U.S. Department
of Transportation
**Federal Highway
Administration**

Spread Footings for Highway Bridges

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

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FOREWORD

This report presents the results of a comprehensive investigation of the performance of spread footings as a support for highway bridge abutments and piers. The observations included precise measurements of settlement, tilt, contact stresses, and applied loads. Comparisons between predicted settlement calculations and actual measurements were made to evaluate several commonly used predictive techniques. This report will be of interest to bridge engineers and geotechnical specialists concerned with the design of bridge foundations.

Sufficient copies of the report are being distributed by FHWA Bulletin to provide a minimum of two copies to each regional and division office, and three copies to each State highway agency. Direct distribution is being made to division offices.

Charles F. Galambos / for
Richard E. Hay, Director
Office of Engineering and Highway
Operations Research and Development

NOTICE

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16. Abstract

A long-term study of the settlement performance of 21 bridge foundations supported on cohesionless soil--sand or silt-- was completed to provide a reliable data base for engineering evaluation. The observations documented the successful performance of the foundations where total average settlement of 0.75 in was recorded while an average of only 0.25 in of settlement was noted after construction of the bridge decks. Geotechnical engineering methods for prediction of settlement of foundations on sand were evaluated with five methods compared in detail to observed data. Recommendations for design are provided.

Geotechnical instrumentation monitoring systems for observation of settlement and tilt and related parameters are described along with preparation of a computerized data base for data storage and retrieval.

Design recommendations for foundations on rock as well as risk-based design concepts are presented. Related risk based documents include:

Report No.	Title
FHWA/RD-87/010	Geotechnical Risk and Reliability: The State of the Art
FHWA/RD-87/011	Geotechnical Risk Analysis: A User's Guide

17. Key Words	18. Distribution Statement
Settlement; spread footing foundations; risk analysis; bearing capacity of rock; geotechnical instrumentation	No restrictions. This document is available to the public through the National Technical Information Services, Springfield, Virginia 22161

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METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km

AREA

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha

MASS (weight)

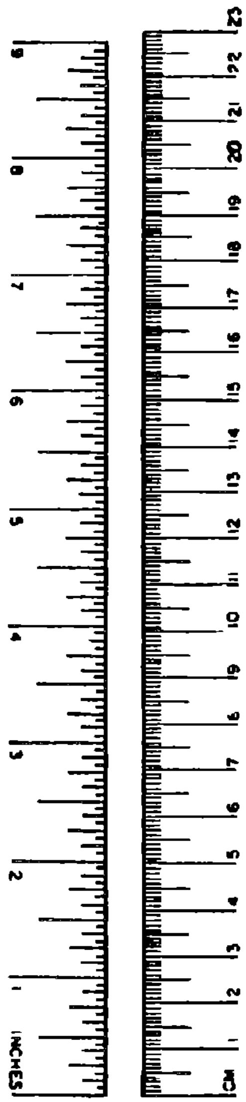
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t

VOLUME

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3

TEMPERATURE (exact)

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi

AREA

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	

MASS (weight)

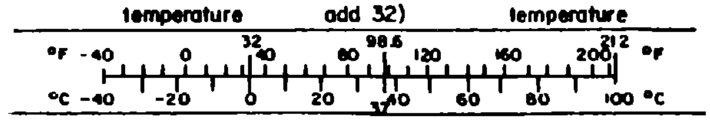
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	

VOLUME

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ml	milliliters	8.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3

TEMPERATURE (exact)

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION

1.1 BACKGROUND

The selection of a foundation system for support of a highway bridge involves consideration of performance and cost. By performance, the foundation must provide the bearing capacity required to support the piers, abutments and deck and must not develop excess settlement which would damage the structure including bridge decking and related wing walls etc. In situations where the choice of foundations is obvious, the designer does not have to make a difficult choice; e.g., the use of piles to support a foundation overlying soft compressible organic soil or clay or the use of footings on rock. However, in other subsurface conditions, particularly cohesionless sand or silt, the designer can face a situation where cost and performance must be more carefully evaluated. The cost savings for use of a shallow spread footing foundation may be calculated with some confidence. The designer must also be able to predict and evaluate performance, particularly settlement performance with similar confidence.

This study of the performance of bridge foundations on sand was undertaken to provide a case history basis for understanding the actual time-settlement behavior of typical spread footing foundations on sand or cohesionless soil. In addition, standard geotechnical methods for prediction of settlement were compared to the actual settlement.

During a period of approximately 3 years, 21 foundations were monitored from initial construction through completion and actual use. The bridges performed satisfactorily with post deck settlement averaging about 0.25 in. Methods for prediction of total settlement utilized readily available parameters such as standard penetration resistance (SPT or N values) and cone penetration data to evaluate soil compressibility and allow empirical prediction of settlement. Predicted total settlement was, on average, within 0.4 in. of observed total settlement.

1.2 PURPOSE AND SCOPE

The research study involved several subtasks as outlined below:

- Develop and implement a geotechnical instrumentation monitoring system to record settlement, tilt and related performance of bridge foundations. Ten case history bridge sites were selected and monitored.

- Obtain reliable data on the settlement performance and develop a computerized data base system for storage and retrieval and analysis of the data. The program was developed and used to store and present data for the report. Copies of the program are available.

- Review and evaluate the methods for predicting settlement of footings on sand and select appropriate methods to predict settlement behavior. Ultimately five methods were selected for study.

- Prepare a manual for application of risk-based analysis methods to geotechnical design problems with particular application to shallow foundation design. The users manual was completed as a separate document and is available through NTIS.

- Design of shallow foundations on rock was also reviewed. Design recommendations were provided linking conventional geology concepts, rock core analysis and allowable bearing capacity for use by bridge foundation designers.

Each of the tasks have been presented in separate chapters of the final study report. In addition, a 30-minute video tape presentation was prepared summarizing the results of the study and indicating the cost benefits of spread footings.

1.3 ACKNOWLEDGEMENTS

The Departments of Transportation in each State contributed greatly to this study, cooperating and assisting with the implementation of instrumentation program. Contract documents prepared by Haley & Aldrich, Inc. were issued through each State as part of bid documents or through the addenda process and established an orderly method for interaction with the bridge contractors. The assistance of each Department with these practical contract matters was key to the success of the program. The selection of the 10 case study bridge sites was completed after a review of bridge design and construction schedules through the New England States. Case study bridges were ultimately selected in Connecticut, Massachusetts, New York, Rhode Island and Vermont.

The Transportation Departments of several additional States including Maine, New Hampshire and Pennsylvania, expressed interest in the program but unfortunately their construction schedule did not include appropriate bridge sites in the time frame required for the research project.

2. MONITORING BRIDGE FOUNDATION PERFORMANCE

2.1 INTRODUCTION

General

The following report sections provide a summary of a field instrumentation and monitoring program completed on ten selected highway bridges. The program provided well-documented settlement performance data on the initial through post construction stages of each of these bridges so that the actual settlement of each bridge could be compared with the predicted settlement using the appropriate analysis procedures.

Objectives and Scope

The objective of the bridge foundation performance monitoring program was to provide reliable performance data to confirm that spread footing foundations bearing in sand can support bridges equally well compared to more expensive piles or other deep foundation systems. In order to accomplish this task, the following program was undertaken:

- Select ten candidate bridges for performance monitoring to be constructed in a time frame consistent with the anticipated duration of the proposed study.
- Determine important parameters to be monitored to document satisfactory foundation performance.
- Develop a basic, low cost and reliable instrumentation system to monitor important behavioral parameters.
- Install the instrumentation system at each of the candidate bridges.
- Monitor candidate bridge performance both during and following construction.
- Develop and provide a computerized data storage and retrieval system for use in processing and summarizing the data obtained.

2.2 SELECTION OF CANDIDATE BRIDGES

General

Proposed bridge construction plans and construction schedules were reviewed at the Departments of Transportation in the six New England States, New York and Pennsylvania to select a total of ten candidate bridges to be instrumented and monitored. Evaluation criteria were established in order to review each potential bridge site prior to the selection of the candidate bridges.

Evaluation Criteria

Potential bridge sites were reviewed in conjunction with the following criteria:

a. Bridge to be located within the New England or Middle Atlantic States.

b. Bridge to be representative of general highway construction.

c. Bridge foundations to consist of spread footings bearing in granular soils.

d. Basic test boring and laboratory soil test data had to be available.

e. Design and construction schedule to be within a time frame consistent with the anticipated duration of the research project.

Following review of all the anticipated bridge construction activity proposed for the research area, ten candidate bridges were selected for instrumentation and performance monitoring. Instrumentation plans were prepared and arrangements were made to add the proposed instrumentation work into portions of the formal contract documents for each candidate bridge.

Summary of Candidate Bridges

Candidate bridges were located in Connecticut, Massachusetts, New York, Rhode Island and Vermont at the locations shown in figure 1. A case history summary sheet for each of the bridges has been prepared and included in appendix A. Each summary sheet includes a photograph of the completed structure and information on the bridge location, structural design features, general subsurface information, construction data and a summary of the instruments installed at the bridge site.

As noted in the summary sheets, four of the instrumented bridges were single span structures. Two double-span and three 4-span bridges were also monitored in addition to a single 5-span structure. Nine of the structures were designed to carry highway traffic while one instrumented bridge consisted of a 4-span railroad bridge across an Interstate highway. The bridges included 5 simple-span and 5 continuous-beam structures.

Subsurface bearing soils beneath the spread footings at each of these bridges range from naturally deposited sand and gravel from approximately 20 to 90 feet thick to compacted granular fill ranging from approximately 4 to 28 feet in thickness. Compressible silt strata were present beneath the granular foundation bearing soils at two bridge sites. Foundation preparation at these sites required the placement of embankment preloads prior to the construction of these structures. Test boring location plans for each candidate bridge have been included as appendix B. Refer to chapter 3 for further discussion of test boring data.

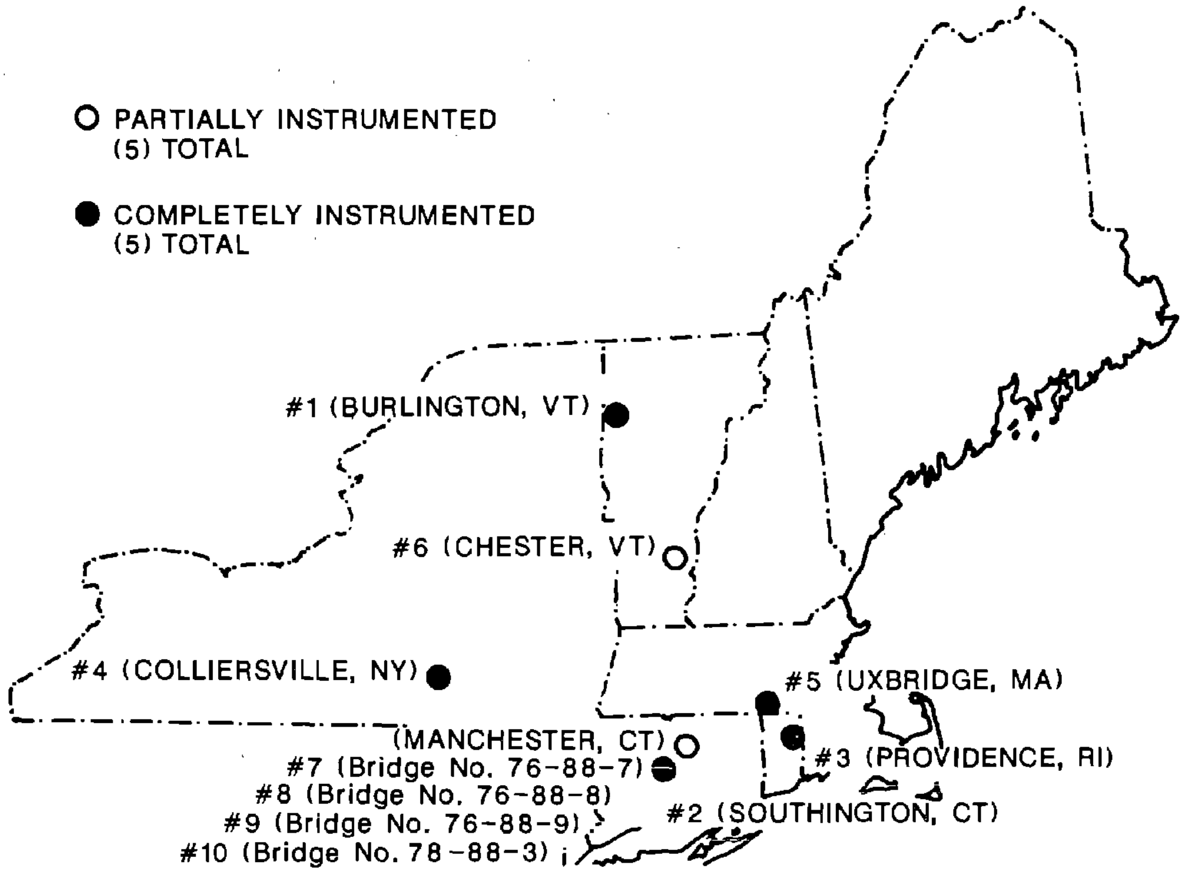


Figure 1. Instrumented bridge locations.

