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SYNOPSIS: A full-scale compressive load test was conducted on a drilled pier in the Pierre Shale Formation near Fort Collins, Colorado, to verify design parameters. The test pier was designed based on presumptive design criteria for both end-bearing and skin friction in the shale. The maximum test load of 6.7 MN (750 tons) resulted in a deflection of approximately 230 mm (9.0 in.). Instrumentation within the pier allowed determination of the actual end-bearing and skin friction values at various applied loads. Based on results of the test, production piers were redesigned for skin friction only and shear rings were added to enhance shaft resistance.

INTRODUCTION

Construction of a new industrial plant approximately 80 km (50 miles) north of Denver, Colorado, USA just east of the City of Fort Collins, Colorado involved installation of approximately 750 drilled piers bearing in the Pierre Shale Formation (Fig. 1). Piers were sized to support column loads ranging from approximately 2.2 to 8.0 MN (250 to 900 tons). Maximum allowable settlement of each pier was approximately 13 mm (1/2 in.). To optimize foundation design rather than rely on presumptive bearing pressures thought to be conservative, the designers recommended that a full-scale instrumented load test be conducted on at least one drilled pier. Considering the large number of piers involved it was anticipated that the load test would result in cost savings.

SUBSURFACE CONDITIONS

The site is underlain by Pleistocene Age glacial, alluvial and aeolian deposits, and the Cretaceous Age Pierre Shale Formation. The profile is relatively consistent across the site. Conditions near the test pier based on test borings B-9 and B-13 are described as follows and shown in Fig. 2.

Aeolian Deposits

The first ±3 m (10 ft) of soils encountered are silty clays with a trace of sand or gravel. The soils are visually classified as medium strength (stiff) and low plastic. They were transported and deposited by wind and can thus be referred to as loess. The upper surface of these materials has been modified and reworked by cultivation.

Alluvial Deposits

The clay is underlain by ±6 m (20 ft) of alluvial sands and gravels. In general these materials are fine to coarse-grained sands with some fine gravel. Layers of silty and/or clayey sands are noted. Blow counts indicate medium dense to dense conditions. Standard Penetration Test (N) values range from 16 to 58 in the two adjacent borings. Groundwater was encountered at the top of this layer in both of the adjacent borings and in the test and reaction piers.

Shale

The sand and gravel is underlain by the Pierre Shale to the depth of exploration, 28 m (90 ft). The upper portion of the shale has weathered to very stiff to hard, olive-tan, highly plastic clay with very fine sand. This weathered zone ranges in thickness from about 1.5 m (5 ft) at the test pier location to about 2.4 m (8 ft) at the nearest adjacent boring. The transition to the unweathered shale is gradual. The unweathered shale is a hard, dark gray, thinly bedded shale with some thin sandstone layers. Blow counts from the Standard Penetration Test range from about 50 blows for 150 mm (6-in.) of penetration to about 50 blows for 64 mm (2.5 in.) of penetration. Pressuremeter and unconfined compression tests in other borings indicates the average undrained shear strength of the unweathered shale is about 1.91 MPa (20 tsf). The water content of the unweathered shale averages 16 percent with a dry density of 2.22 Mg/m^3 (139 pcf). The Pierre Shale is reportedly 2,440 to 3,660 m (8,000 to 12,000 ft) thick and is interbedded with sandstones.

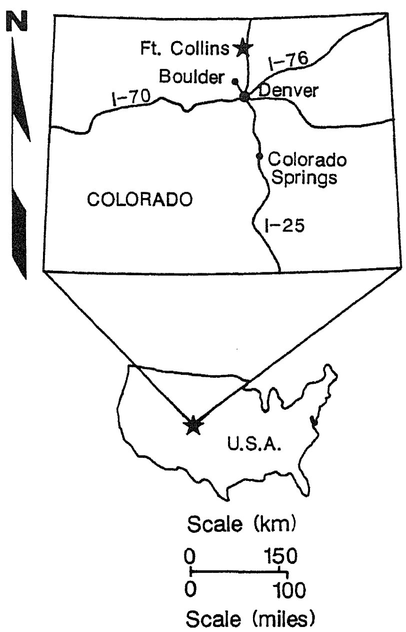
Groundwater

Groundwater was encountered at or near the top of the sand layer (el 1,525 m). The alluvial deposit is an aquifer used by local farmers for irrigation.

