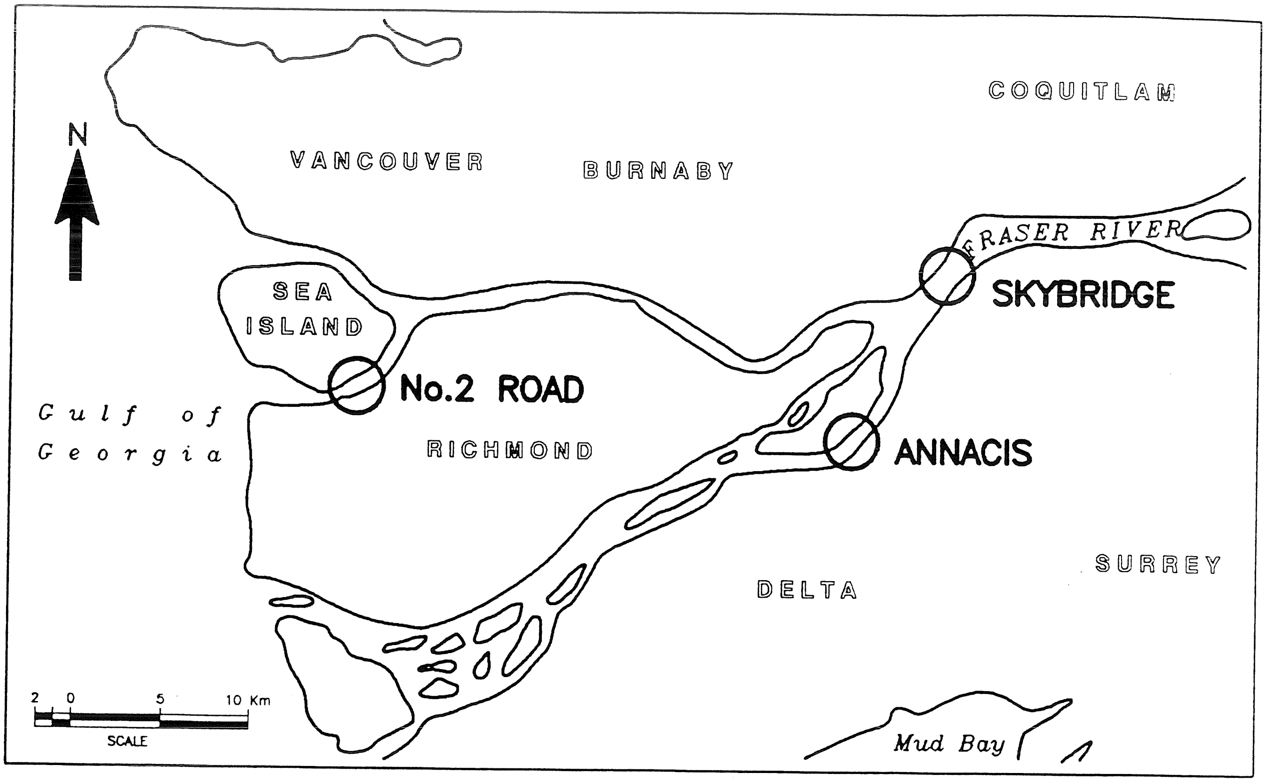

FIG. 1. Location of bridge sites

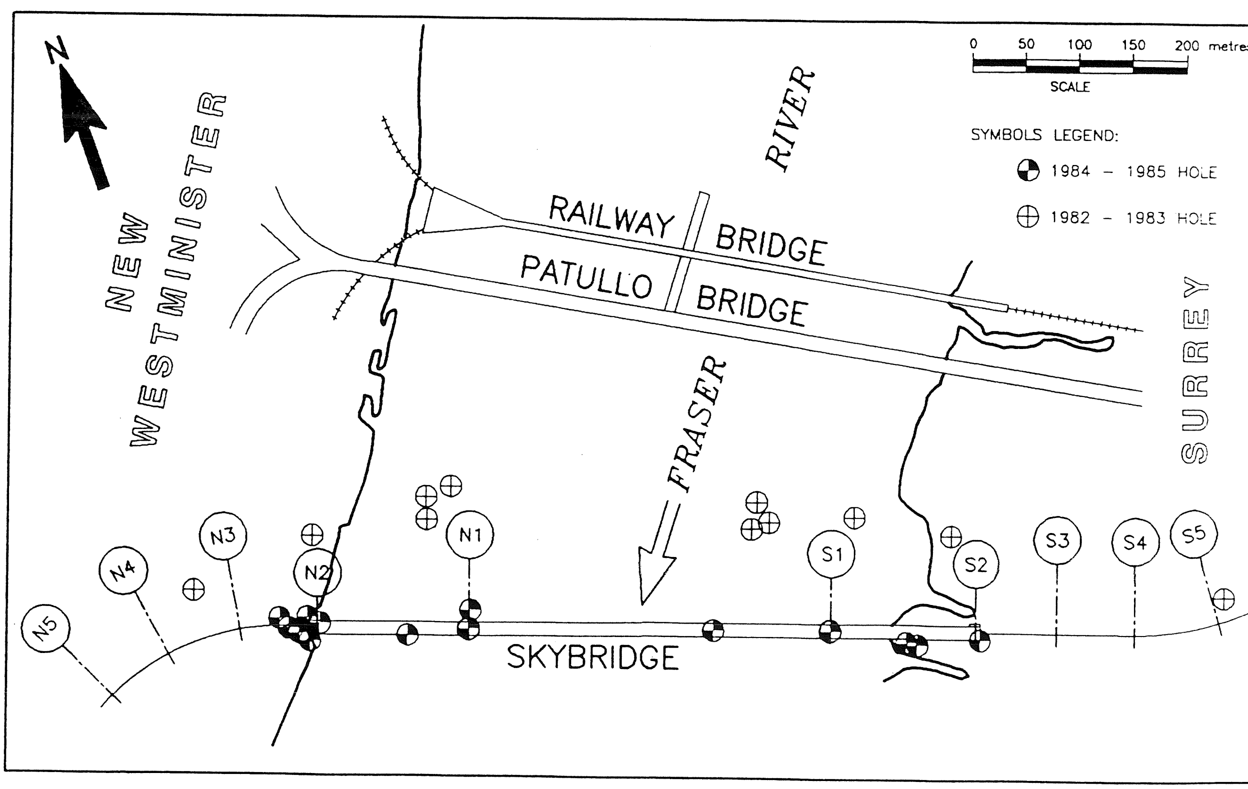

FIG. 2. Skybridge – Site plan showing drill holes

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CAD No. FIG_3.DWG FILE No. 1-2-8-3 APRIL 29, 1994