2.3 SELECTION OF PERFORMANCE PARAMETERS

General

The design of spread footing foundations on granular soils is generally governed by tolerable settlement criteria. Therefore, foundation settlement was determined to be the most important performance parameter to be monitored. In addition, tilt/overturning of the bridge structures was also monitored to provide a practical indication of structure movement and to allow comparison with contact stress. At five selected bridges, applied loads and corresponding foundation contact stresses were also monitored.

The following sections provide a generalized description of how the parameters were monitored throughout the course of the study. Detailed descriptions of the geotechnical instrumentation systems utilized are presented in separate sections.

Foundation Settlement

Settlement of the bridge structures was monitored at each bridge site using equipment shown in figure 2. In addition to bridge structure settlement, deep seated soil compression was monitored at two bridge sites, using equipment shown in figure 3, to determine the distribution of soil settlement with depth.

Traditional optical survey techniques utilizing fixed reference points comprised the primary method of monitoring movements of the bridge structures both during and after construction. Monitoring of the settlement of the heel of a buried abutment footing was accomplished using a "settlement profiler" which will be described in subsequent sections.

Foundation Contact Stress

At five selected bridges, actual load applied to the foundation bearing soils was monitored in the form of contact or bearing stress. These applied stresses are responsible for the development of compression of the granular subgrade soils and corresponding bridge foundation settlement. Therefore, it was determined that the total contact stress applied to the foundation bearing soils be measured at various locations beneath individual bridge footings as shown schematically in figure 4.

These measurements were made by installing an array of contact stress cells beneath an individual footing which provided measurements of contact stress at each cell location. In addition, review of these data with respect to the spacial location of each cell beneath the footing provided an indication of the distribution of contact stress beneath the footing.

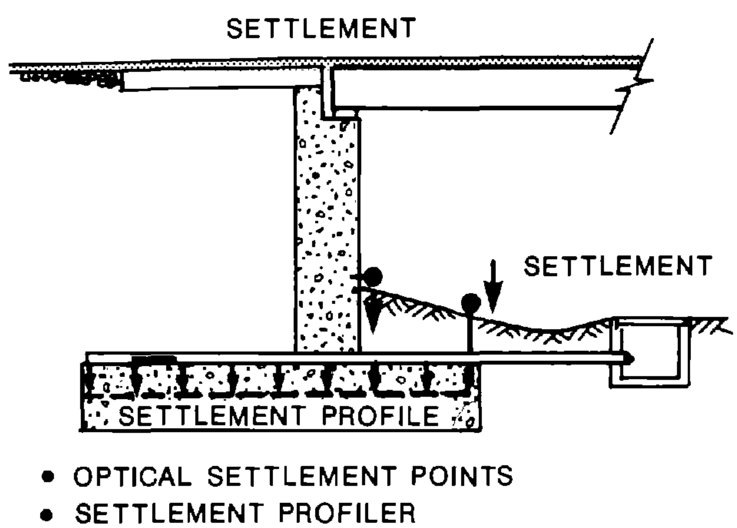


Figure 2. Settlement equipment for structures.

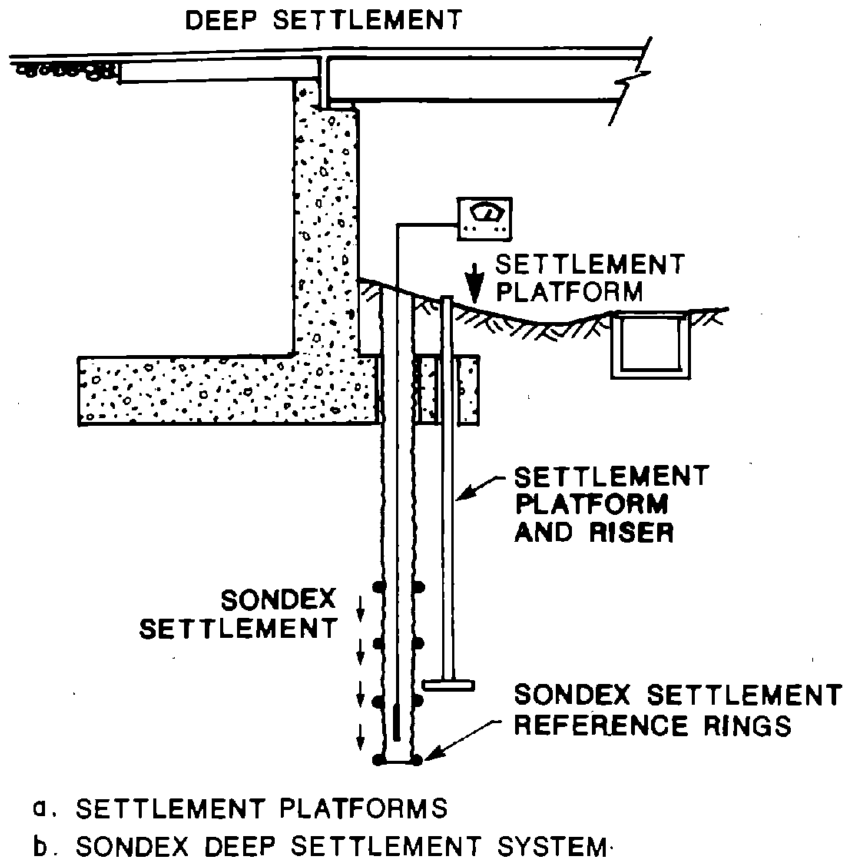


Figure 3. Settlement equipment in soil.

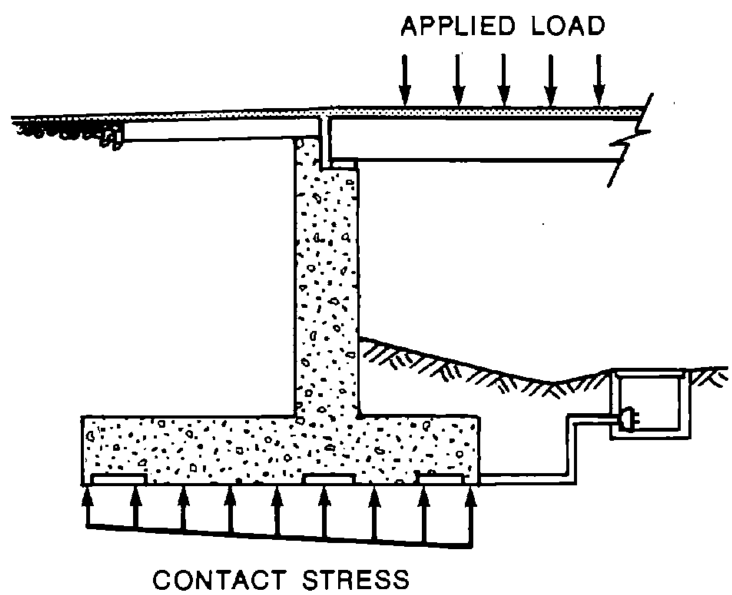


Figure 4 Contact stress and test loading.

Applied Loads

Actual load which is applied to the bridge foundation represents another important parameter. It is this load which the bridge designer should use to estimate settlement and establish the corresponding size of the spread footings. This parameter is difficult to monitor. Insertion of load cells beneath beam seats would register actual load of the deck system and live load. However, the measurement would only record 10 to 20 percent of the total load at footing level. Moreover, several departments expressed reluctance to interfere with the normal bridge seat geometry and design.

Consequently, a decision was made to compute applied loads based on the actual volumes and unit weights of the materials used to construct the bridge. These data were obtained from the individual bridge design plans and construction documentation such as the actual number of cubic yards of concrete placed or the number of tons of steel actually installed. In order to measure the impact of traffic loading, arrangements were made at several bridge sites to provide a controlled loading of the completed structures using loaded trucks of a known weight. These bridge load tests, illustrated in figure 4, consisted of placing loaded trucks at various locations on the completed bridge deck and monitoring the changes in the settlement and stress instrumentation. These data provided calibration of bridge and instrument behavior in response to a known live load.

Tilting

The tilting or overturning of the individual bridge structures was determined to be an important performance parameter to monitor since overturning is normally analyzed as part of an abutment wall design. The tilt of abutment stem walls and selected pier columns was monitored throughout construction utilizing fixed reference points and a portable tilt-sensing accelerometer shown in figure 5. In addition, profiles of settlement across the tops of abutment wall footings obtained through the use of the settlement profiler device also provided another measurement of overturning.

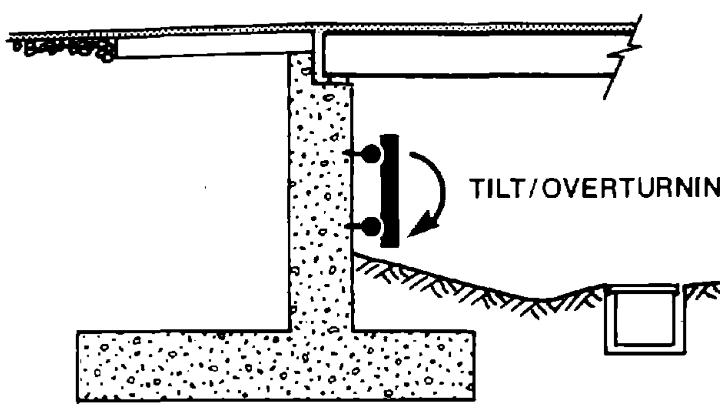


Figure 5. Tilt measurement equipment.

Data obtained from successive monitoring of overturning during construction provided an indication of the relative stiffness of the moment connection between the foundation footing and the abutment wall/pier primarily in response to placement of earth backfill and traffic loads. In addition, the footing settlement profiles obtained with the profiler device provided an indication of the overall footing behavior during construction and was reviewed in conjunction with the tilt/overturning data and distribution of contact stresses.

2.4 GENERALIZED DESCRIPTION OF INSTRUMENTATION SYSTEMS

General

The instrumentation systems developed for performance monitoring of the ten candidate bridges consisted of the implementation of a partial instrumentation system on five bridges, a full instrumentation system on the remaining five bridges and a data storage, processing and retrieval system. The goals of the instrumentation program were to develop a simple, low-cost and reliable system to monitor key performance parameters necessary to document satisfactory bridge performance. The system was to be capable of being used by qualified FHWA and State Department of Transportation personnel.

The following sections provide a description of the various aspects of the instrumentation systems developed. Figures 6 and 7 provide a conceptual representation of the partial and full instrumentation systems and appendix C contains instrumentation plans used for a fully instrumented bridge.

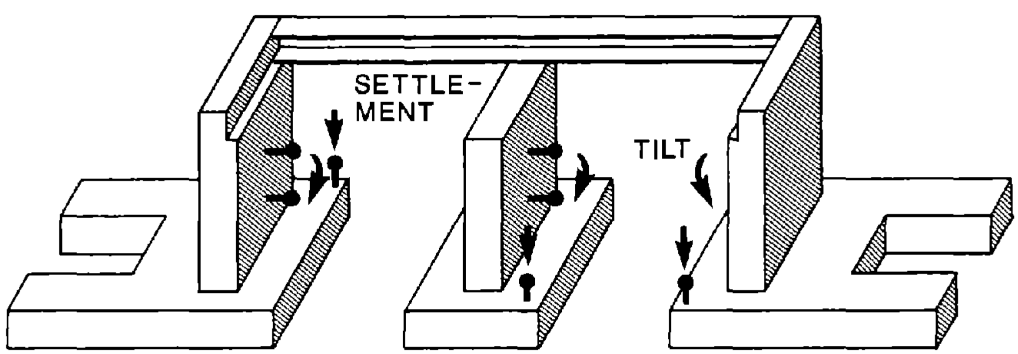


Figure 6. Conceptual plan: partially instrumented bridge.

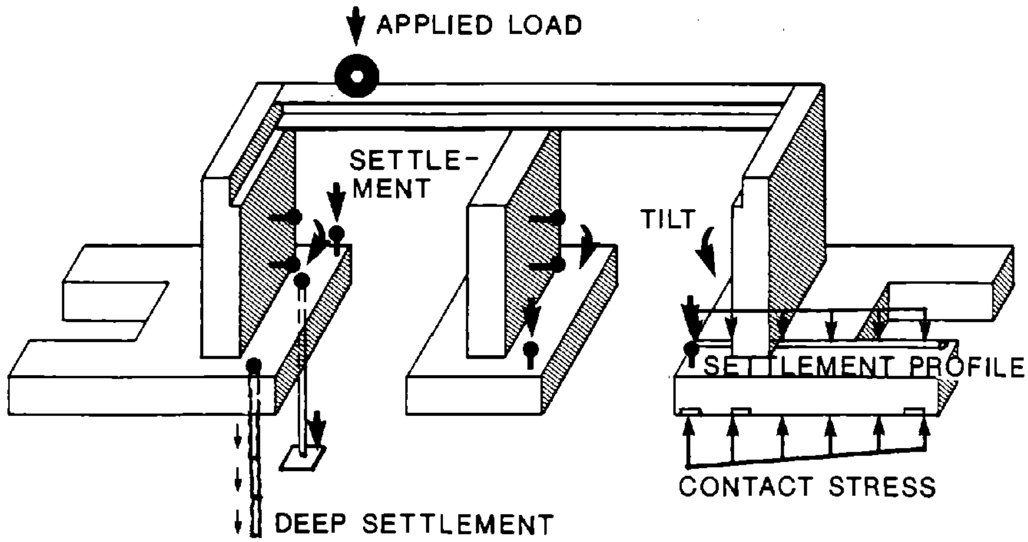


Figure 7. Conceptual plan: fully instrumented bridge.