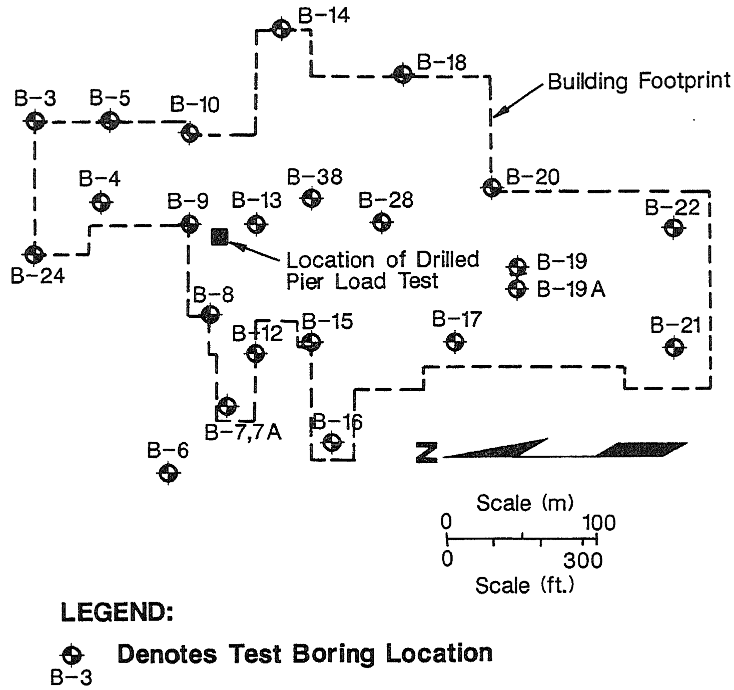
LOAD TEST SET UP

Reaction for the load test was provided by two nominally 762 mm (30-in.) diameter drilled piers each located 3.05 m (10 ft) on center from the test pier as shown in Fig. 2. Reaction piers were drilled to depths of 20 m (65 ft) forming rock sockets in the Pierre Shale about 9.2 m (30 ft) long. A 2.1-m (6.5-ft) deep reaction beam centered over the test pier spanned between the reaction piers.

The reference beams consisted of two W8x35 beams, one on each side of the test pier cap oriented normal to the alignment of the test and reaction piers. The reference beams were supported on two 457 mm (18-in.) diameter by 3.05 m (10 ft) deep drilled piers located 3.05 m (10 ft) on center from the test pier.



VICINITY MAP



SITE PLAN

Fig. 1 Vicinity Map and Site Plan

DRILLED PIER DESIGN

Drilled piers were designed on empirical rules based largely on Standard Penetration Test results. From previous experience with shale in the Denver area, empirical values for the allowable end-bearing and skin friction are calculated as follows (units are in tons per square foot):

$$qa = \text{Allowable end-bearing pressure} = \frac{N}{4}\ (\text{tsf}) \quad (1)$$

qa should not exceed 30 tsf (2.9 MPa)

$$fs = \text{Allowable skin friction} = \frac{qa}{10}\ (\text{tsf}) \quad (2)$$

Jubenville, et al indicates the same equations, although no limiting values were given.

The Standard Penetration resistance in the unweathered shale was generally in the range of 50 blows for 75 to 100 millimeters of penetration (3 to 4 in.) or an equivalent N-value of 150 to 200. Considering this, design values of qa and fs were 2.9 MPa and 0.29 MPa (30 tsf and 3 tsf), respectively.

LOAD TEST CONSTRUCTION

Construction of the load test consisted of three primary components: 1) construction of the test pier and two reaction piers with pier cap, 2) placement of the reaction beam, and 3) fabrication of the deflection reference system.

Installation of the drilled piers began on October 11 and was completed on October 13, 1983. Piers were installed by personnel and equipment of the Meredith Drilling Company. The contractor used a Williams LDH pier drilling rig to advance the shafts. Conventional flight augers, fitted with 50 mm (2-in.) wide high-strength steel teeth were used to excavate both the soil overburden and the shale bedrock. A 457-mm (18-in.) diameter auger was used to excavate the test pier, and a 813-mm (32-in.) diameter auger was used to drill each of the reaction piers. To provide clearance for the temporary steel casings, a 50 mm (2-in.) wide, angled "side cutter" tooth was inserted on the augers to increase the actual upper excavated shaft diameters to 508 and 864 mm (20 and 34 in.), respectively.

Temporary steel casings were used in each of the three piers. A 9.75 m (32-ft) long by 508 mm (20-in.) (O.D.) by 6.35 mm (0.25-in.) thick steel casing was used for the test pier and a 10.2 m (33.4-ft) by 813 mm (32-in.) (O.D.) by 6.35 mm (0.25-in.) thick steel casing was used for both reaction piers.