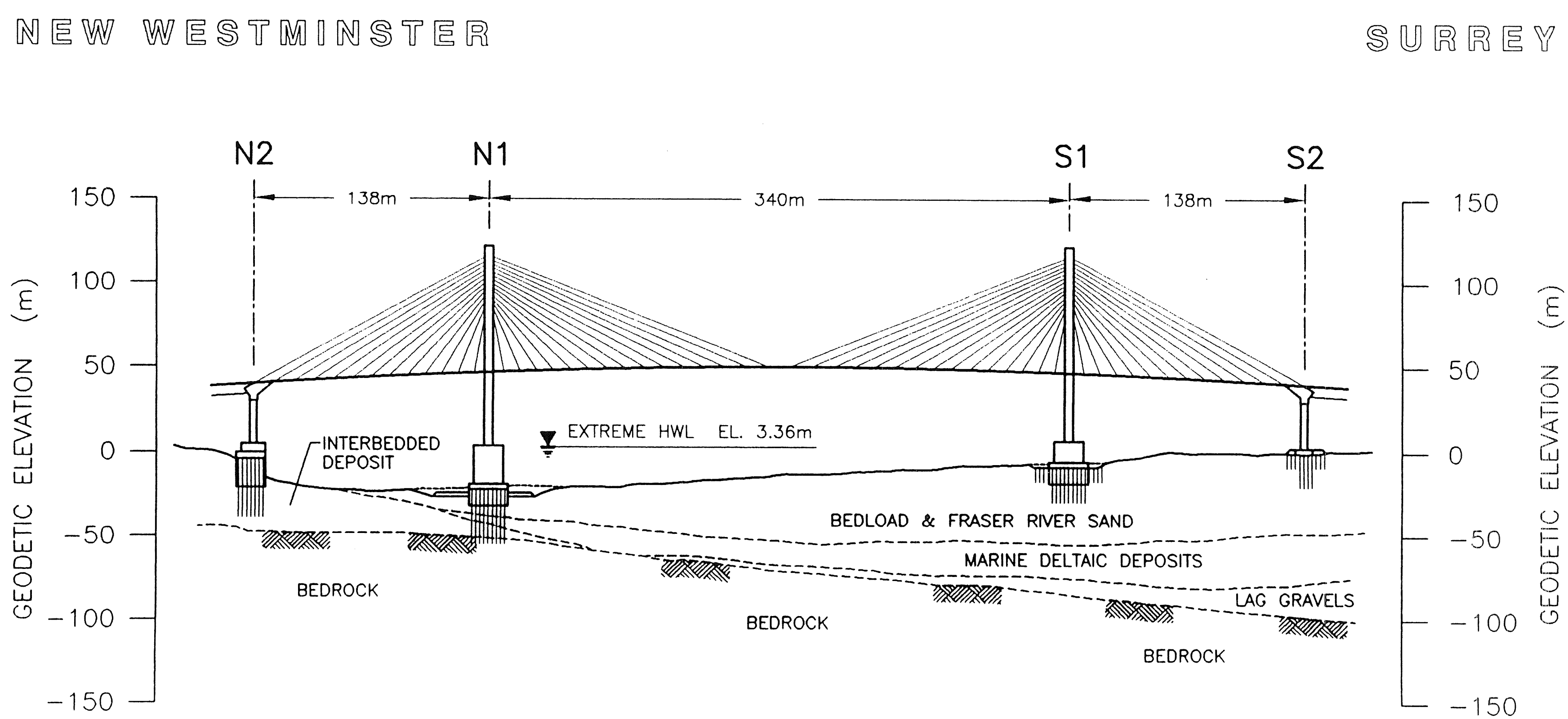

FIG. 3. Bridge elevation and geologic profile, looking upstream

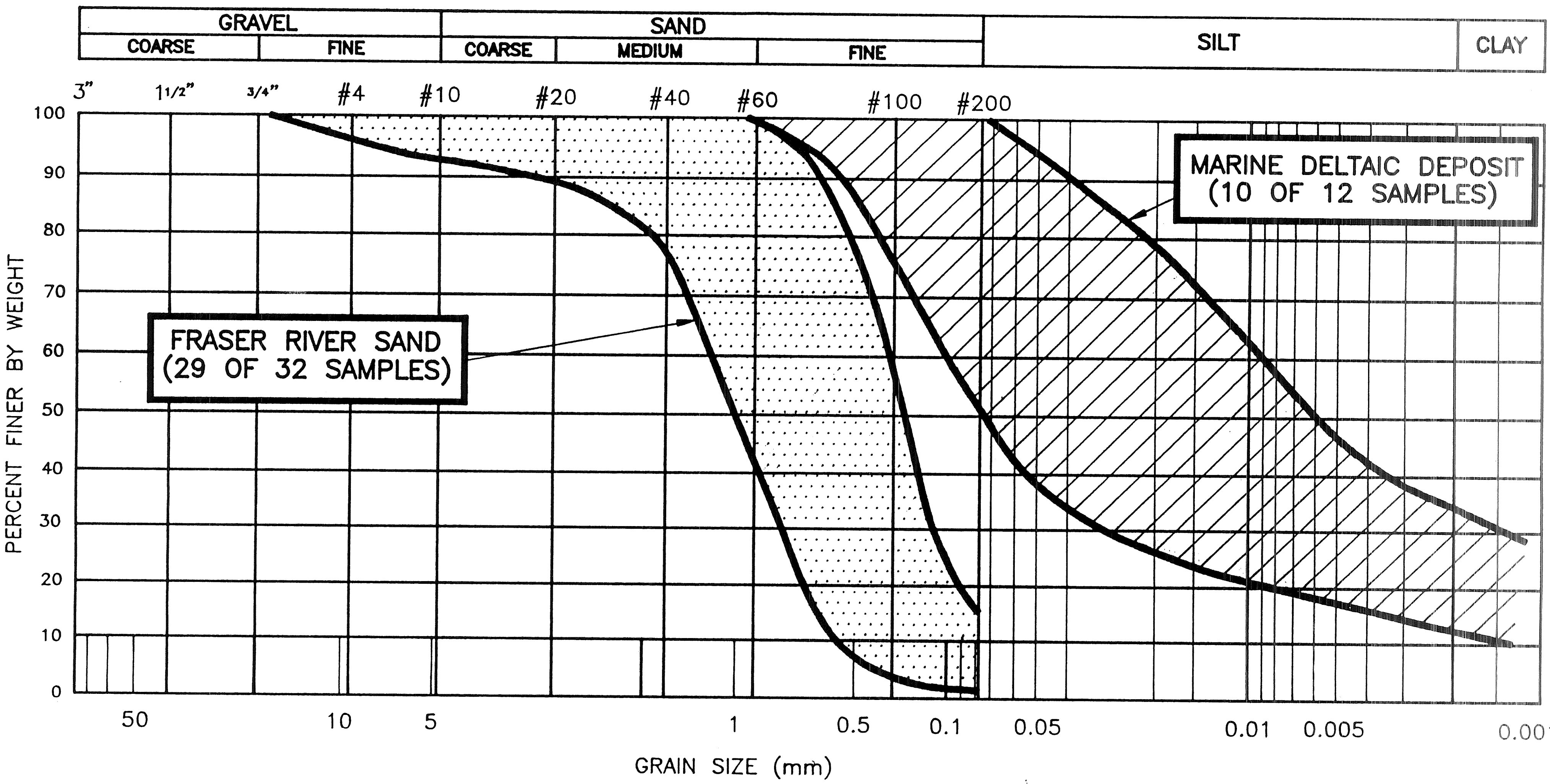