Partially Instrumented Bridges

Five of the candidate bridges were selected to receive partial instrumentation systems which were developed to monitor settlement and overturning. The implementation of these instrumentation systems required the installation of fixed reference points on the individual bridge structures and periodic monitoring of these points during the construction process.

Reference points used for the project consisted of two types. For measurement of footing settlement, exposed ends of rebar cast into the footing or a length of 1-in steel pipe nipple threaded into pipe couplings cast into the footings were used as reference points. Abutment, wingwall and pier settlement was monitored with reference points developed for use with both the settlement and tilt monitoring equipment. These reference points consisted of stainless steel anchor rods cast directly into selected locations at individual bridge structures. The exposed ends of these rods presented a female thread which received a removable reference point consisting of a 1-in dia. stainless steel ball at the end of a male thread. By screwing the reference point balls into the anchor rods, a rounded, well defined reference location was established which could be monitored and subsequently removed to prevent damage due to construction activity or vandalism. The basic pieces are shown in figure 8.

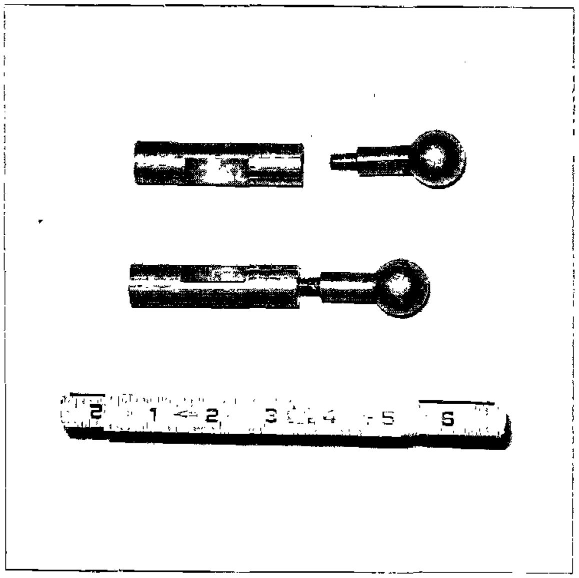


Figure 8. Settlement monitoring reference points.

Settlement monitoring of the partially instrumented bridges was accomplished by traditional optical survey methods. However, specialized survey equipment and procedures were utilized to provide measurement of settlement to within ± 0.005 ft (±1/16-in). Equipment included a Lietz Model B-1 automatic level equipped with a parallel plate micrometer and a Wild 10 ft invar rod. Specialized procedures included balanced fore and backsight distances, rounded, well defined turning points, maximum site distances of 75 ft, and a bubble level to plumb the rod for each reading.
Tilt/overturning was monitored with the use of a Slope Indicator Co. (SINCO) tiltmeter system consisting of a Model 50306 Digitilt indicator and a Model 50344 Tiltmeter sensor. A SINCO Model 50373 brass tilt plate was mounted to a specially fabricated portable tilt measurement bar provided by Geokon, Inc. as shown in figure 9. A "Vee" notch along the length of the rear side of this portable measurement bar was designed to be placed on a pair of stainless steel reference points installed about 3 ft apart along an imaginary vertical line on the bridge structure. The tiltmeter sensor was then placed on the brass tilt plate and the corresponding signal monitored with the portable indicator. Data was recorded manually and the corresponding angle of the tilt plate from vertical was subsequently computed. Figures 10 and 11 show the use of the tiltmeter monitoring system.

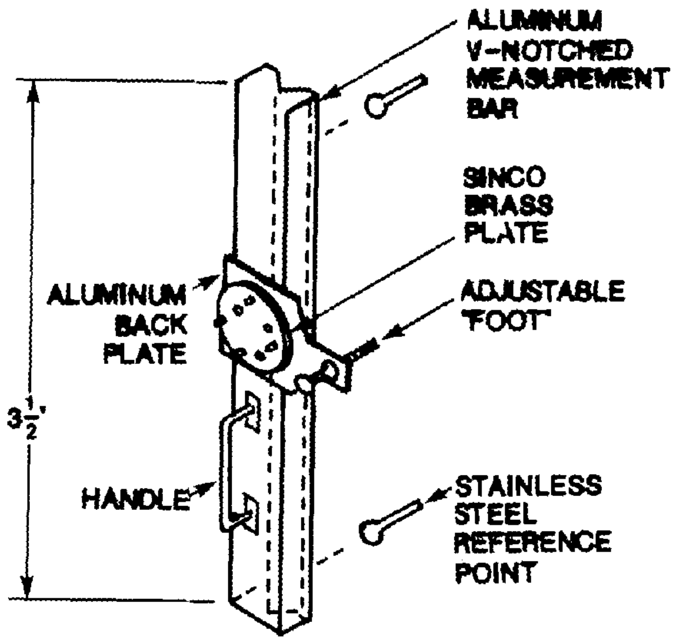


Figure 9. Portable tilt measurement bar.

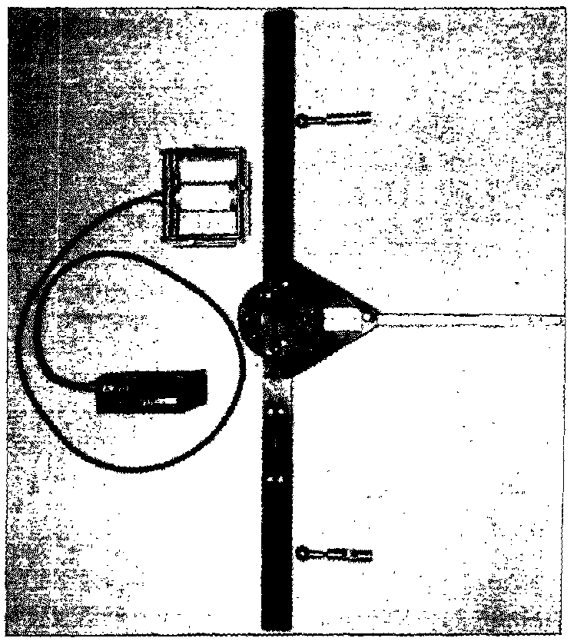


Figure 10. Tiltmeter measurement system.

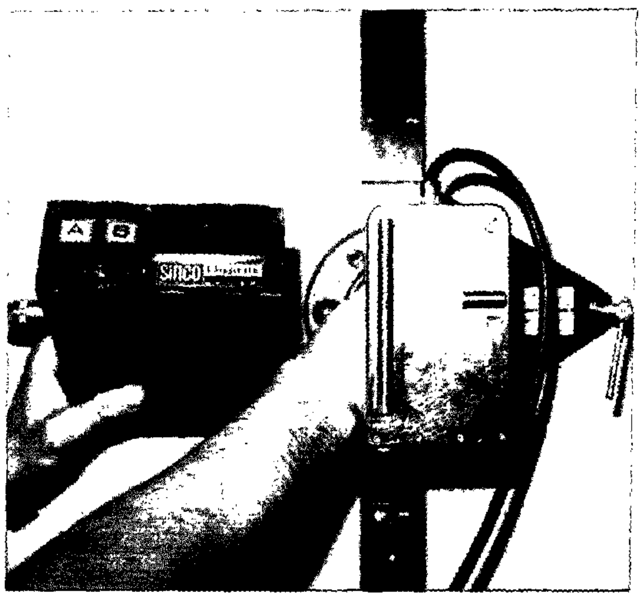


Figure 11. Tiltmeter measurement system in use.

Fully Instrumented Bridges

Five bridges were selected to receive full instrumentation systems which were developed to provide the overall foundation performance data. In addition to tilt and settlement, the full instrumentation system provided for the measurement of additional parameters which included settlement profile, deep seated settlement, foundation contact stress and applied loading. As in the partial instrumentation system, the fully instrumented bridges were monitored at key stages in the construction process to document the ongoing effects of construction on the foundation performance. The following paragraphs describe the procedures used to measure these performance parameters. In addition, appendix C contains typical contract documents used to describe a fully instrumented bridge system implemented during the research.

The profile of settlement across backfilled abutment footings was monitored using a remote settlement profiling device designed and assembled in cooperation with Geokon, Inc. This device, which was monitored from an instrumentation manhole constructed at the toe of the footing, permitted remote monitoring of settlement of the buried portion of the footing. Data was obtained throughout construction and after opening of this bridge to traffic without interfering with these operations.

The settlement profiler consisted of a sensor which was traversed through a PVC conduit fixed to the top of the abutment footing. The sensor was connected via a mercury filled nylon tubing to a fixed mercury reservoir placed at a known elevation. The mercury head differential between the reservoir and the sensor was measured at each of the traverse points along the PVC conduit thereby providing a measurement of the top of the footing along the line of the PVC conduit. Mercury was chosen as the fluid because of its high density which allowed determination of settlement to within $\pm 1/8$ in.

Deep seated settlement was measured at two bridge sites where compressible silt strata were found to underlie the granular foundation bearing soils. This settlement, associated with the consolidation of these compressible soils, was subtracted from the settlement measured directly on the individual bridge structures to determine the net settlement attributed to compression of the granular foundation bearing soils.

Traditional settlement platforms shown in figure 3 comprised the first deep settlement monitoring system. A settlement platform consisted of a steel riser pipe attached to a plywood board which was placed on a subgrade surface. The tops of the steel riser pipes were monitored with optical surveys to measure settlement of the plywood platform as backfill was placed.

A second deep settlement measurement method utilized the Slope Indicator Company SONDEX system illustrated in figure 3. This system uses a compressible corrugated polyethylene pipe installed in a drilled borehole. Reference points consisting of wire rings were wrapped around the outside of the pipe at 5-ft increments. The depth to these reference points was measured with the SONDEX sensing probe. The corresponding elevation was computed as construction proceeded. These data provided a profile of settlement vs depth below the instrumented bridge footings.

Specially fabricated contact stress cells recorded the contact bearing stress between the base of the bridge footings and the bearing soil. These stress cells were distributed across the base of the abutment footings as shown on the instrumentation plans included in appendix C. These instruments were placed with the pressure sensitive face directly on the subgrade bearing soils while the insensitive back cell face, comprised of a 1/2 in thick steel plate, was cast directly into the footing. Readout cables for these stress cells were tied to the footing rebar cage prior to concrete placement and were terminated at the location of instrumentation manholes which were subsequently constructed at the toe of the abutment footing. These cells provided a measure of both the magnitude and distribution of contact stress beneath the footing.

Test loads were applied to the fully instrumented bridge structures. Tests were completed by placing loaded trucks of known weight at various locations on the completed bridge structure prior to opening the structure to traffic. With these loads in place, all the instruments at the bridge were monitored in order to document their response to the placement of the known applied loads. In general, the test loads resulted in recoverable stress changes and settlement, representing only a small percentage of total applied load or total observed settlement.

Data Storage and Retrieval System

A data processing system was developed for use in processing, tabulating, plotting and storing instrumentation data obtained for the ten instrumented bridges. Modified DBASE II software was developed to handle data from the seven instrument types used during the research which included Optical Settlement Survey, Tiltmeter, Contact Stress Cell, Settlement Platform, Load Cell, SONDEX and Settlement Profiler. The software is menu-driven and is designed for use with an IBM-PC XT microcomputer equipped with a line printer and an HP Model 7475A plotter. Data may be reviewed visually on the PC monitor screen or printed in tabular form on the line printer. In addition, the plotter may be used to prepare a graph of individually measured parameters vs elapsed time.

All of the data obtained during the study was processed with this newly developed software and was stored on floppy diskettes. In addition, a user's manual for the software package was prepared. The manual, program, and copies of the project data are available upon request.

2.5 SUMMARY OF INSTRUMENTATION DATA

General

The following sections provide a general summary of the instrumentation data obtained during the study. A summary of the numerical values of the parameters measured at each bridge is provided in table 1. All the instrumentation data obtained during the project has been stored on floppy diskettes. The comparison between actual and predicted settlement performance is summarized in detail as part of the work completed under a separate task and is presented in chapter 3 of this report.

Settlement as a Function of Applied Loads

The construction of the individual instrumented bridges gradually applied load and corresponding foundation stresses to the granular foundation bearing soils. In direct response to key construction phases such as abutment wall construction, backfill placement, girder placement, bridge deck construction, etc., corresponding settlement of the bridge structures were monitored. Settlement developed in response to footing construction was not measured since the settlement reference points were initially cast into the footing concrete.

As described in the following section, about 70 percent of the total settlement occurred prior to the placement of the bridge deck structure. The settlement developed during the period when the majority of the design loads were placed on the spread footing foundations. During the test truck loading and subsequent traffic loading, little or no incremental settlement was observed. These loads represented only a small percentage increase in the total structural load already in place on the footings.

Settlement as a Function of Time

As indicated in table 1 and the bar graph included as figure 12, total settlement monitored at each of the instrumented bridges ranged from 0.02 to 2.72 in with an average total settlement of 0.61 in (less than 3/4 in). Note that consolidation settlement of underlying compressible silt at bridge nos. 1 and 4 (Burlington, VT and Colliersville, NY) ranged from 0.66 to 0.99 in which was subtracted from the total observed settlement to provide net settlement values attributed to elastic compression of the granular foundation bearing soils.

When settlement is plotted against elapsed time however, it is apparent that a large portion of the observed total settlement developed prior to the placement of the bridge deck structure. This typical behavior is displayed in figure 13. For the ten instrumented bridges, the total post deck settlement ranged from 0.02 to 0.85 in with an average post deck settlement

Table 1. Ranges of measured parameters.