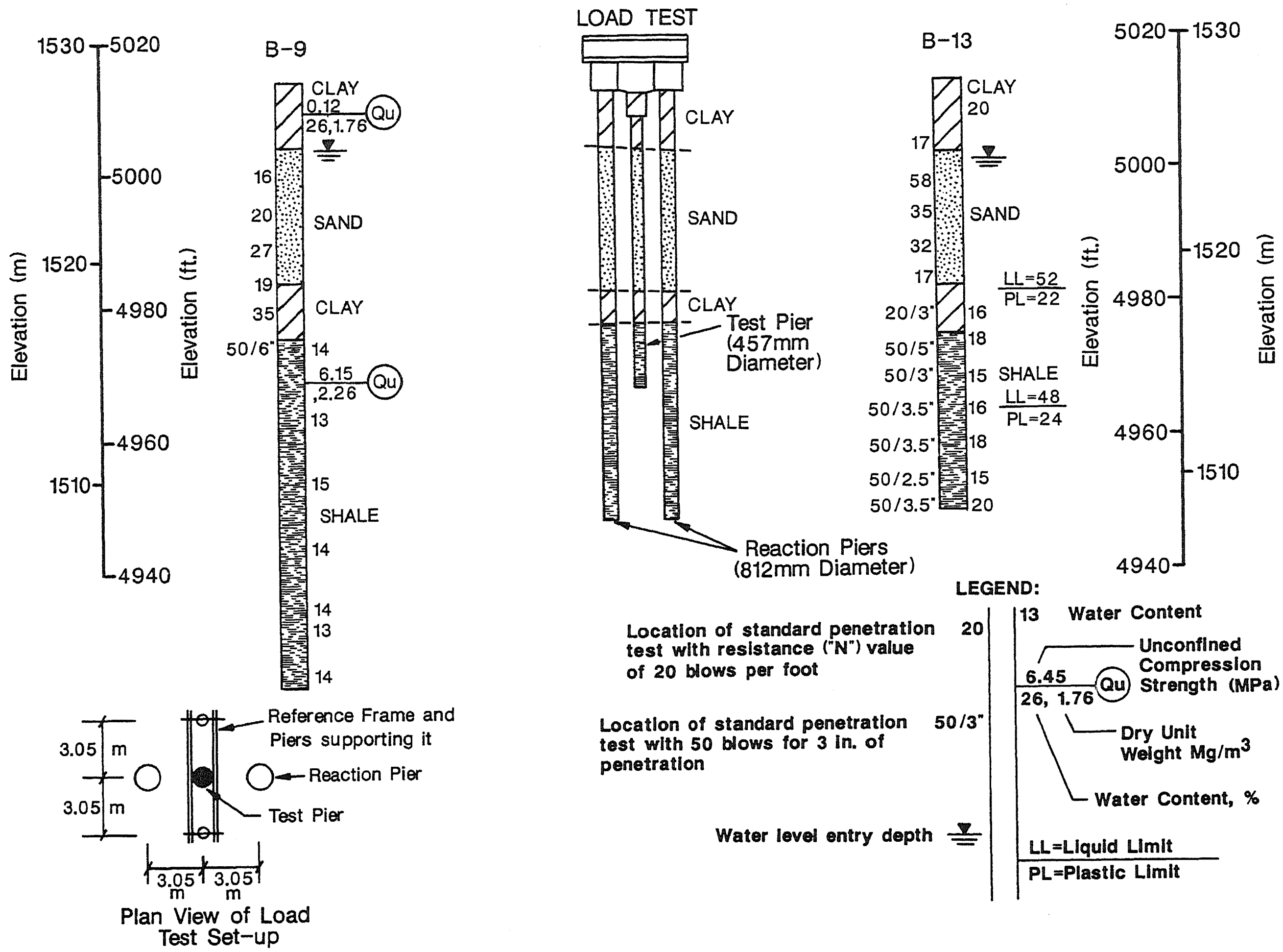


Fig. 2 Test Pier Subsurface Profile

Similar drilling procedures were used to construct each of the three piers. The augers were initially advanced through the surficial silty clay layer (3 m thick) to the top of the sand/gravel layer (±6 m thick) where groundwater first entered the shaft. At this depth the contractor added water to the excavation and mixed a drilling slurry. Varying amounts of the surficial clay spoil were added to increase the consistency of the slurry. Drilling continued through the sand/gravel layer and into the weathered shale (±1.5 m thick) by the slurry method. At a penetration of approximately 0.6 to 1 m (2 to 3 ft) into the weathered shale, the temporary steel casing was set and turned an additional 0.3 to 0.6 m (1 to 2 ft) into the weathered shale to form a watertight seal. The contractor then mechanically bailed out the slurry and continued drilling with the auger in the dry hole to the desired bearing elevation. The effectiveness of the casing/shale seal varied. Water flow through the unweathered shale into the excavated shafts was noted in the east reaction pier and in the test pier.