FIG. 4. Gradation envelopes for the two principal soil strata

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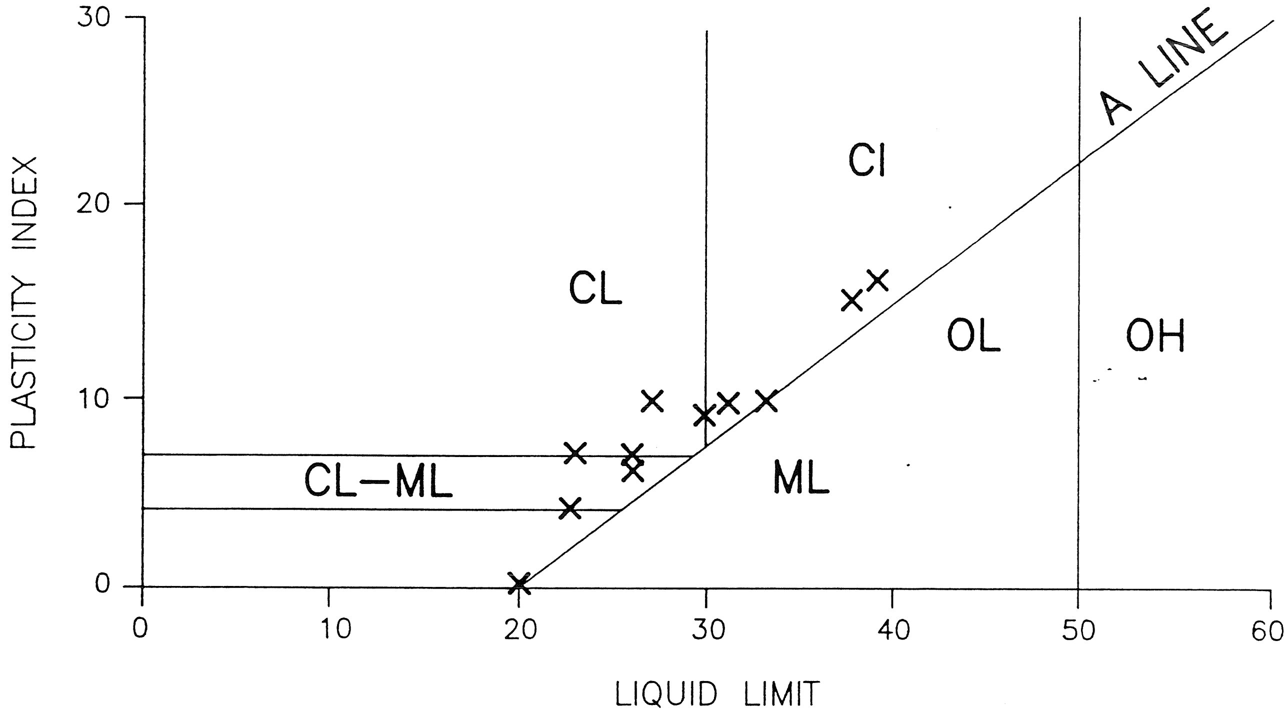