BRIDGE NO.	BRIDGE LOCATION	STRUCTURAL ELEMENT	SETTLEMENT (IN.) TOTAL	CONSOLIDATION	NET	TILT (DEGREES) FACE[1]	WINGWALLS	CONTACT STRESS (KSF)	REMARKS
001	Burlington, VT.	Abutment No. 1	1.00 to 1.33	0.66 to 0.97	0.34 to 0.36	0.020 to 0.022	-0.017 to 0.018	2.39 to 18.31	Sondex Settlement 0.79 in.
		Abutment No. 2	0.58 to 0.76	NM	NM	0.058 to 0.078	0.002 to 0.020	NM	Consolidation Settlement not monitored
002	Cheshire, CT.	Abutment No. 1	0.78 to 1.15	NA	NA	0.011 to 0.018	NM	4.06 to 8.98	Bridge not complete as of 6/86
		Pier	0.58 to 0.65	NA	NA	-0.030 to 0.017	NA	NM	Deck finished
		Abutment No. 2	0.73 to 0.80	NA	NA	-0.108 to 0.057	NM	NM	
003	E.Providence R.I.	West Abutment	0.37 to 0.46	NA	NA	-0.021 to -0.044	NM	NM	
		Pier 1	0.02 to 0.24	NA	NA	-0.032 to -0.044	NA	NM	
		Pier 2	0.24 to 0.29	NA	NA	-0.020 to -0.031	NA	NM	
		Pier 3	0.97 to 0.98	NA	NA	-0.065 to 0.017	NA	NM	Disturbed subgrade soils
		East Abutment	0.46 to 0.64	NA	NA	0.001 to 0.015	NM	-0.30 to 4.34	
004	Colliersville, NY	South Abutment	1.06 to 1.21	0.72 to 0.74	0.34 to 0.47	-0.014 to -0.031	0.031 to -0.011	0.17 to 1.89	
		Pier	NM	NM	NM	NM	NM	NM	Pile foundations
		North Abutment	0.30 to 0.38	NM	NM	-0.038 to -0.040	-0.005 to -0.010	NM	Consolidation settlement not monitored
005	Uxbridge, MA	North Abutment	0.16 to 0.28	NA	NA	-0.024 to -0.117	-0.008 to -0.105	NM	
		South Abutment	0.10 to 0.86	NA	NA	-0.040 to 0.236	0.011 to 0.236	0.28 to 2.03	
006	Chester, VT	East Abutment No. 1	2.18 to 2.72	NA	NA	-0.037 to -0.063	NM	NM	Disturbed subgrade soils
		West Abutment No. 2	0.76 to 0.95	NA	NA	0.052 to 0.080	NM	NM	
007	Manchester, CT	Abutment No. 1	NM	NA	NA	NM	NM	NM	
		Pier 1	0.60 to 1.07	NA	NA	NM	NA	NM	

NA Not Applicable
NM Not Measured

1. (+) Tilt denotes movement of the top of the abutment/wingwall/pier away from the side on which the reference points are installed.

Table 1. Ranges of measured parameters (continued).

BRIDGE NO.	BRIDGE LOCATION	STRUCTURAL ELEMENT	SETTLEMENT (IN.) TOTAL	CONSOLI-DATION	NET	TILT (DEGREES) FACE[1]	WINGWALLS	CONTACT STRESS (KSF)	REMARKS
007	Manchester, CT (cont.)	Pier 2	NM	NA	NA	NM	NA	NM	
		Pier 3	NM	NA	NA	NM	NA	NM	
		Abutment No. 2	0.20 to 0.64	NA	NA	-0.058 to -0.077	NM	NM	
008	Manchester, CT	Abutment No. 1	0.42 to 0.59	NA	NA	-0.021 to 0.001	NM	NM	
		Pier 1	0.32 to 0.36	NA	NA	NM	NA	NM	
		Pier 2	NM	NA	NA	NM	NA	NM	
		Pier 3	NM	NA	NA	NM	NA	NM	
		Abutment No. 2	0.61 to 1.04	NA	NA	-0.055 to -0.085	NM	NM	
009	Manchester, CT	Abutment No. 1	0.49 to 0.78	NA	NA	-0.063 to 0.090	-0.038 to -0.081	NM	
		Abutment No. 2	0.19 to 0.37	NA	NA	-0.114 to 0.137	0.054 to -0.088	NM	
010	Manchester, CT	Abutment No. 1	-0.01 to -0.04	NA	NA	NM	NM	NM	Post-construction settlement only
		Pier 1	-0.04 to -0.05	NA	NA	NM	NM	NM	
		Pier 2	-0.04 to -0.05	NA	NA	NM	NM	NM	Post-construction settlement only
		Pier 3	-0.02 to -0.08	NA	NA	NM	NM	NM	Post-construction settlement only
		Pier 4	0.04 to 0.08	NA	NA	NM	NM	NM	Post-construction settlement only
		Abutment No. 2	0.34 to 0.60	NA	NA	NM	NM	NM	Post-construction settlement only

0100W

NA Not Applicable
NM Not Measured

1. (+) Tilt denotes movement of the top of the abutment/wingwall/pier away from the side on which the reference points are installed.

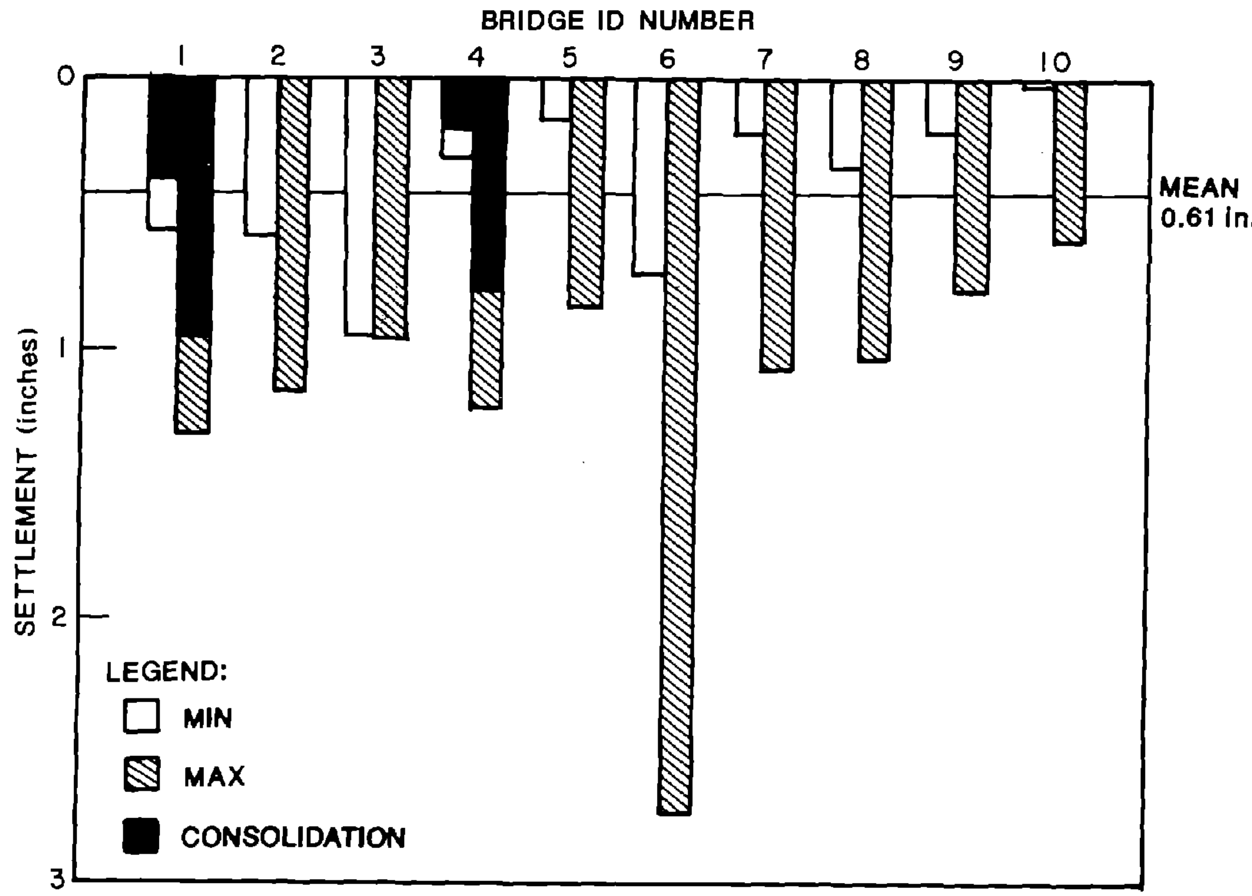


Figure 12. Measured settlement.

of 0.21 in (less than 1/4 in) as shown in figure 14. In addition, it is important to note that little additional settlement of the instrumented bridges was observed for the one to two year period after the opening of these structures to traffic.

<u>Tilting</u>

Data obtained from the tiltmeter measurement system indicate that the abutment walls of the instrumented bridges tilted (overturned) from 0.23 degrees towards the approach backfill to 0.12 degrees away from the approach backfill. The mean value of observed abutment wall tilt was 0.023 degrees towards the approach backfill indicating that overall, the instrumented abutment walls were subject to little or no overturning. These data are displayed in the bar graph included as figure 15 and summarized in table 1.

By comparison, for the active earth pressure case to develop, a wall deflection equal to 0.005 times the height is often stated as a guideline (Lambe & Whitman, 1969). This guideline is equivalent to a tilt of 0.3 degrees.

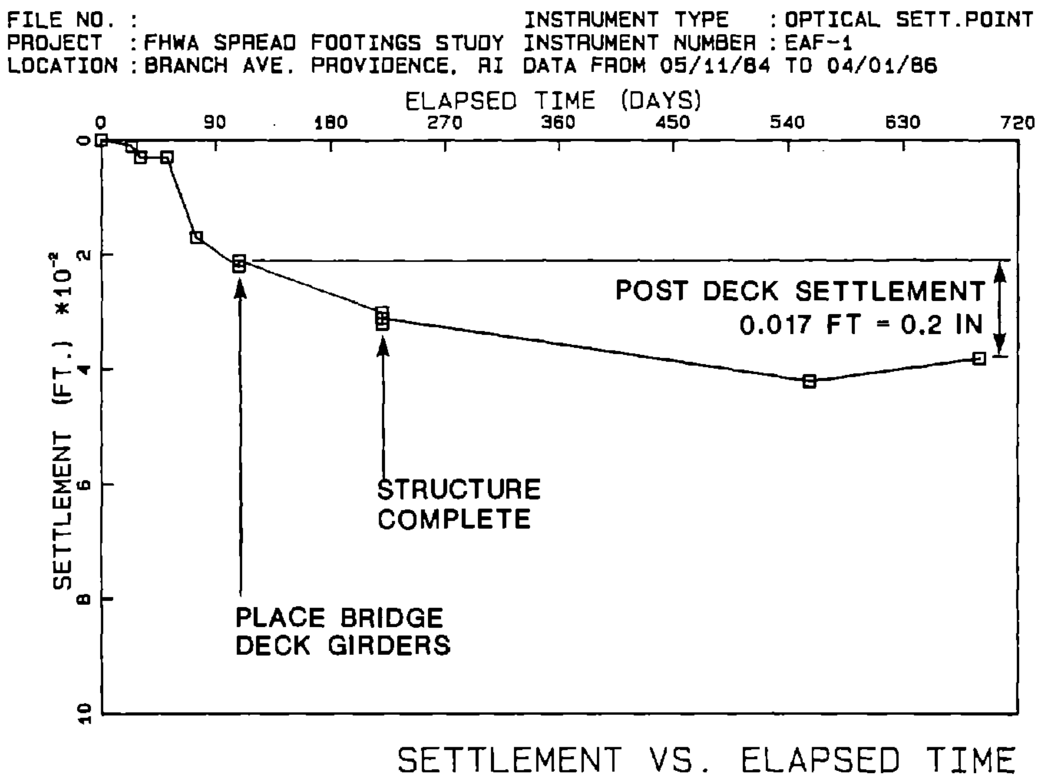


Figure 13. Typical settlement vs time.

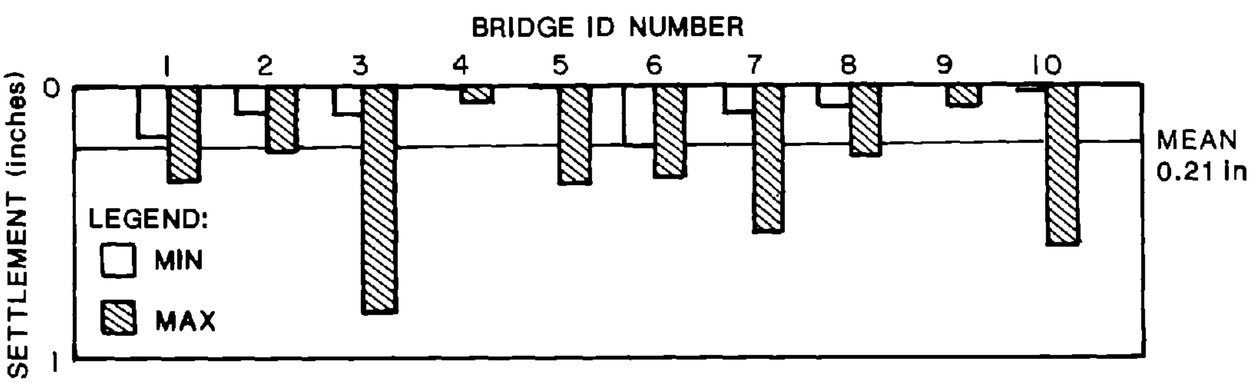
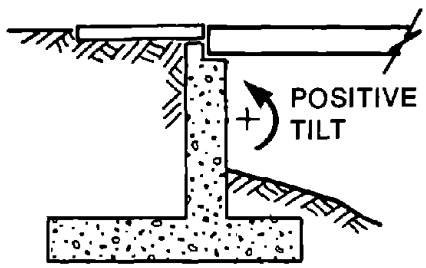


Figure 14. Post deck settlement.