To minimize water softening of the base of the test pier, the drilling contractor halted drilling 0.6 to 1 m (2 to 3 ft) above the desired bearing elevation until the instrumented reinforcing cage was set and concrete arrived on-site. Prior to the final setting of the reinforcing cage, the electrical instrumentation and telltale were attached to the inside of the steel cage. After the instrumentation was attached to the steel reinforcement, the cage was lifted out of the hole, the remaining 0.6 to 1 m (1 to 3 ft) of shale was excavated, and the cage was reinserted into the hole. The delay between final drilling of the pier and placement of concrete was about 30 minutes. Concrete was then placed continuously to the bottom of the pier cap. In an effort to minimize the potential for disturbance of the electrical instrumentation, a full-length, "Elephant Trunk" was used to limit free-fall of the concrete. A log of the test pier is given in Fig. 3.

Concrete Testing

A total of six concrete cylinders were cast from the test and reaction piers. Cylinders were cured in both the field and laboratory and tested for strength and modulus at curing periods of approximately seven to eight days, sixteen to eighteen days, and twenty-two to twenty-four days. The average compressive strength and modulus at the time of the load test were 47.2 MPa and 9,930 MPa (6,840 psi and 1,440,000 psi), respectively.

INSTRUMENTATION

The instrumentation used on the pier load test was divided into three primary categories: 1) pier deformation measurement devices, 2) pier butt displacement measurement devices, and 3) load measurement devices. The two types of pier deformation measurement devices were strain gages and "Carlson Reinforced Concrete Meters." Pier displacement measurements were made using a wire gage, telltale, Brunson automatic level with optical tooling attachment and four optical scales, and three dial gages to measure the vertical displacement of the test pier cap. Load measurements were made using a hydraulic pressure gage and a 8.9 MN (1,000-ton load cell).

Strain Gages Devices

Five strain gages were installed in the test pier: one in the center of the test pier cap, one at the top of the weathered shale, one at the interface between the weathered and unweathered shale, one 1.5 m (5 ft) below the interface, and 2.7 m (9 ft) below the interface (0.3 m above the bottom of the pier). The strain gage device numbers and locations are shown on the Test Pier Log (Fig. 3).

The strain gage located 1.5 m (5 ft) below the interface of the weathered and unweathered shale was found to be defective during the initial loadings and the data are not presented herein.

The purpose of the strain gages was to measure axial deformation of the piers at discrete locations. These measurements were then used to evaluate load transfer with depth. The strain gages ("Micro Measurements" CEA-06-125UT-120) were mounted on 13 mm (0.5-in.) square bars 300-mm (12-in.) long suspended vertically in the center of the reinforcing cage. The linearity and calibration factors for each of the gages were determined in the laboratory using a specially made loading frame. The results of the strain gage device calibration program showed that the devices were linear and within the tolerances published for each of the gages.

Carlson Reinforced Concrete Meters

Three Carlson Reinforced Concrete Meters (Carlson gages) were used to measure axial deformations in the test pier. The first gage was placed near the center of the test pier cap. The second gage was installed at the top of the weathered shale and the third gage was installed at the interface between the weathered and unweathered shale.

The locations of the Carlson gages are shown in the Test Pier Log (Fig. 3). The Carlson gages were installed at the same elevations as strain gages 3A, 3B, and 3C. The purpose of the Carlson gages was to provide alternate measurements of the axial deformations at three locations to verify the measurements of the strain gage devices. A Strain Gage Bridge was used to monitor the output from the Carlson Gages. The Carlson gages were factory calibrated for stress, strain, and temperature. Factory calibrations checked in the laboratory prior to use on the project indicated that the Carlson gages were operating properly.

Telltale

A telltale was installed inside the reinforcing cage of the test pier. The purpose of the telltale was to monitor the displacement of the pier tip.