FIG. 5. Plasticity chart for marine deltaic deposit

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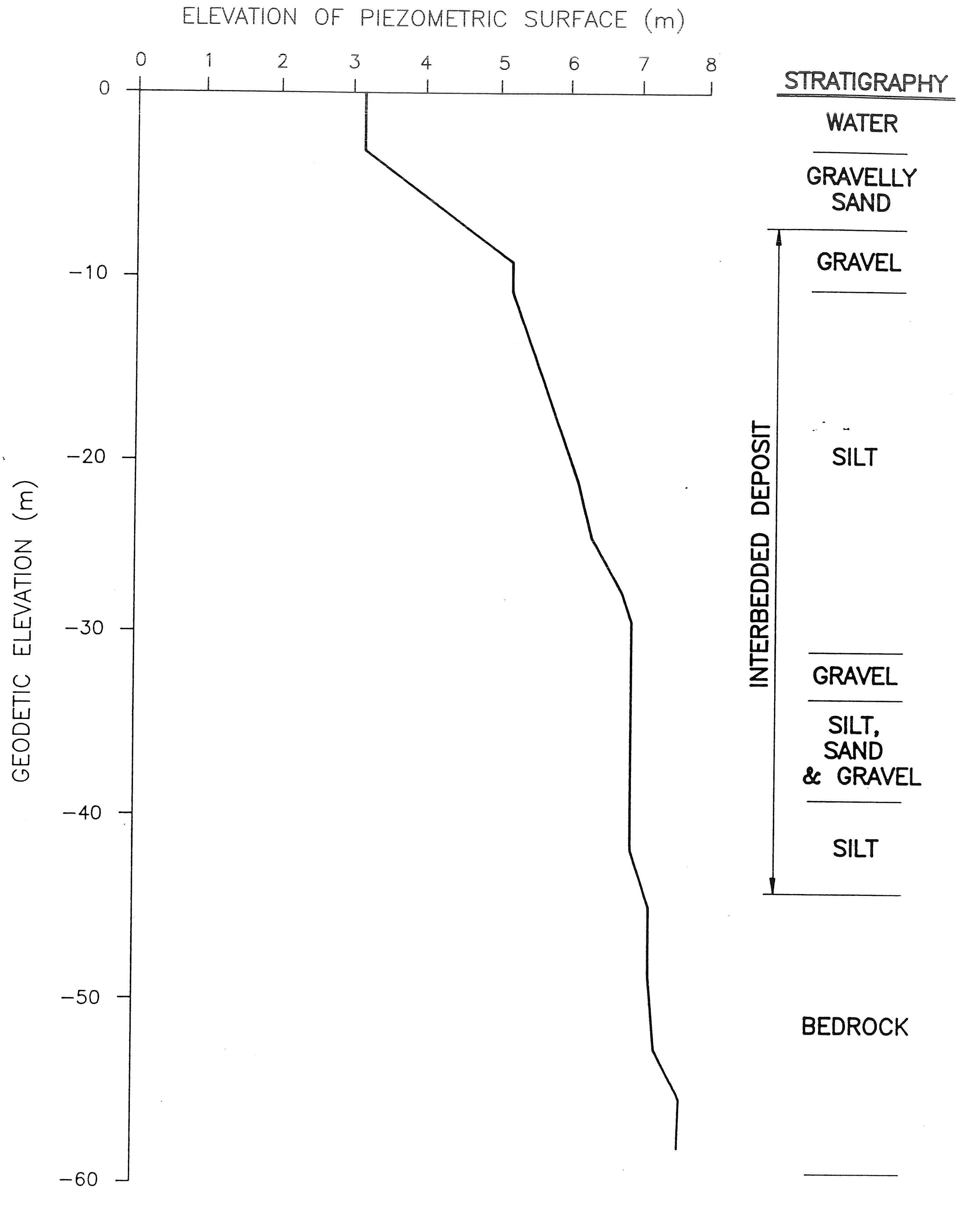