Sign Convention:

Positive tilt is defined as a tilt of the top of the abutment wall toward the approach backfill.

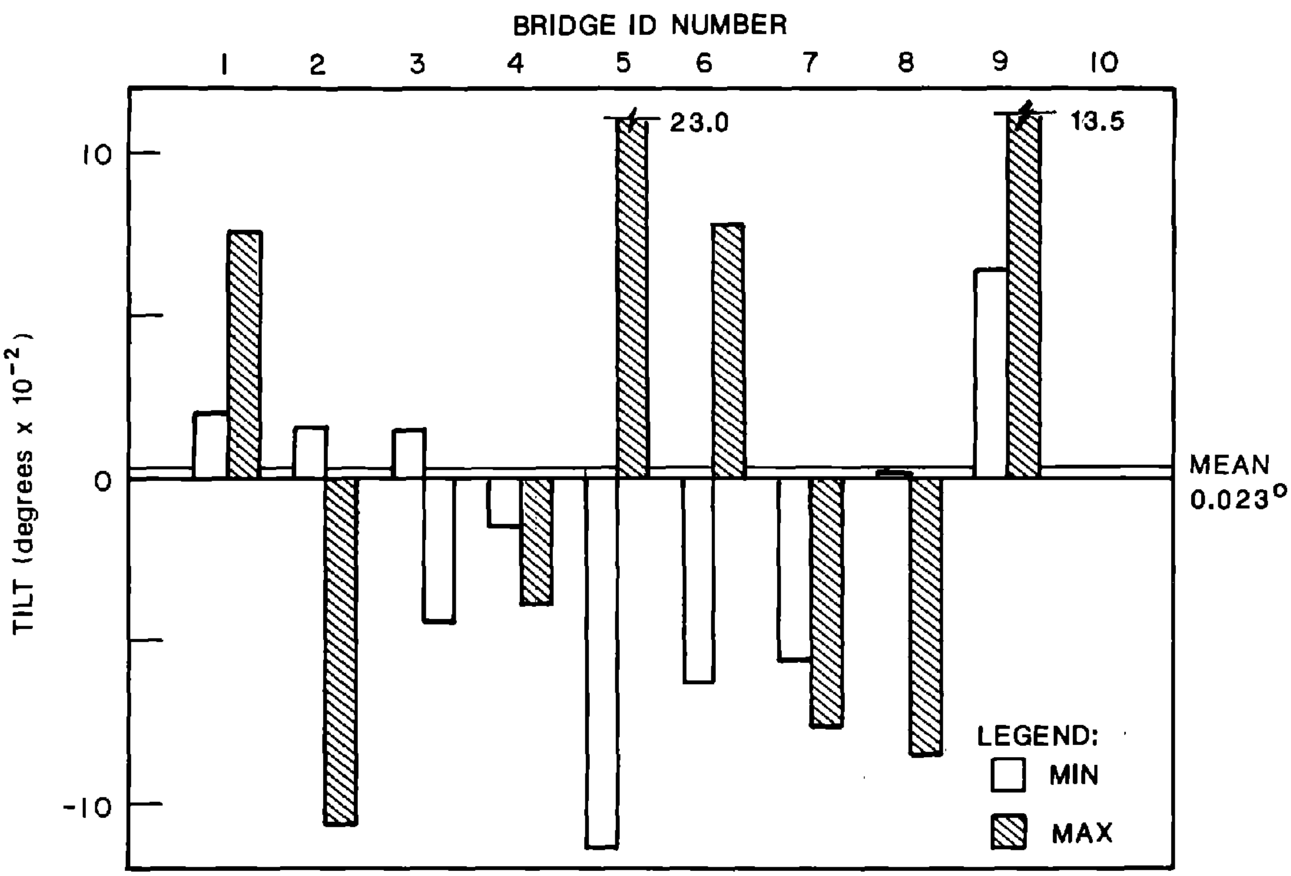
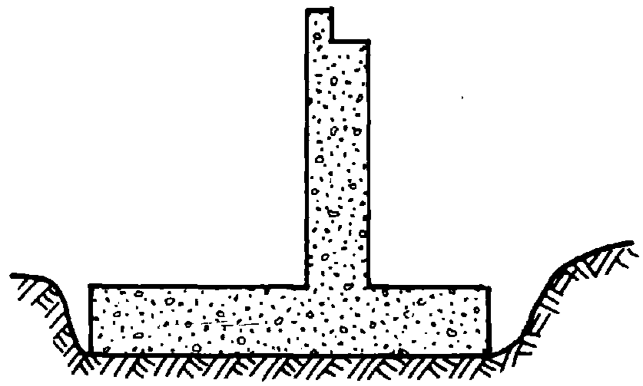


Figure 15. Tilt of abutment walls.

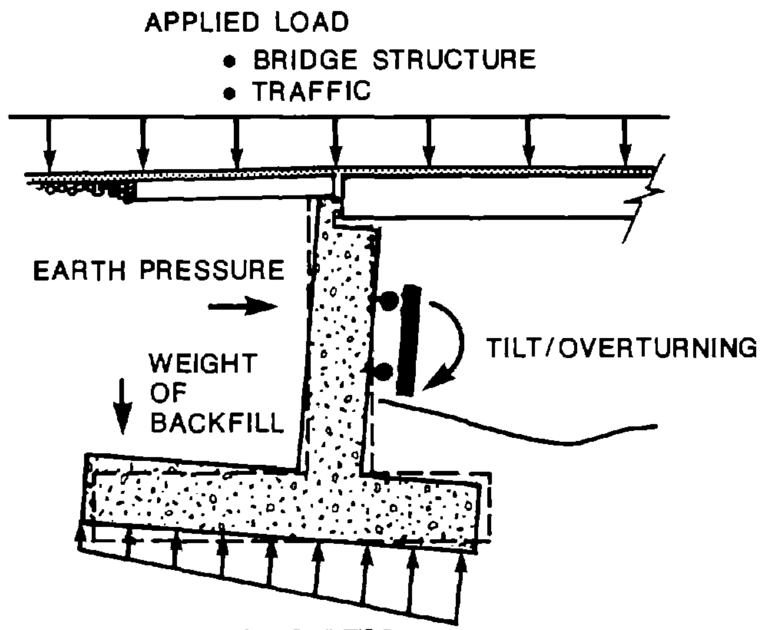
Based on the conflicting behavior of tilt developing both towards and away from the approach backfill, no general conclusions were drawn on the anticipated direction and magnitude of tilt at these structures other than to conclude that tilt/overturning was in effect very small, and would indicate the active pressure condition was not mobilized.

Foundation Contact Stress Distribution

To evaluate the distribution of contact stress beneath the abutment wall foundations, a total of eight to ten contact stress cells were placed at various locations across the base of the abutment wall footings at the fully instrumented bridges. Data from these instruments were to be used in conjunction with the tilt/overturning data to evaluate the general abutment wall/footing behavior conceptualized as shown in figure 16.



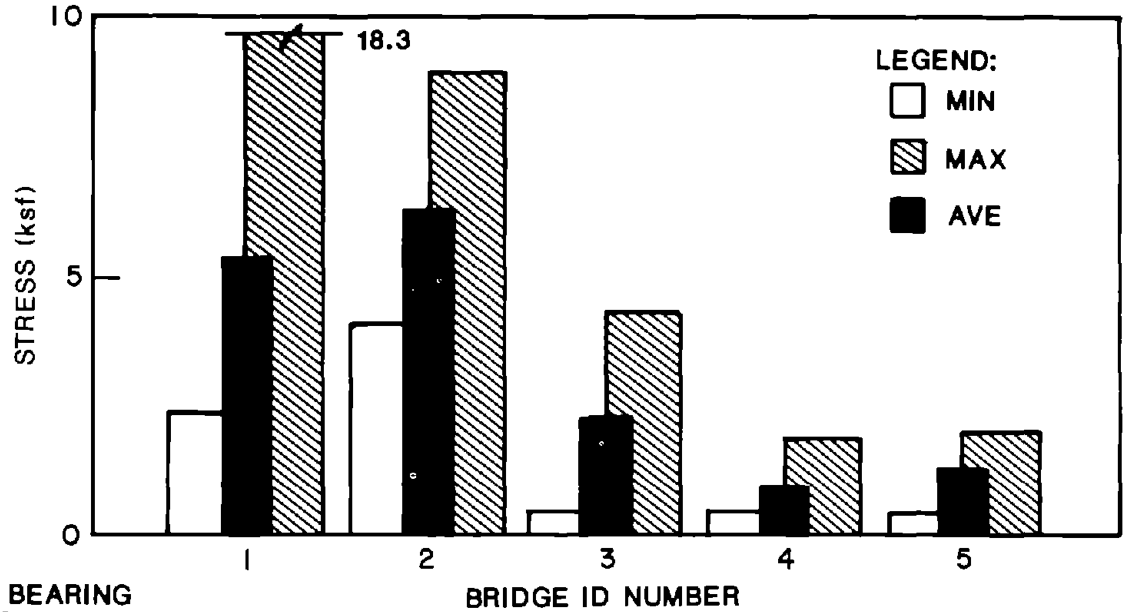
ABUTMENT FOOTING INITIAL CONDITION



ABUTMENT FOOTING AFTER CONSTRUCTION

Figure 16. Contact stress and tilt - theoretical model.

Data from the stress cells presented in figure 17 was generally found to be somewhat erratic and different from the anticipated values based on computation of the applied foundation bearing stresses. However, average contact stress values for three of the five fully instrumented bridges agreed reasonably well with the foundation bearing pressures.



DESIGN BEARING PRESSURE SOURCE					
H&A COMPUTATIONS(1)	3.65	2.64	2.85	2.82	2.85 ksf
BRIDGE PLANS	4.0	5.0	4.0	5.0	5.0 MAX TOE PRESSURE, ksf

NOTE: (1) INCLUDES WEIGHT OF CONCRETE IN FOOTING

Figure 17. Stress cell data - graphical summary.

Measured stresses should be compared only to carefully computed design stresses including the weight of the concrete footing. Design stresses noted on contract drawings often reflect maximum allowable values, not the actual design value.

Individual cell readings often showed marked variations from the average values. Cells at the toe of the footing could register lower stresses than at the heel or centerline. At the Uxbridge, MA bridge site, for example, stress cell data shown in figure 18 for the south abutment illustrate the random pattern of readings.

The reasons for the erratic stress cell behavior are believed to be the result of several factors which include: local variations in the subgrade foundation bearing soils, temperature effects during curing of the concrete footings and an insufficient number of these expensive stress cells to provide statistically meaningful results.

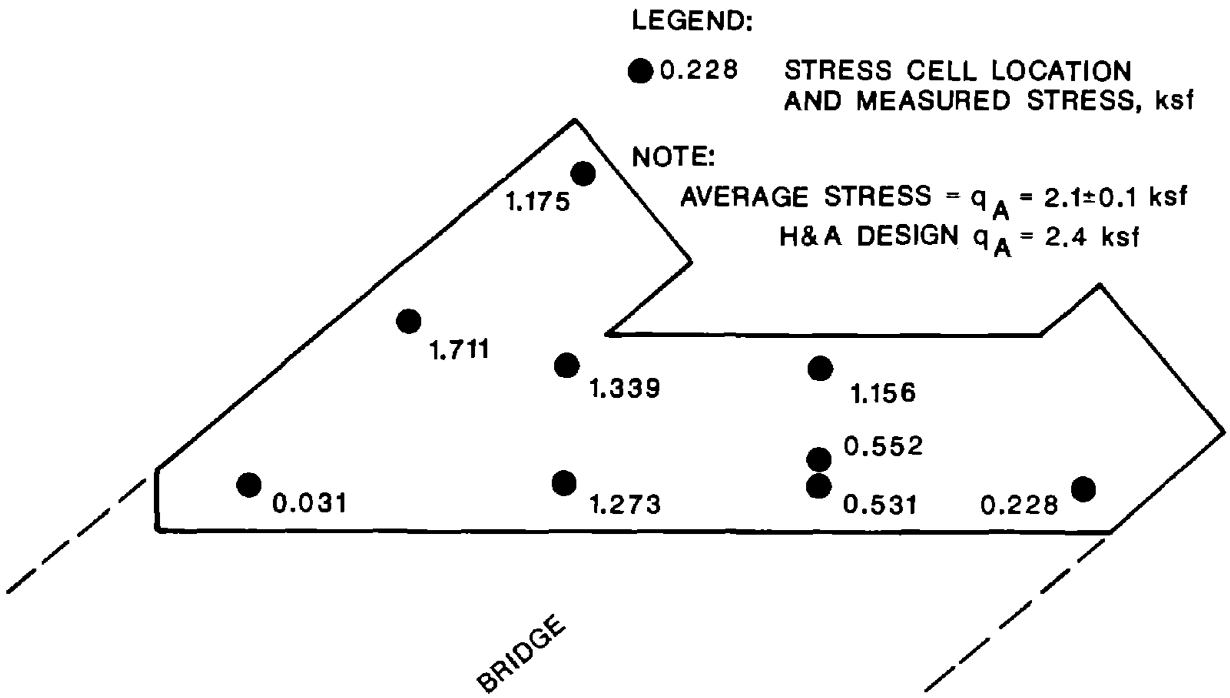


Figure 18. Stress cell data for Uxbridge, MA.