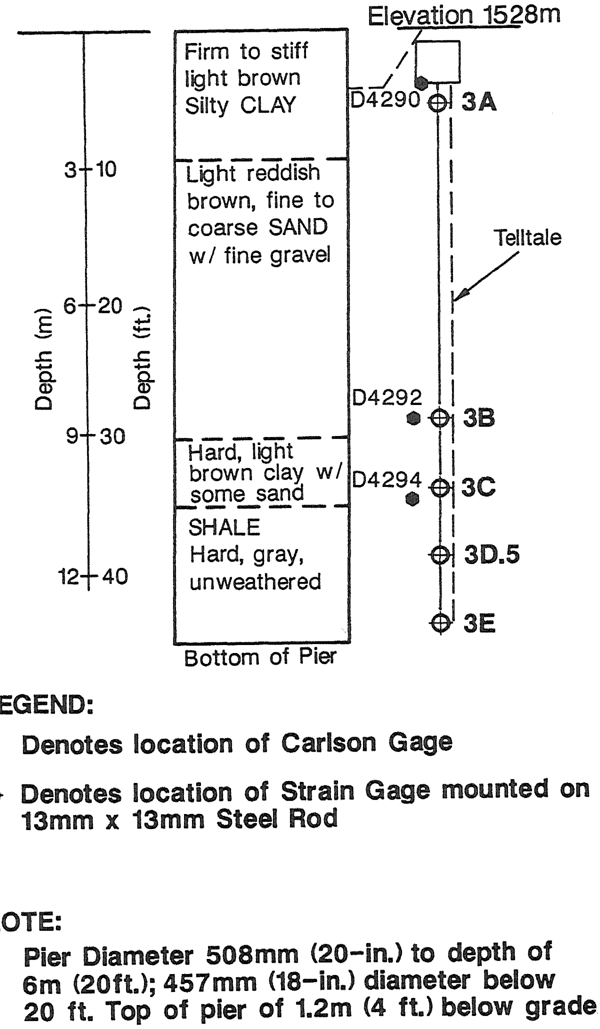


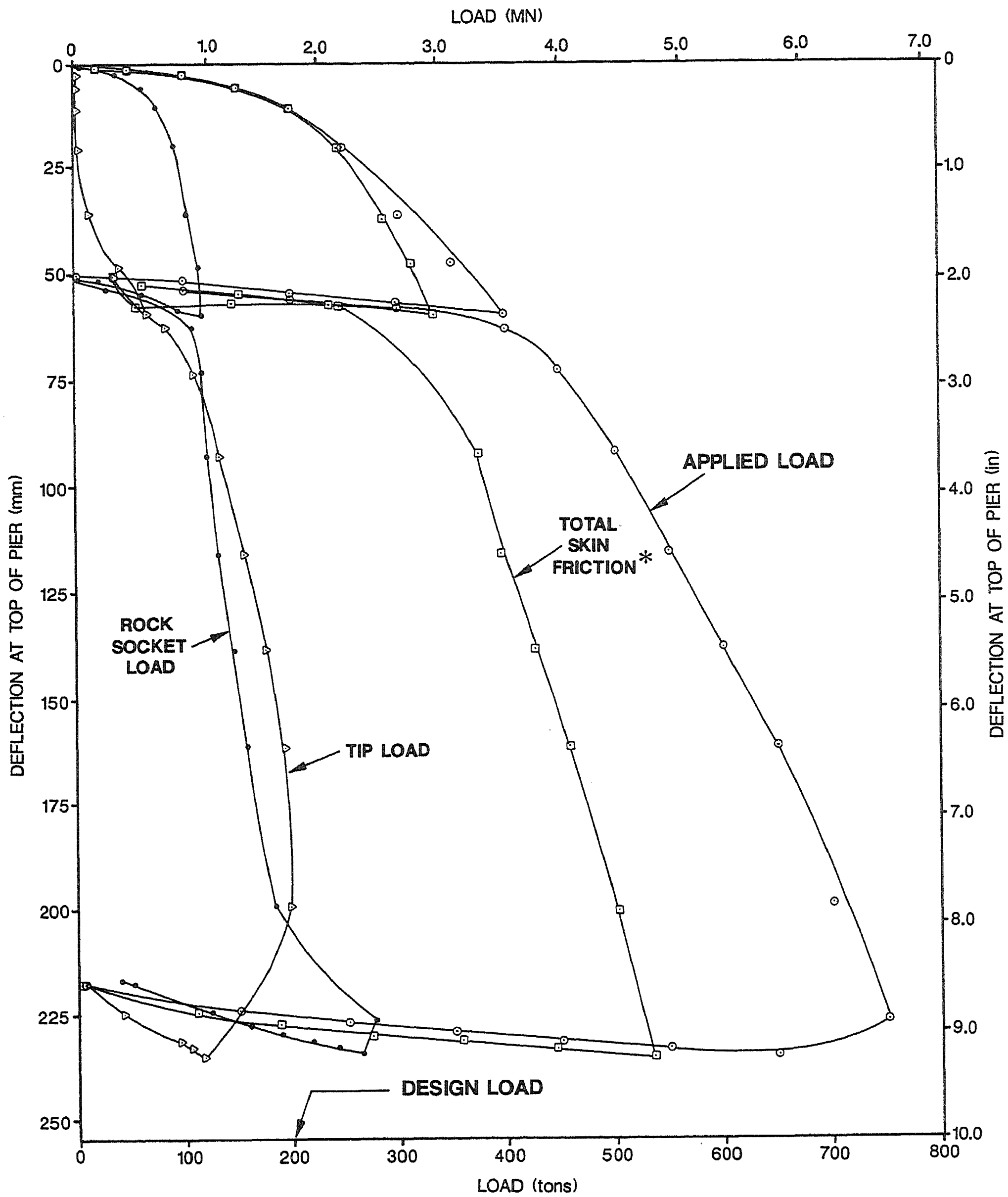
Fig. 3 Detailed Test Pier Log and Instrumentation

The telltale consisted of a 25 mm (1 in.) I.D. steel pipe which ran the length of the test pier at its center with a 13 mm (0.5-in.) square bar inside. The bar exited the pier horizontally through a pipe tee in the pier cap. Movement of the bar was monitored by a dial gage mounted on the reference frame.