FIG. 6. Measured piezometric levels at Pier N2

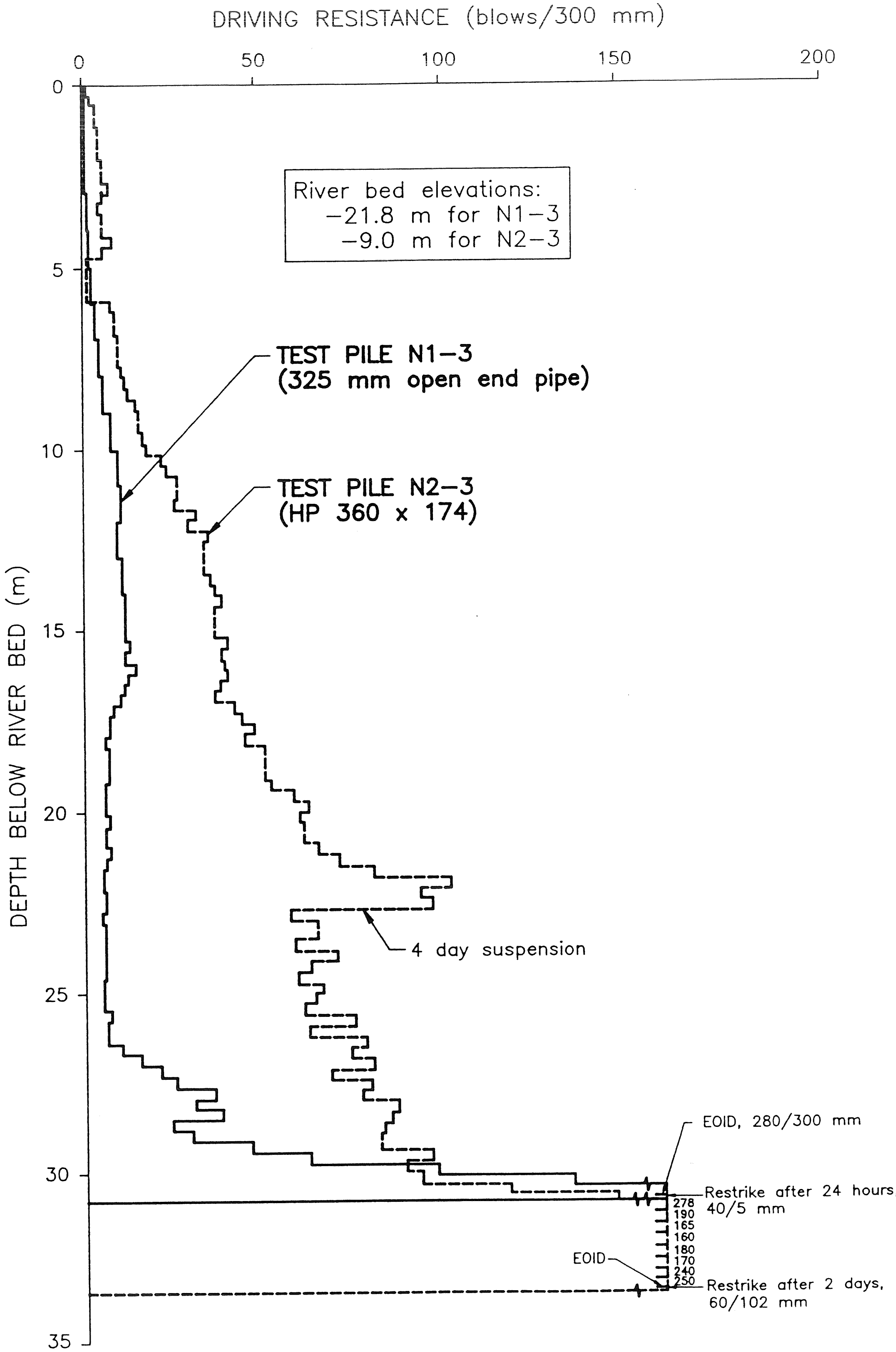

FIG. 7. Test Pile Driving Records

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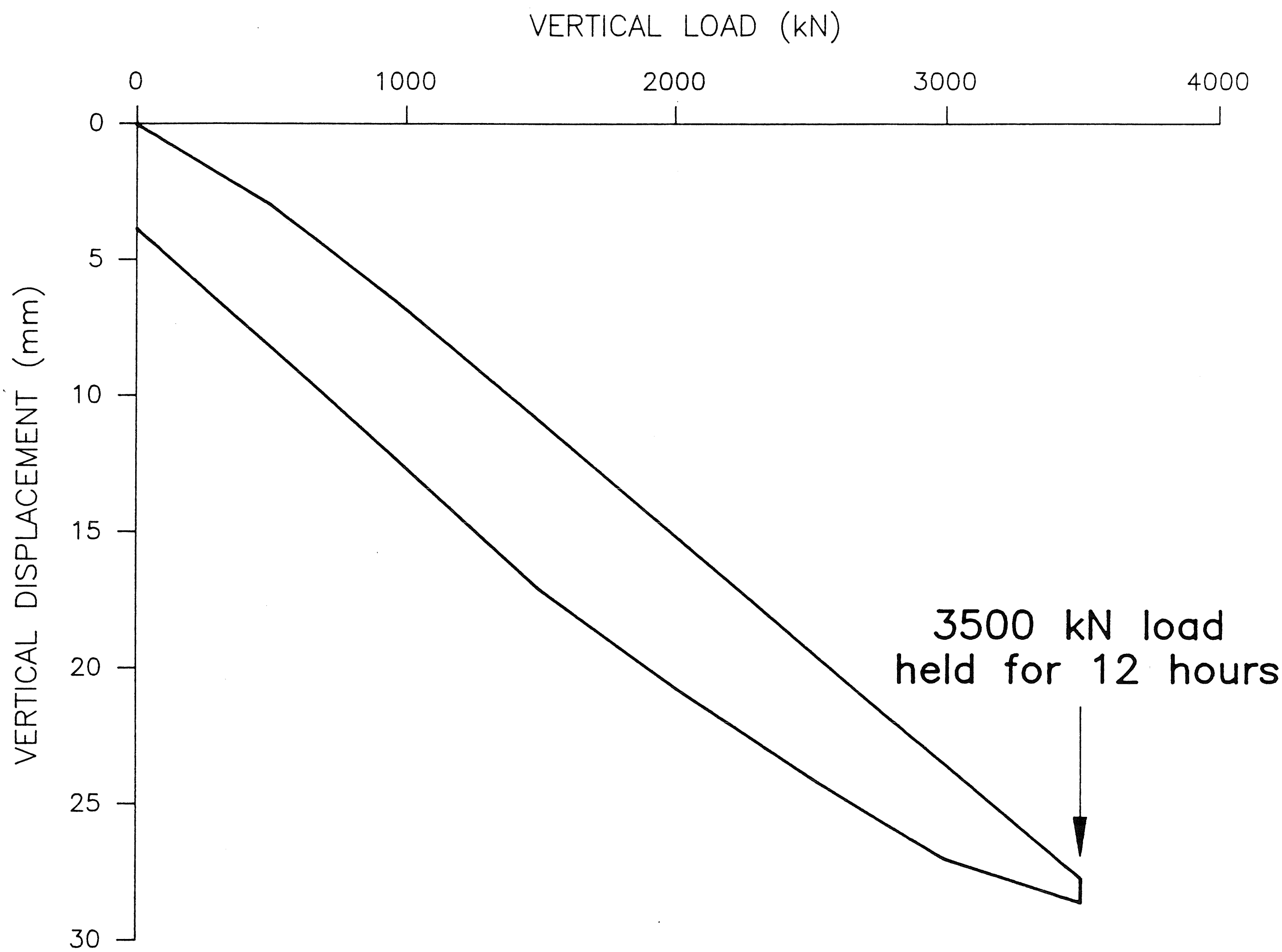