2.6 EVALUATION OF INSTRUMENTATION PROGRAM

General

The following sections provide a summary of the individual instrumentation equipment used during the project and comments on its use and performance. In addition, evaluation comments are provided for both the partial and total bridge instrumentation systems.

Partially Instrumented Bridge System

The use of the partial instrumentation system, designed to monitor settlement and tilt was determined to be most effective in terms of cost, complexity and results. The parameters monitored by this system are judged to be the most important in the evaluation of satisfactory bridge performance and are easy to comprehend and monitor. The partial system was especially easy to implement on bridges which had progressed into the construction phase. The instrumentation work involved did not require the preparation of lengthy contract documents and did not impact the contractor's construction sequence, thereby adding to the cost of the project.

Fully Instrumented Bridge System

The fully instrumented bridge system, designed to monitor the parameters of settlement, deep settlement, settlement profile, tilt, contact stress and applied loading provides the opportunity to monitor a wider range of important performance parameters and offers flexibility in the means of measuring these parameters.

In general, all of the instrumentation equipment utilized in the fully instrumented bridges performed satisfactorily with the exception of the erratic data obtained from the contact stress cells as previously noted. However, there are several drawbacks to the use of the full instrumentation system which include:

- The system requires the use of sophisticated equipment and readout techniques. Consequently, personnel with background and experience in instrumentation are required to install and monitor the instruments.
- Close coordination with bridge design and construction are required as many items must be effectively incorporated into the bridge contract documents and executed in strict accordance with the contractor's construction schedule.
- The full instrumentation system is relatively expensive to implement. Premium costs include sophisticated instrument sensors and readout equipment, preparation of contract documents and the experienced personnel required to install, monitor, process and interpret the instrumentation data.

2.7 RECOMMENDATIONS

Instrumentation Systems

The use of the partial instrumentation system to monitor settlement and tilt was judged to be the most satisfactory and cost effective system for documenting bridge performance. The relatively small cost of the system combined with the overall ease of implementing the program and its overall satisfactory performance make it the most promising system for continued long-term use by FHWA and/or individual State DOT's.

Use of the full instrumentation system should be limited only to more advanced research oriented purposes. The relatively high cost of the system combined with the fact that the system requires personnel experienced in instrumentation programs makes the full instrumentation system less desirable for continued routine use.

Coordination of Effort

During the course of implementing both the partial and fully instrumented bridge systems, it is most important to assure that the instrumentation program is closely and carefully coordinated. This coordination begins in the design stage when the details of the instrumentation program are developed and integrated into the individual structure design.

During the construction phase, careful coordination and communication is required between the instrumentation staff, the contractor and the State DOT personnel involved with the project. This coordination is required to assure that instruments are properly installed and protected during construction and that proper advance notice is provided so that the instruments may be monitored at key construction phases.

In addition, carefully kept, well documented bridge construction records maintained daily, preferably by a project manager or clerk-of-the-works must be provided. These records should include such data as the extent of construction completed daily, structural members and their respective weights placed, volumes of concrete and backfill placed and the results of quality control tests completed, especially in-place field unit weight tests of structural backfill. These construction records are essential in order to evaluate the instrumented bridge performance data in conjunction with the ongoing construction operations. It is only through an overall coordinated effort that a satisfactory documentation of bridge performance may be successfully completed.

Long-Term Evaluation

The data obtained as part of this research document the satisfactory performance of ten highway bridges supported by spread footing foundations bearing in sand. It is recommended that the use of the partial instrumentation system be continued on a wide scale basis to provide additional, well documented, statistically meaningful performance data on more bridges founded on spread footing foundations.

In addition, it is recommended that long-term data on bridge performance, in particular settlement behavior, be maintained for several years after completion of the bridge structure. This will provide documentation of the long term, satisfactory performance of these instrumented bridges and their spread footing foundations.

3. PRESSURE DISTRIBUTION AND SETTLEMENT OF BRIDGE FOOTINGS ON SAND

3.1 INTRODUCTION

General

Design of bridge foundations consisting of footings bearing on sand is primarily a problem of predicting settlements of the footings under the anticipated loads. Methods for predicting settlement of individual footings and differential settlements between footings on sand must be sufficiently accurate so that bridge foundations can be designed with confidence that the bridge will perform acceptably over its lifetime. Designing foundations for settlement control contrasts with bridge foundations consisting of end-bearing piles or caissons or other deep foundations where the bearing capacity of the supporting geologic material is the primary concern. Settlement is often considered to be a secondary design issue for such deep foundation units because the magnitude of settlement is usually tolerable if the foundations have adequate safety against load capacity failure.

Design of bridge footings bearing on sand typically involves determination of a bearing pressure (referred to as the allowable bearing pressure), and corresponding lateral dimensions of the footings, which in turn allows prediction of settlement. In practice, lateral dimensions of bridge footings are often governed by the geometry of the supported member (abutment, pier, wingwall, etc.) and by requirements for overturning resistance, as well as settlement.

Design of bridge footings also involves structural detailing of the foundation units which are usually constructed of reinforced concrete. Requirements for footing thickness and reinforcement are, theoretically, functions of the actual distribution of soil contact pressure acting on the underside of the footings. Some knowledge of the distribution of the contact pressure is therefore also necessary to develop an efficient and safe structural design for the footings.

This research was directed primarily toward evaluating available methods and procedures for predicting settlement of footings bearing on sand. Conclusions have been developed on the applicability of existing methods to the design of bridge footings. Limited research was also conducted on methods for calculating the ultimate bearing capacity and on soil contact pressure distribution.

Objectives and Scope

Specifically, this research included the following work scope items:

- Review of current state-of-practice for design of footings on sand with regard to settlement, bearing capacity and contact pressure.

- Research and review in detail the various methods available in the published literature for calculating the settlement of footings on sand.
- Select one or more methods that appear to be the most promising for calculation of footing settlement.
- Review the available published literature for case studies of settlement of footings on sand for full-size structures.
- Calculate the predicted settlement for the footings described in the literature case studies using the selected settlement methods and compare to reported footing settlement.
- Calculate the predicted settlement for the footings instrumented for this study using the selected settlement methods and compare to actual measured footing settlement.
- Discuss the validity of the selected settlement prediction methods and if possible make recommendations for obtaining improved predictions.

3.2 FOOTING DESIGN CONSIDERATIONS

Requirements for a Satisfactory Footing Design

The basic requirements for a satisfactory bridge footing design are:

1. Settlement - The short-term and long-term settlement of the footings must be sufficiently small in magnitude so as not to impose excessive stresses on the structure nor impede the proper function of the bridge.
2. Bearing Capacity - The footings must be safe against failure (rapid and large settlement) from normal operating loads and occasional severe loads such as earthquakes, high winds, impacts, etc.

A satisfactory footing foundation design requires more than structural analyses and detailing of the footing. Knowledge of the site geology and subsurface soil and groundwater conditions is also necessary. In many cases, design of footing foundations requires more data on subsurface conditions and on soil properties than may be necessary for design of pile foundations. Similarly, more geotechnical engineering may be needed to properly assess footing feasibility and performance, particularly if the sand bearing soils are underlain by compressible soil such as clay.

A typical footing design procedure involves the following steps:

- Determining the geometry, magnitude and direction of the loads to be supported by the footings.
- Evaluation of the site history, geology and anticipated subsurface conditions, including whether the site has been preloaded geologically or by previous structures or embankments.

- Planning and conducting a subsurface exploration and soil testing program to provide sufficient information on the subsurface soil and groundwater conditions, and on soil compressibility and bearing capacity.
- Develop idealized subsurface profile(s) and compressibility parameters for use in design calculations.
- Select trial footing sizes based on bearing capacity considerations, experience or the bridge geometry.
- Calculate settlements of the trial footings. Modify footing dimensions to achieve tolerable calculated settlements.
- Confirm that there is an adequate factor of safety against bearing capacity for the final footing geometry.

As is the case with most geotechnical problems, design of footings involves more than using the appropriate design equations. Judgement is needed to evaluate soil type, compressibility and preloading, to assess the implications of all the relevant factors, and ultimately to decide if footings are technically and economically feasible.

For this study, settlement and bearing capacity analyses have been limited to consideration of conventional static loading situations, typical of most routine bridge projects. On some actual projects, potential effects of dynamic loading due to earthquakes or other causes may govern the foundation design. In such cases, the use of deep foundations such as piling or use of soil improvement methods may be required to provide adequate foundation support. Considerations of these special foundation conditions have not been included in this study.

Applicable Soil Conditions

This study focused on footings bearing on sand. For purposes of calculating settlement using the selected methods, "sand" is considered to include cohesionless, inorganic soils such as sand, gravel and non-plastic silt. Non-plastic is interpreted to correspond to a plasticity index (PI) essentially equal to zero. Applicable soil deposits defined by two common soil classification systems are given as follows:

Classification System	Included Soil Types
AASHTO	A-1, A-3, and non-plastic (PI nearly equal to 0) soils in A-2 and A-4 groups. (AASHTO Std. Specs, 1978)
UNIFIED	GW, SW, GP, SP, GM, SM and ML (PI nearly equal to 0). (Casagrande, 1948)

In addition to visual classification and laboratory grain-size testing, the Atterberg Limits Test (ASTM Test Designation D423 and D424) should be used to confirm that the plasticity index is essentially zero, and therefore that the soil is cohesionless.

Although the settlement calculation methods were each originally developed for footings on cohesionless sand, in practice they are used with soils having a very low, non-zero plasticity index, such as a PI up to 4. However the applicability of the methods decreases with increased plasticity and PI.

It is important to note that the available settlement and bearing capacity calculation methods for cohesionless soils are likely to be less reliable for geologic materials containing significant percentages of silt or gravel. Interpretation of in situ testing results and calculations of settlement and bearing capacity for such materials should be made by experienced engineers using judgement and caution.

Procedures for evaluating bearing capacity and settlement for soil types other than those listed above may differ from procedures used for sand. The feasibility of using footings and using methods for predicting settlement of footings bearing above such other materials should be evaluated by experienced geotechnical engineers.

In Situ Testing

Calculations of settlement of footings on sand requires an estimate of the soil compressibility. In the United States, sand compressibility is usually estimated by performing in situ penetration tests. By far the most common in situ test used in the United States to estimate compressibility of cohesionless soils is the Standard Penetration Test, SPT (ASTM D1586). The Standard Penetration Resistance N-value is used in settlement calculations as an indicator of in situ relative density which in turn is correlated with compressibility.

By the nature of the SPT and the variability of soil deposits in situ, variation and scatter in SPT results are inevitable, even within a given site. Many sources of variability and scatter in results have been identified (Kovacs, et. al., 1977). To maximize repeatability and the usefulness of the SPT in settlement calculations, the test must be carefully performed in accordance with standard procedures.

Four issues have been identified as having particularly significant impact on SPT results and can therefore affect settlement calculations based on the test results:

> Turns of the Rope - For non-trip hammers such as illustrated in figure 19, the test results are very sensitive to the number of turns (wraps) of the rope around the cathead used to lift the 140-lb weight. Two wraps are standard. Three or more wraps can prevent "free-fall" of the weight and significantly reduce the energy applied to the drilling rods and split-spoon sampler.

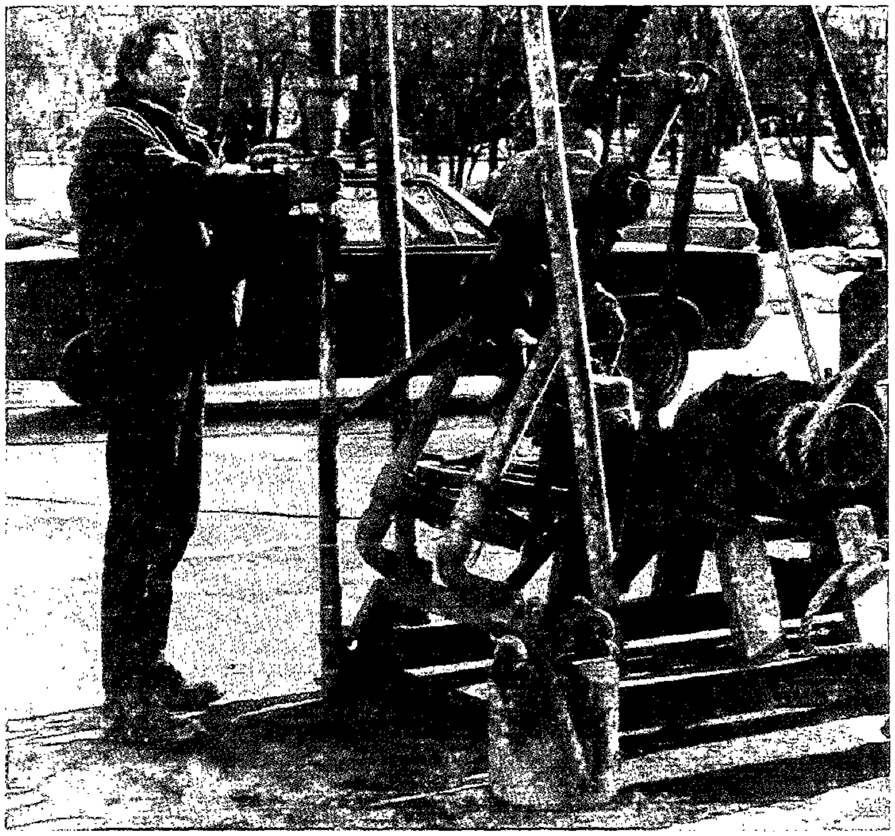


Figure 19. Standard Penetration Test.