Load Measurement Devices and Jack

Applied load was monitored by a calibrated hydraulic pressure gage on the jack and a 8.9-MN (1,000-ton) capacity electric load cell. Unfortunately the load cell malfunctioned during the test, necessitating use of the pressure gage to monitor load. The test loads were applied by a 10.7-MN (1,200-ton) hydraulic jack. An automatic hydraulic pump was used to increase, decrease, and maintain constant loads throughout the test using nitrogen gas.

To maintain a fairly constant temperature throughout the duration of the test and to provide protection for instrumentation and read-out devices, the test area was enclosed with reinforced polyurethane. A thermostatically controlled propane heater was used as required.



* Total skin friction assumed to be the difference between applied load and tip load.

Fig. 4 Load – Deflection Curves

TEST PROCEDURE

The testing system and loading procedures followed steps outlined by the ASTM standard D1143-81 "Piles Under Static Axial Compressive Load". Loading procedures used during the test were in general agreement with those outlined in Sections 5.1 and 5.3 of the ASTM standard, "Standard Loading Procedures" and "Loading in Excess of Standard Test Load", respectively.

As the test progressed, it was necessary to modify certain ASTM procedures. These modifications regarded the duration of time for maintaining constant load at each increment and the 12-hour holding period at 200 percent of design load. To better define the time-rate of settlement and load-transfer relationships, certain load increments were held in excess of the two hour limit defined in the above standard. The 12-hour holding period for 200 percent of the design load was dropped from the test procedure as a result of the magnitude of settlement already experienced and because the pier was later to be reloaded to failure.