FIG. 8. Load versus settlement for Test Pile N2–3

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CAD No. FIG_9.DWG

FILE No. 1-2-8-3 APRIL 29, 1994

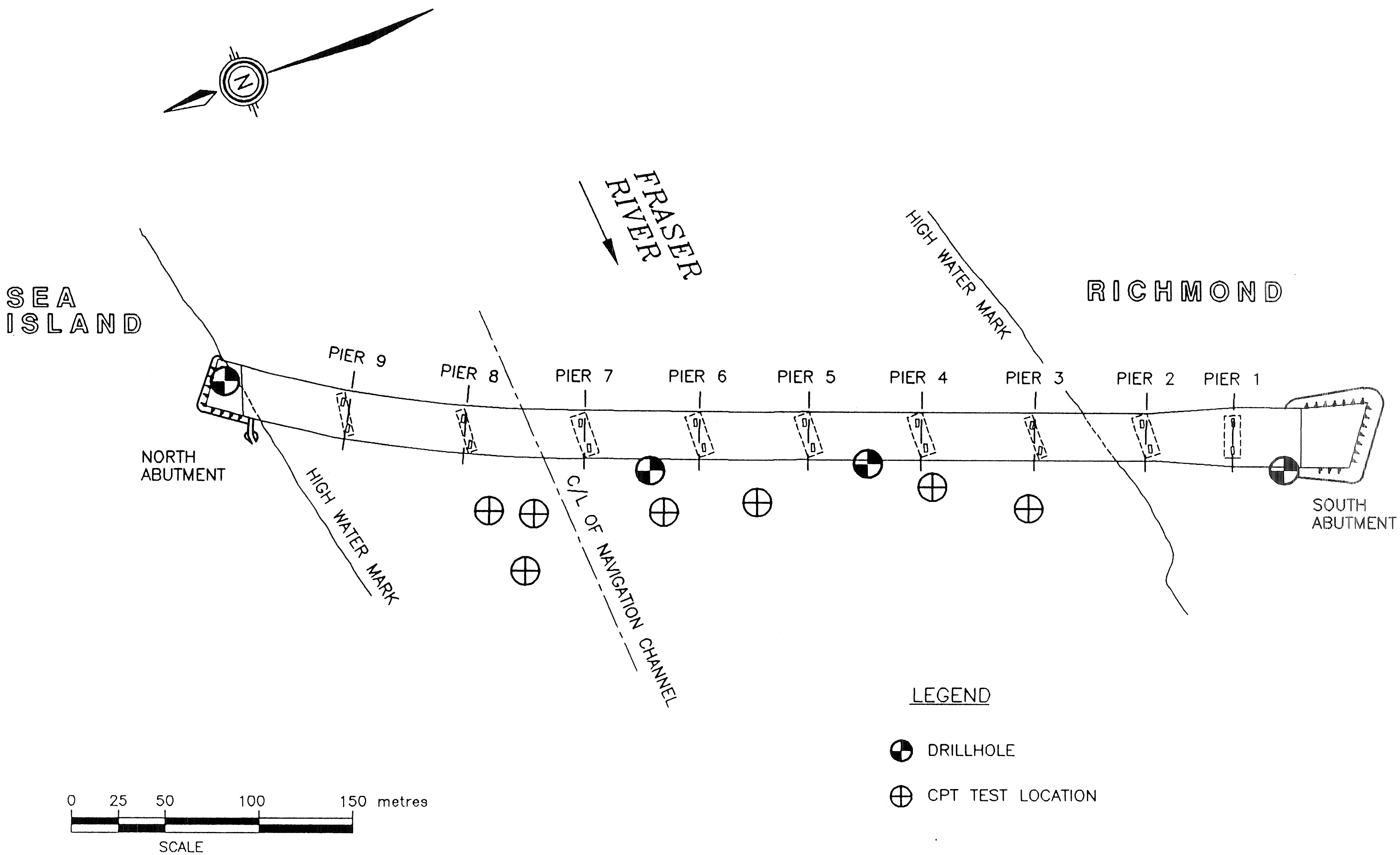