Hammer Drop Height - The 30-in standard drop height should be carefully observed and maintained. Differing fall heights will alter the applied energy.

Sampler Geometry - ASTM specifies standard dimensions, including inside and outside diameters, and other characteristics of the split spoon sampler. Alternate geometries or configurations can change the results.

Water Level in the Casing - When performing the SPT in cohesionless soils below the water table, the level of water or drilling mud in the casing or hole must be maintained high enough to prevent upward seepage of water into the borehole. Upward seepage reduces the effective vertical stress in the soils below the borehole and can lead to reduced SPT N-value. The use of drilling mud is desirable when drilling below the water table, particularly if running sand conditions are observed. Use of casing is preferred to use of hollow stem augers to advance the hole. Hollow-stem augers should only be used if it is confirmed that the water or drilling mud is capable of maintaining the stability of the bottom of the hole.

Some settlement calculation procedures use compressibility as estimated from the Cone Penetration Test, CPT (ASTM D3441). Cone resistance has been correlated to equivalent soil modulus of elasticity. This in situ test is performed in the United States, but much less frequently than the SPT. CPT equipment is becoming more available in the U.S., and is in use by the FHWA and some States. Problems associated with variation in test procedures appear to be less of an issue with CPT, in part because it is a more specialized test requiring special equipment and trained personnel. However as with any in situ test, variability and scatter in results occur. The CPT is best suited for use in sandy soils containing little or no gravel. Gravelly or cobbly soils can give misleading results or even prevent penetration of the cone. CPT test procedures are summarized by Schmertmann (1978).

Correlations between observed footing or structure settlement and the results of in situ penetration testing such as the SPT or CPT introduce uncertainty in settlement calculations. It is implicitly assumed in the correlations that the quantity measured in the in situ test can account for all important factors affecting sand compressibility. Although conventional in situ tests can reflect many of the important factors, they do not account for all the sand compressibility characteristics that influence footing settlement.

Less common methods such as plate or screwplate load tests, dilatometer or pressuremeter tests can often provide better information on sand compressibility than can penetration tests. The lesser availability and higher cost of these tests has inhibited their routine use in most parts of the country, although they are used in certain parts of the U.S. There has been less experience in the U.S. with use of these tests, and fewer data available on the accuracy of settlement calculations based on them, compared to SPT or CPT. Because of the lack of availability of necessary test equipment and the lesser experience in their use, these methods are not at present feasible for routine use in design of bridge foundations.

Bearing Capacity

The ultimate bearing capacity of a footing is the load or soil pressure at which the footing shears through or punches into the supporting soil. For sand, the ultimate bearing capacity depends primarily on the relative density of the soil and the confining pressure (depth of footing embedment). A minimum factor of safety of three (3.0) against bearing capacity failure is common design practice for footings on sand.

Bearing capacity will usually not be a controlling factor in footing design for sands having Standard Penetration Resistance N-values exceeding about 10 blows per ft. Similarly, the bearing capacity of footings on sand is rarely a concern for footings larger than 3 to 5 ft in their smallest plan dimension. For footings larger than 3 to 5 ft, designing for a tolerable settlement will normally provide the necessary safety against bearing capacity failure.

Calculations of bearing capacity of footings on sand depend on the footing plan dimensions, embedment depth below ground surface and the position of the water table, the soil friction angle and unit weight. Procedures for calculating ultimate soil bearing capacity, under vertical or inclined loading, are well documented in soil mechanics literature and textbooks (NAVFAC, 1982; Terzaghi and Peck, 1967; Peck et. al., 1974 for example).

The allowable bearing pressure (used to size the footing) is the average pressure at the base of the footing such that 1) an adequate factor of safety against bearing capacity is provided and 2) the expected settlement is acceptable. In practice, a bearing pressure obtained by dividing the ultimate bearing capacity by three is often used to develop trial footing sizes for settlement calculations.

Footing Contact Pressure

Contact pressure is the vertical soil reaction stress acting on the base of the footing. The actual distribution of contact pressure depends on the loading conditions, the relative rigidity of the footing compared to the soil, and the stress-strain characteristics of the soil.

One method of evaluating contact pressure is to model the soil as an elastic half-space. Under this assumption, footing on the surface of elastic "soil", the distribution of contact pressure for a concentrically loaded footing is a function of relative footing stiffness only, and lies between the following two extreme conditions:

- Rigid Footing - All points on a rigid footing settle uniformly. Theoretically, the contact pressure is nearly infinite under the edge of the footing, and less than the average pressure at the center.

- Flexible Footing - The center of a flexible footing settles more than the edges, and the contact pressure is uniform.

Reinforced concrete footings bearing on sand are relatively rigid compared to the soil. If the soil were perfectly elastic, the contact pressure under the rigid footing would be very high at the edges compared to the center. However for footings bearing at typical depths in real sand soils, the shear strength of the sand at the edge of the footing is limited because of relatively low confining pressure, which reduces the stress at the edge. The combined effect of rigid footing and low soil shear strength at the footing edge results in a contact pressure distribution somewhere in between the two extremes described above. The distributions are illustrated in figure 20a.

As the load on a footing increases, the pressures beneath the footing increase and approach the distribution at failure shown on figure 20b (Terzaghi, 1943). The distribution at failure is shaped as shown because the cohesionless soil derives its strength from the confining pressure. Under the center of the footing, the footing load provides the maximum

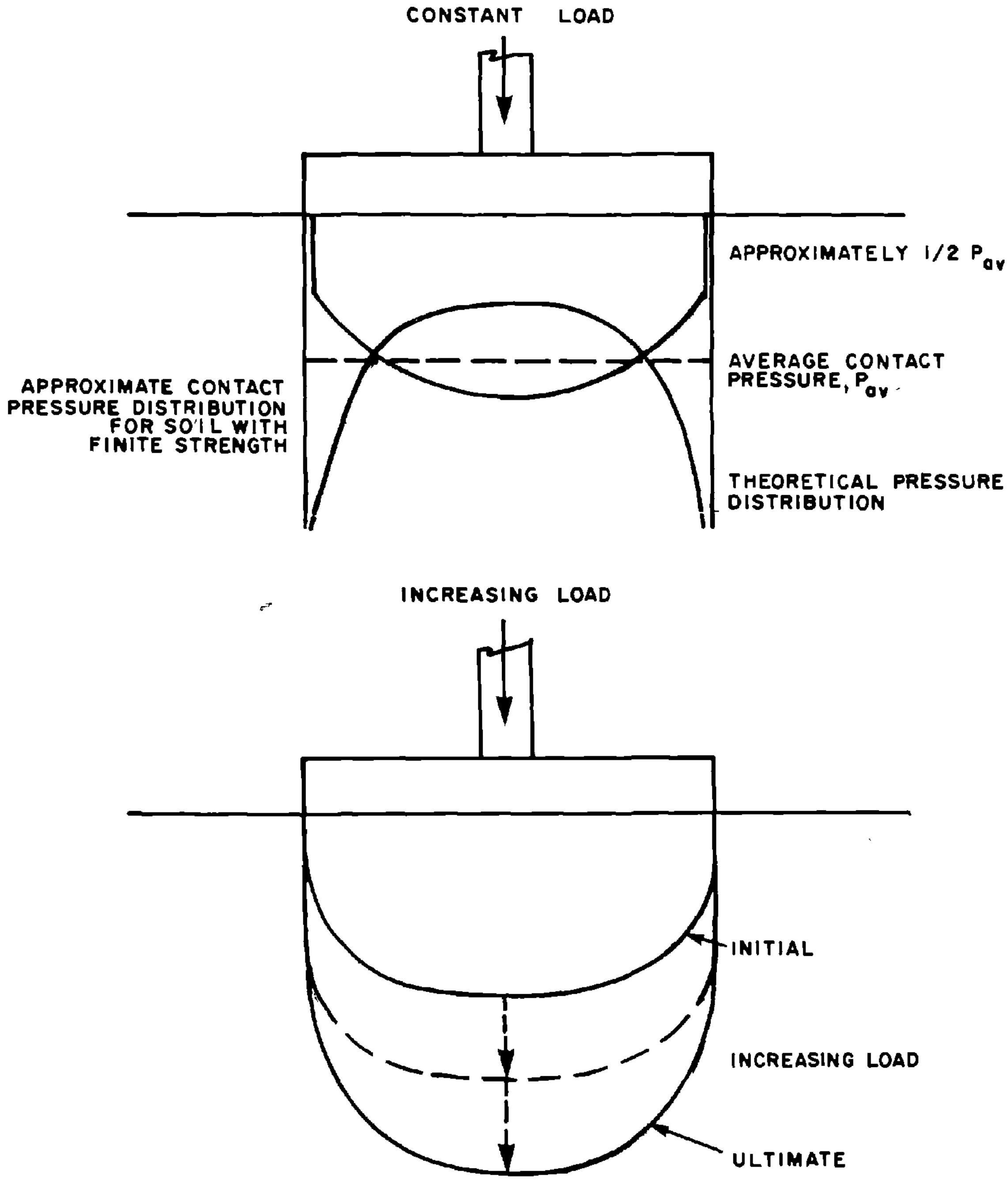


Figure 20. Theoretical distribution of footing contact pressure.

confining pressure, so the strength of the sand and the contact pressure are also maximum. The average contact pressure at failure is the ultimate bearing capacity.

The usual assumption in the structural design of a footing is that the contact pressure under the footing is uniform (concentric loading) or varies linearly (under eccentric loading). In reality, the contact pressure is not linear or uniform, especially under footings subjected to complex loading conditions or having irregular plan shapes.

As discussed under the report section, Monitoring Bridge Foundation Performance, individual contact cell pressures measured at the study bridges often differed markedly and erratically from predicted values. Experience has demonstrated that appropriate structural designs can be achieved using the conventional assumption of uniform or linear pressure distribution for concentric or eccentric loading, respectively.

Procedures for determining a distribution of contact pressure under concentric and eccentric loading, for use in structural design of footings, are described in geotechnical engineering textbooks and literature (Peck, et. al. 1974, NAVFAC 1982, FHWA 1983). The most commonly assumed distributions are shown on figure 21.

The distribution of contact pressure is not normally considered explicitly in calculations of footing settlement. The pressure distribution is accommodated implicitly in the calculation equations and methods. It should be noted that settlement calculation methods, such as discussed later in this report, have been developed for relatively rigid footings. As a general rule such methods should not be used, or be used only with recognition of their limitations, for calculating settlement due to embankment loadings or of large mat foundations. This limitation is appropriate because the implicit rigid footing contact pressure distribution assumed by the methods is not consistent with the more nearly uniform stress distribution produced under embankments or large mats.

Settlement and Sand Compressibility

Many methods have been developed by various researchers and engineers to predict or calculate settlement of concentrically loaded footings bearing on sand. The state-of-the-art methods for calculating settlement of footings on sand are not as standardized, and possibly not as accurate, as those for cohesive soils. This is due in part to the difficulty and limited accuracy in estimating sand compressibility, and the natural variability of sand deposits in situ.

In the U.S., the typical procedure for determining compressibility of cohesive soils is by laboratory testing of "undisturbed" soil samples. Recovery of undisturbed samples of naturally-deposited sand, for purposes of laboratory determinations of in situ compressibility, is not normally feasible. Disturbance during sampling, handling and sample preparation destroys in situ stress conditions and particle arrangement. Such disturbance masks the stress history of the sample and changes the sample compression characteristics. It is also difficult or impossible to re-create the in situ confining stress conditions in the laboratory, which are very important to the stress-strain characteristics of sand.