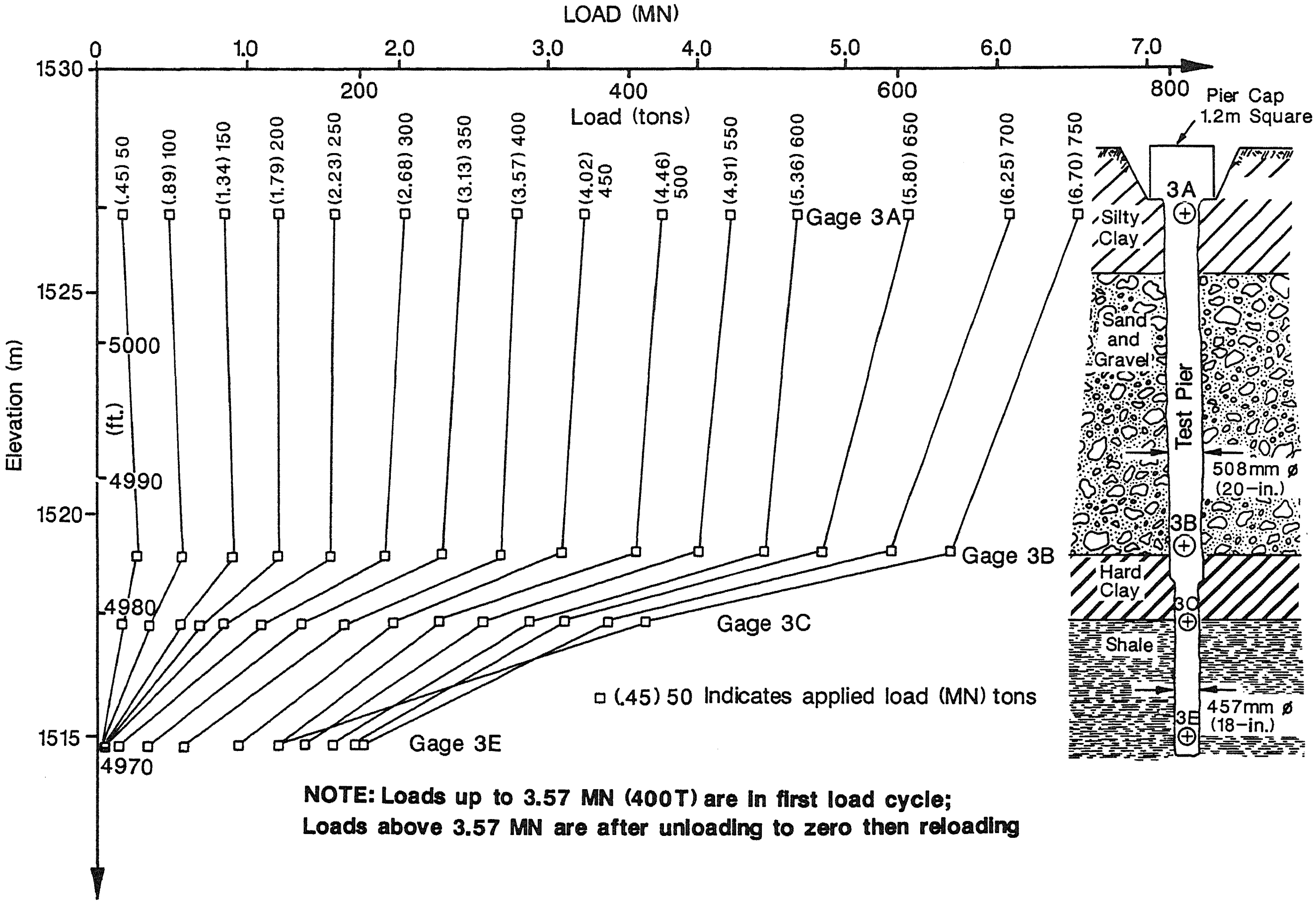


Fig. 5 Load Transfer with Depth

Another deviation from the ASTM standard regarded the percentage of design load to be applied in excess of the standard test load. Load increments of 25 percent design load instead of 10 percent were used to define the load-settlement curve for those loads in excess of the standard test load. This was done because of the relatively high maximum test load.

TEST CHRONOLOGY

The load test ran through the weekend of November 4, 5, and 6, 1983. The initial load was applied at 11:13 a.m. on Friday, November 4, and the test was completed 7:30 p.m. Sunday, November 6, 1983. The pier was loaded in 0.45 MN (50-ton) increments to 3.57 MN (400 tons), or 200 percent of the design load. Each load increment was maintained until the rate of axial deflection (herein called axial strain rate) was less than 0.25 mm (0.01 in.) per hour. The load was then removed in 0.89 MN (100-ton) increments to zero load. A minimum period of one hour was used for each of the increments during the unload cycle. The pier was left unloaded for one hour prior to the second load cycle.

During the second load cycle, the pier was loaded in 0.89 MN (100-ton) increments to 3.57 MN (400 tons). Each load increment was maintained until the axial strain rate was less than 1.27 mm (0.05 in.) per hour. From 3.57 MN to 6.7 MN (400 to 750 tons), the loads were added in 0.45 MN (50-ton) increments. The axial strain rate at 6.25 MN (700 tons) held constant at 2.5 mm (0.10 in.) per hour.

The pier experienced rapid and large axial displacement at the maximum load of 6.7 MN (750 tons). The load was maintained until the instrumentation could be read (about 30 minutes). The load was then removed in 0.89 MN (100-ton) increments to 1.34 MN (150 tons) with the last unload increment from 1.34 MN to zero load. The unload increments were maintained until the axial strain rate was less than 0.25 mm (0.01 in.) per hour (typically less than 20 minutes).

LOAD TEST RESULTS

Results of load test are given in Figs. 4 and 5 and information for selected loads are summarized in Table 1. These data are based on loads calculated from the strain gauges and top-of-pier (butt) deflections from dial gauge readings. Strain gauges and Carlson gauges were used to estimate load distribution within the pier. Values between corresponding devices agreed within about ±10 to 20 percent. Only the strain gauge data are shown because the greater number of strain gauges provided more information. Deformation

TABLE 1 Summary of Rock Socket Stress and End-Bearing Stress for Several Loads

Load MN (tons)	Butt Displacement mm (in.)	Rock Socket Bond Stress MPa (tsf)	End-Bearing Stress MPa (tsf)	Remarks
1.79 (200)	11.3 (0.443)	0.17 (1.8)	0.11 (1.19)	Design Load
3.57 (400)	59.4 (2.34)	0.27 (2.8)	3.62 (37.8)	Twice Design Load
5.36 (600)	138.2 (5.44)	0.34 (3.5)	9.46 (98.8)	Three Times Design Load
6.70 (750)	226.1 (8.90)	0.62 (6.5)	10.8(2) (112.4)	Ultimate Load

Notes: 1. The data for 1.79 and 3.57 MN are during the first load cycle.
2. Maximum end-bearing stress occurred at a total load of 6.25 MN (700 tons) compared to 6.7 MN (750 tons) for the maximum rock-socket bond stress.

measurements from the dial gages, wire and scale, and Brunson Level were all similar. Movement of the tip determined by the telltale was suspect and the telltale was not functional beyond a movement of 132 mm (5.16 in.).