FIG. 9. No. 2 Road Bridge – Site plan showing testhole locations

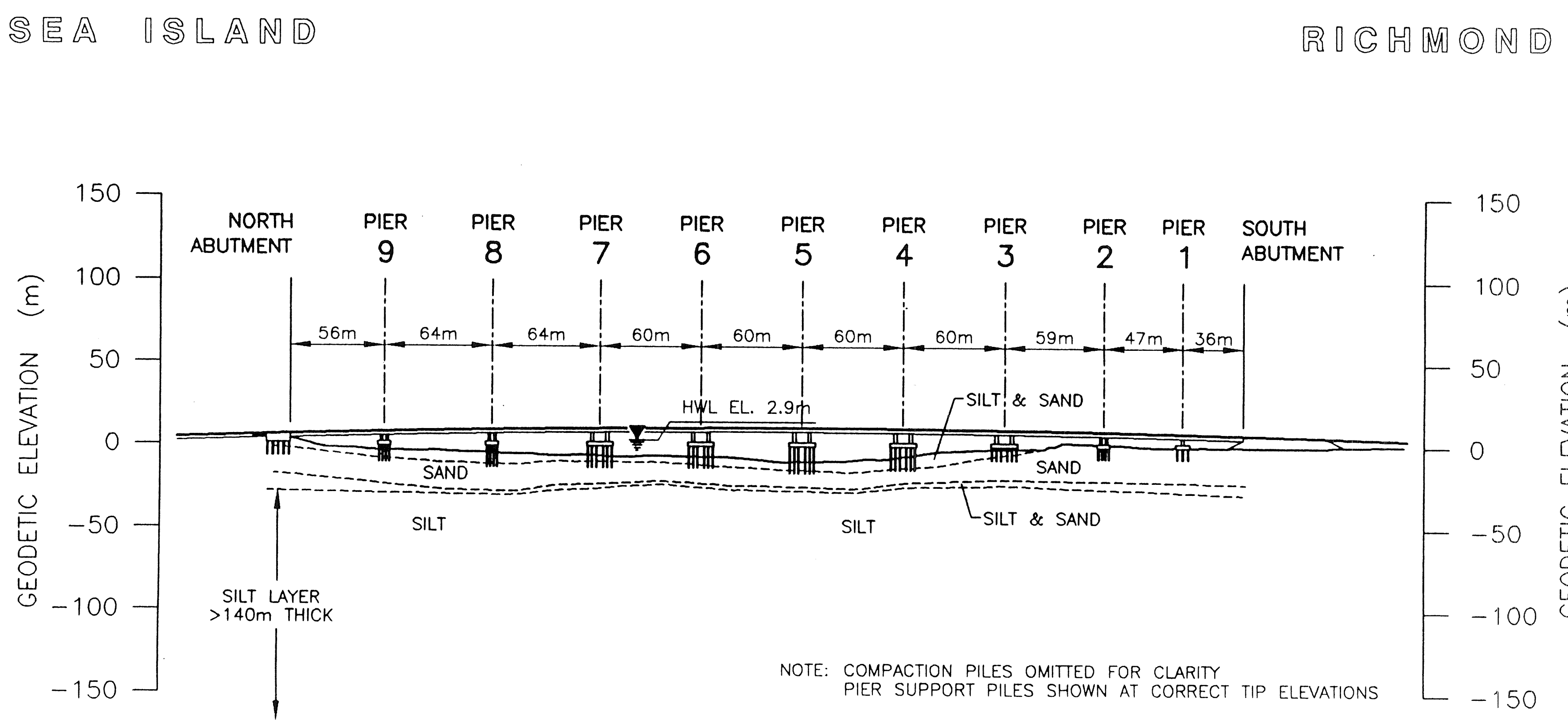

FIG. 10. No. 2 Road Bridge – Geological profile on bridge alignment

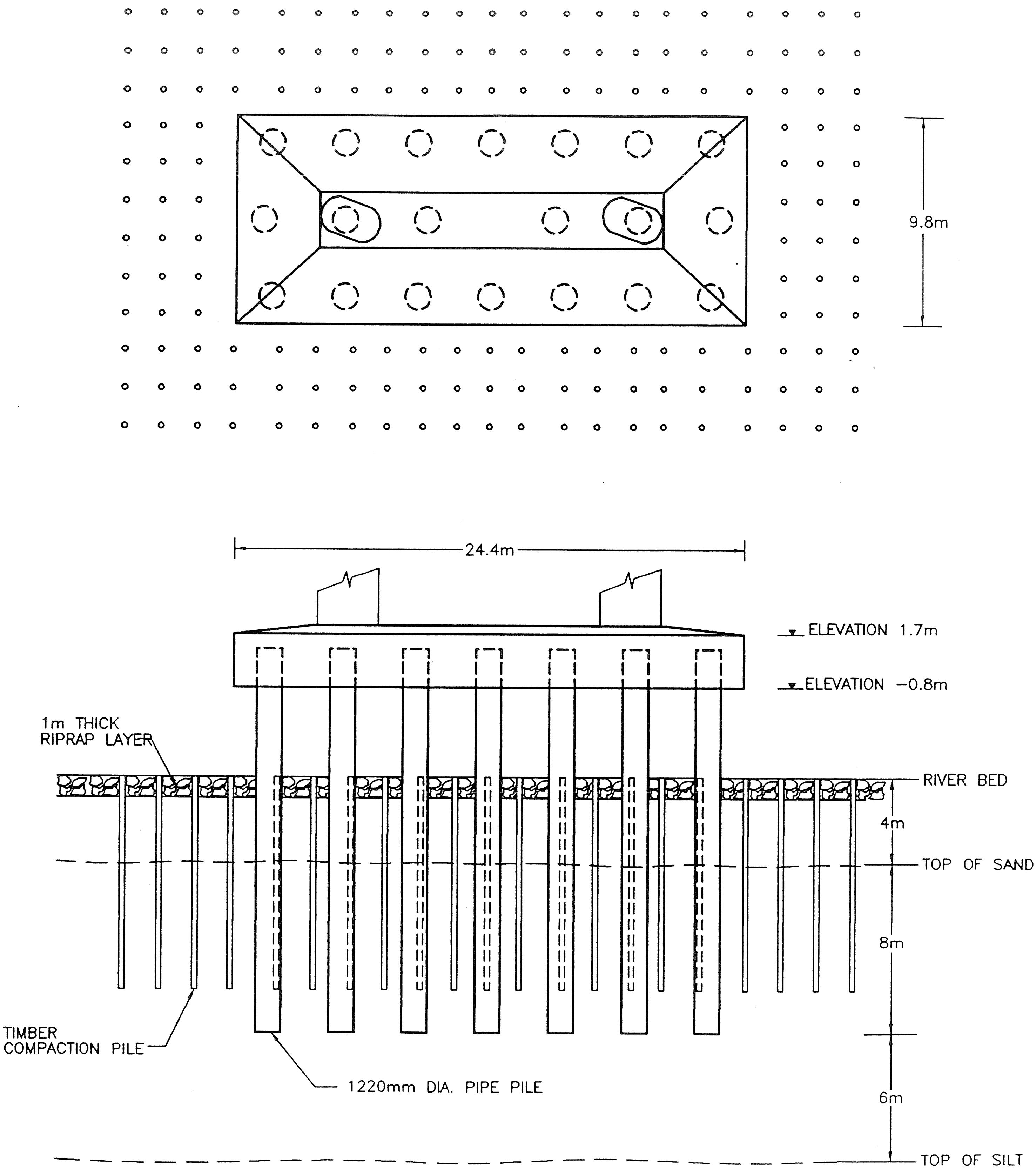

FIG. 11. No. 2 Road Bridge – Typical River Pier

April 28, 1994

FILE No. 1-2-8-3

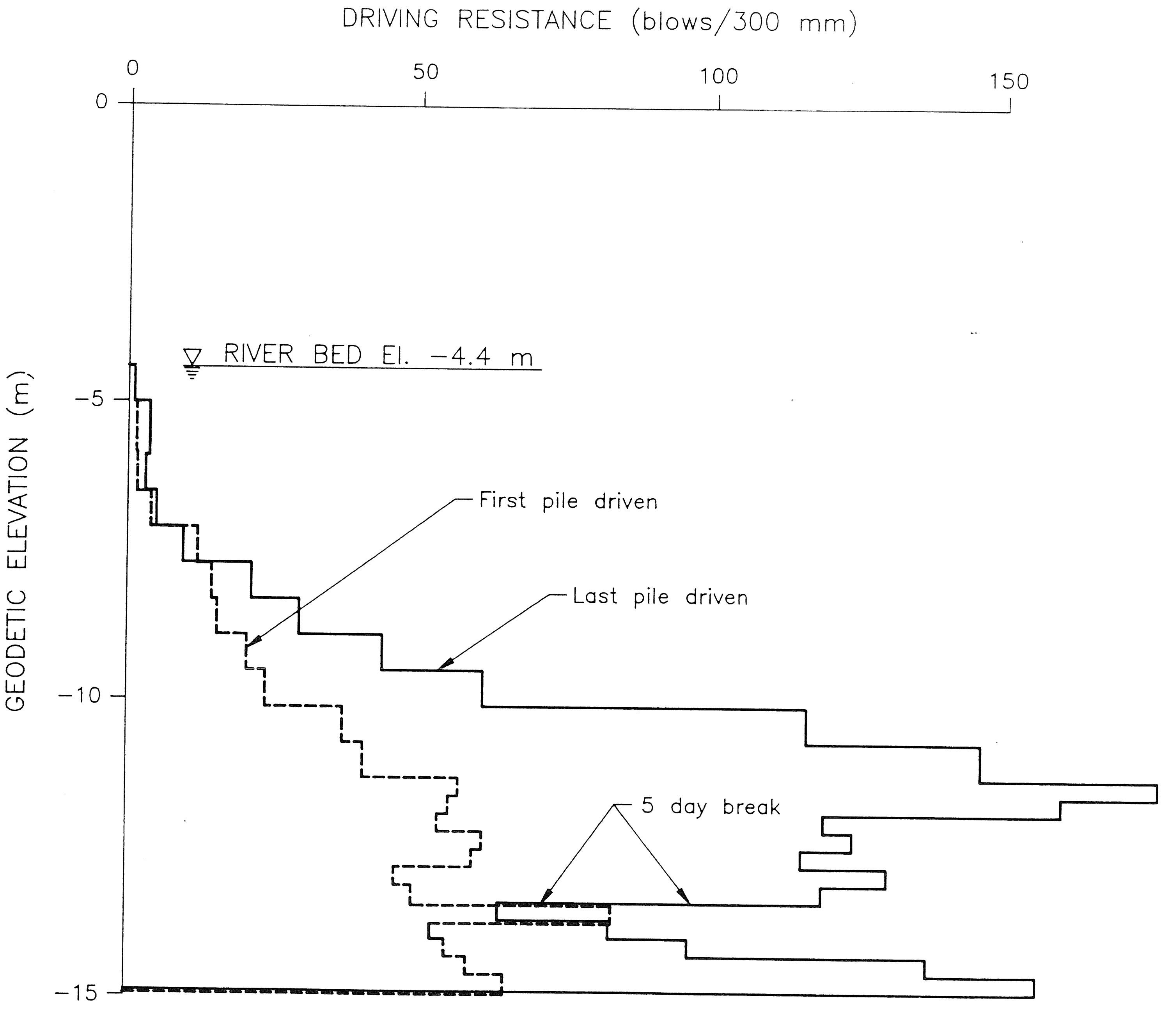

FIG. 12. No.2 Road Bridge – Typical Pile Driving Results (Pier 7)

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