Data indicate that at the design load of 1.79 MN (200 tons) butt deflection was approximately 11.2 mm (0.44 in.) or about 2.2 percent of the butt diameter of the shaft. Load at that point was carried predominantly by skin friction with negligible contribution from the pier tip. Loading to twice the design load or 3.57 MN (400 tons) increased butt deflections to 59.1 mm (3.3 in.) or 16.5 percent of the butt diameter. At 3.57 MN (400 tons), the load was resisted primarily by skin friction although the end-bearing contribution had begun to increase between approximately 2.2 to 2.7 MN (250 to 300 tons) and by twice design load was 0.60 MN (67 tons). The large deflection (16.5 percent of the butt diameter) necessary to mobilize end-bearing suggests slippage within the rock socket and possible compression of debris- or water-softened shale under the pier tip.

After loading to 3.57 MN (400 tons), the pier was unloaded, then reloaded to failure. Approximately 50 mm (2 in.) of permanent set occurred upon unloading from 3.57 MN (400T). During reload, the load deflection relationship increased at nearly a constant rate with approximately two-thirds of the load resisted in skin friction and about one-third in end-bearing. Between 6.25 and 6.7 MN (700 and 750 tons) strain rate increased indicating failure. Also at that point, the end-bearing contribution decreased and skin friction within the rock socket increased. The maximum end-bearing load of approximately 1.78 MN (199 tons) occurred at a total load of 6.25 MN (700 tons). This corresponds to a maximum end-bearing stress of approximately 10.7 MPa (112 tsf). The maximum skin friction in the rock socket was 2.46 MN (276 tons) which occurred at the maximum applied load, 6.7 MN (750 tons). The maximum bond stress within the rock socket at that load was approximately 0.62 MPa (6.5 tsf). This maximum bond stress corresponds to about 33 percent of the undrained shear strength of the shale.

DISCUSSION

The load-deflection relationship up to the design load of 1.79 MN (200 tons) was within the criteria established by the client. However, above the design load deflections became much greater. These larger deflections are believed to have been caused by slippage of the shaft in the socket. Air and water slaking of the pier shaft during construction may have contributed to shaft slippage.

The test results raised concern that production piers might experience large settlements if constructed with a smooth socket. To limit settlement, it was felt that the shearing resistance between the pier shaft and shale must be increased. A practical means to accomplish this was by use of shear rings which would "key" the pier shaft into the shale. Shear rings would also mobilize a greater percentage of the shear strength of the shale as shown by Horvath, et al. Tests by Horvath, et al (1983) showed that skin friction of 40 to 60 percent of the shear strength of the rock or .76 MPa (8tsf) to 1.15 MPa (12 tsf) in this case was reasonable using roughened sockets. Tests on piers with shear rings by Glos, et al indicated that up to 90 percent of the shear strength of the rock was mobilized. On the other hand, it was felt that there was little that could be done practically to increase end-bearing, especially at small deflections.

Based upon the results of this load test and the references cited, the design parameters were revised for the production piers. It was recommended that production piers be designed for skin friction only, based on an allowable value of the 0.48 MPa (5 tsf) and that shear rings be installed in each pier. It was recommended that shear rings be spaced at approximately 610 mm (24 in.) along the rock socket and that each shear ring be approximately 50 mm (2 in.) deep by 100 mm (4 in.) high. A minimum socket length of 3 m (10 ft) was specified.

In addition, it was recommended that piers be concreted immediately upon excavation of the rock socket to minimize deterioration of the shale by slaking.

PRODUCTION DRILLED PIER CONSTRUCTION

Approximately 750 drilled piers varying in diameter from 610 mm (24 in.) to 1.22 m (48 in.) were constructed between February 21 and May 2 of 1985. Typical production using 4 rigs was 18 piers per day with a maximum production rate of 33 piers per day. Piers were installed by similar techniques as the test pier, i.e., slurry drilling and casing through the loessial and granular formations and drilling the rock socket in the dry using earth augers. Shear rings were cut with simple attachments to the drill tools with a minimal increase in time and cost over drilling a smooth rock socket.

The cost of the drilled pier load test including engineering was approximately $100,000. The modification in the design resulting from the load test, however, was estimated to have reduced construction cost by about $400,000.

CONCLUSIONS

This study leads to the following conclusions.

> The Pierre Shale is apparently sensitive to air and water slaking which can increase deflections of smooth-socketed drilled piers beyond tolerable values.
>
> Designing based on empirical rules in the Pierre Shale could lead to piers which experience excessive deflections.
>
> Shear rings are recommended on drilled piers in the Pierre Shale to increase bond resistance and minimize deflections. The contribution of end-bearing at small deflections is negligible.
>
> A well instrumented load test for a major project may result in a safer design and cost savings.

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