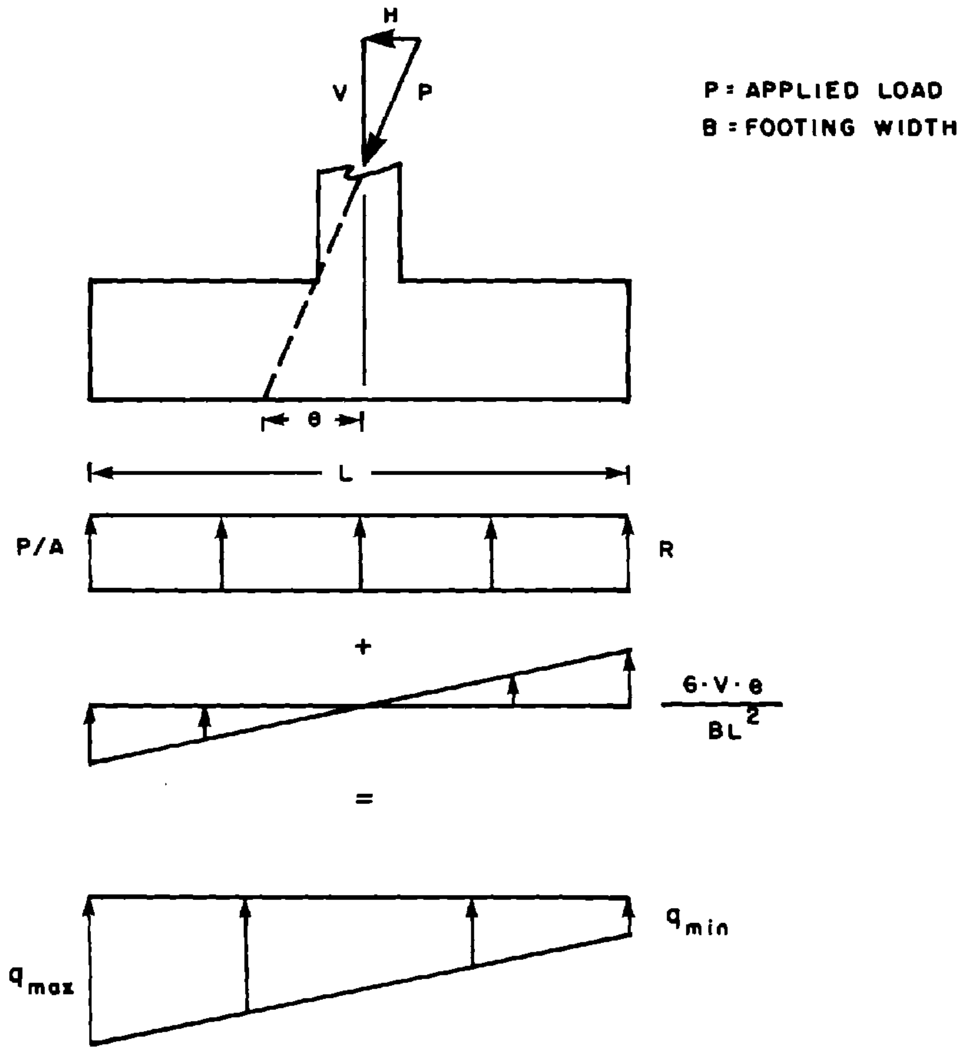


Figure 21. Linear distribution of footing contact pressure.

Because of these difficulties, all of the more common settlement calculation methods rely on empirical correlations between in situ tests and sand compressibility. Some of the methods are based solely on empirical procedures, some on the theory of elasticity and others on principles of one-dimensional compression. No single method has been generally accepted as giving the best results.

The methods account for soil compressibility, footing loading, footing geometry, soil preloading, depth of embedment, position of water table, thickness of the sand layer and time. The methods differ greatly in their procedure for assessing sand compressibility, and in their assumptions of the relative importance and effects of the other factors noted above.

Settlement Due to Embankment Loading

Embankments behind bridge abutments cause settlement of abutment footings by increasing the vertical stresses in the soil below the abutment footing and by increasing the load on the footing itself. Most of the design procedures discussed in Settlement Calculation Methods were developed for rigid footings of finite plan dimensions and are not applicable to this loading condition. Two methods which can be used for this settlement calculation are the Buisman-DeBeer (1965) and Hough (1967) methods which are one-dimensional compression approaches. The increased vertical stress in the ground can be calculated by elastic methods (Poulos and Davis, 1974). It should be noted that Martens and DeBeer (1977) state that the Buisman-DeBeer method can yield very conservative settlement estimates for large area loadings such as embankments.

Studies of highway bridges in Belgium in the 1940's (DeBeer, 1948) indicate that settlement of bridge abutment footings caused by 20-to 25-ft high embankments behind the abutments can equal or exceed the settlement of the footing caused by the bridge dead and live load. Settlements of abutment footings for the instrumented bridges of this study were only slightly greater than settlements of the pier footings.

Settlement Under Lateral or Eccentric Loads

Lateral or eccentric loads can cause non-uniform settlement (rotation) of a footing. The available procedures for predicting footing rotation provide only very approximate results.

One procedure for estimating footing rotation is to 1) calculate the maximum and minimum contact pressure under the eccentric loading, figure 21; 2) use one or more of the settlement calculation methods to estimate footing settlement under the two extreme pressures; and 3) assume that the two extreme calculated settlements represent the minimum and maximum settlement of the two opposite edges of the footing.

Another approach, based on the theory of elasticity for rotation of a rigid footing, is given by Poulos and Davis (1974):

$$= \frac{M\,(1 - \nu^2)}{B^2\,LE}\; I_\theta \qquad \text{(Eq. 1)}$$

where:

= the angular rotation of the footing in radians,
M = the moment on the footing,
ν = Poisson's ratio of the soil,
I_θ = an influence factor depending on the footing length,
B = the footing width,
E = the Young's modulus for the soil.

The determination of the Young's modulus for use in this equation is difficult. Several references (Bowles, 1977 and Canadian Manual on Foundation Engineering, 1975) should be consulted and a range of values used to check the sensitivity of the footing movement to the modulus value.

Due to rigidity of the structure, redistribution of contact pressure during rotation and other factors, these procedures will usually overpredict the magnitude of footing rotation.

The horizontal force resulting from the embankment weight acting on a bridge abutment or wing wall can result in eccentric loading of the supporting footings. The eccentric loading can in turn cause footing rotation and tilt of the abutment or wall. For a properly-designed retaining wall or abutment supported on footings on sand, tilt is typically not a concern. Measured tilt of the instrumented bridge abutments in this study was quite small, averaging 0.023 degree. By comparison, 0.3 degree (0.005 times the height) is normally considered to be required to develop the active pressure condition against the back of the wall. Tilt data for this study are summarized in table 1.

Settlement Due to Vibrations

High levels of vibrations such as those caused by earthquakes could result in significant settlement if loose sands are present within about 60-ft depth below a bridge footing. If the loose sand is saturated, liquefaction of the sand could occur during severe vibration possibly resulting in excessive settlement or a bearing capacity failure of the footing. Evaluation of the potential for such behavior should be conducted for any site where Standard Penetration Resistance N-values for the sand are less than about 15 blows per ft. In particular, the liquefaction potential of saturated sands should be investigated in areas where significant earthquakes are possible. The methodology of Seed and Idriss (1983) is the current state-of-the-art for liquefaction assessment. An FHWA report by Ferritto and Forrest (1977) provides a detailed discussion of liquefaction effects on highway bridges, and FHWA Report 86/102 entitled Seismic Design of Foundations is soon to be available.

Lower level vibrations, such as might result from wind or traffic loading, would not cause liquefaction, but the potential for settlement should be studied during design. Procedures for estimating vibration-induced settlement of sand are currently less accurate than methods for predicting settlement due to static loads, particularly for sources of vibrations other than earthquakes. The available methods are based in part on laboratory testing of sand using cyclic testing devices. Tokimatsu and Seed (1984) provide a procedure for estimating settlements due to earthquake-level vibrations. For lower level vibrations, such as might be induced by wind or traffic loading, Richart, Hall and Woods (1970) present data on vertical strain induced by vibration in laboratory resonant column tests which can be used to estimate settlement. For most situations, settlements due to low-level vibrations are not a concern.

Settlement and Key Factors

There are a number of factors which can influence settlement of footings on sand and should be recognized when performing calculations, but are not explicitly accounted for in most calculation methods. Such factors include:

1. Sources of "static" load on bridge footings include structure dead weight, snow or water loads, some component of vehicle weight, and for abutments the horizontal and vertical components of fill weight. Loads from all these sources should be considered in the settlement calculations. Possible densification and settlements of sand due to dynamic loads such as vibrations from wind, traffic or earthquakes should also be considered.

2. Only a few calculation methods explicitly provide a means to account for the effects of soil preloading. The commonly-used methods are considered to be applicable to normally loaded sands. The settlement of a footing on sand that has been preloaded can be as little as 1/2 to 1/10 the settlement of a normally loaded sand having the same relative density. Preloading can sometimes be determined by knowledge of the geologic history of the soil deposit or of man-made causes of preloading. It is generally believed that a decrease in compressibility due to preloading cannot be detected by penetration testing (SPT or CPT). Plate load tests, screwplate tests, pressuremeter tests and dilatometer tests have been used with varying success to determine preloading.

3. Each calculation method requires determination of soil compressibility parameters and each method was developed using a specific procedure for assessing compressibility. The compressibility values used in each method are not unique properties of the soil, but rather are parameters resulting from correlations with observed settlements. Proper application of the methods requires that the compressibility parameters be evaluated in the same manner as was used in the original development of the methods.

   When using calculation methods based on the determination of an elastic soil modulus from SPT or CPT results, it is important that the modulus be estimated using the same modulus vs. penetration test correlation developed or used specifically for the method. If the modulus was based on an SPT correlation, a modulus from other tests such as pressuremeter, plate load test, seismic test or other tests should not be used. However, if data from the "correct" penetration test are not available, it may be reasonable to estimate the penetration test value from other tests and then use the estimated penetration test value to obtain the modulus. For example, if only SPT data are available when using a CPT-based method, a correlation from SPT to CPT could be used, and then the modulus could be obtained from the correlated CPT value. Use of the additional correlation in this manner will add uncertainty to the calculation results.

Engineering Practice - Settlement and Differential Settlement

A common practice for predicting settlement of footings on sand is to use one or more of the available calculation methods. Engineering judgement is then used to select one of the results, or average the results, based on the range of settlement values obtained. This has been proven to be a valid and appropriate approach. Experience has shown that structure foundations consisting of footings designed in this manner have a very high probability of acceptable performance.

A practical method for calculating differential settlement between adjacent footings on sand involves one or more of the following concepts:

1. If borings are performed at each footing location, calculate the differential settlement as the difference in the estimated total settlement of each footing, calculated based on the individual borings.

2. As suggested by Terzaghi and Peck (1967), if footings are about the same plan dimensions, calculate maximum differential settlement as 50 percent of the maximum total settlement. If footings are of different sizes, calculate differential as 75 percent of the maximum total value.

3. If the penetration resistance of the soil is highly variable from boring to boring, calculate maximum differential settlement as 100 percent of the maximum total settlement.

3.3 SETTLEMENT CALCULATION METHODS

General

Available methods for calculating settlement of footings on sand were developed primarily in connection with design of foundations for buildings. However the methods are used routinely in the design of footings for many other types of structures including bridges, tanks, silos and towers. Some of the methods have gained acceptance in geotechnical engineering practice and have been reviewed in published literature. Others have not been widely used or discussed.

More than twenty "methods" of calculating settlement of footings bearing on sand have been identified in the published geotechnical literature. Many of the "methods" offer unique procedures, while some of them offer significant changes or enhancements to a previously-published approach. One of the primary objectives of this study was to review the many methods and select one or more for further evaluation. The selection was conducted in two stages:

1. Methods were selected for preliminary evaluation. Data from published case histories were used to compare selected methods.

2. Based on these results, methods were selected for final study and comparison, using data from the instrumented bridges.

During the preliminary stage of the study, fifteen of the twenty "methods" which were considered the most promising were reviewed in detail. Summaries of the 15 methods are given in appendix D. The methods can be grouped into three basic categories:

1. Empirical approaches - To differing extent, all 15 methods rely on or were calibrated with observations of settlement and could be considered somewhat empirical. Some methods rely primarily on statistical correlations among the various factors affecting settlement rather than on a theoretical model. For purposes of this study, such methods have been designated as empirical.

2. Approaches based on the theory of elasticity - These model the footing/soil system as a loaded area on or within an elastic half-space.

3. One-dimensional compression approaches - These are based on linear void-ratio vs. logarithm of effective vertical stress relationships.

Of the 15 methods summarized in appendix D, eleven are considered empirical, two are elastic, and two are one-dimensional compression approaches.

As noted above, all of the available methods rely to some degree on empirical relationships between observed footing performance and parameters used in the calculations. The empirical and elastic methods are only applicable to footing settlement calculations, not for settlements caused by large area loads such as embankment fills or grade changes. As discussed below, one-dimensional compression methods are more appropriate for calculating settlements caused by large area loadings.

Each category is discussed below.

Empirical Methods

The 11 settlement calculation techniques based primarily on empirical correlations, and summarized in appendix D, are listed below:

1. Terzaghi and Peck (1948)
2. Meyerhof (1965)
3. Alpan (1964)
4. Peck and Bazaraa (1967)
5. Peck, Hanson and Thornburn (1974)
6. Parry (1971)
7. Schultze and Sherif (1973)
8. Burland and Burbidge (1984)
9. NAVFAC DM-7 (1982)
10. Menard (1975)
11. Schmertmann (1970)

Methods 1 through 8 rely on the Standard Penetration Test to measure soil compressibility. Method 9 is based on a relative density determination, usually correlated with SPT or CPT. Methods 10 and 11 rely on Pressuremeter and CPT tests, respectively.

Methods 2, 3 and 4 are variations of the original Terzaghi and Peck approach. The Terzaghi and Peck method was based on a conservative interpretation of data on plate load tests and observed settlement of footings on sand. This has been recognized to be very conservative in calculating settlement, and methods 2, 3 and 4 have modified the original approach to give results closer to "average" rather than upper limit settlement predictions. One well-documented criticism of methods 1 through 4 is the use of the $(2B/B+1)^2$ factor for extrapolation to larger footing sizes. This factor has been shown to be dependent on soil type and relative density by Bjerrum and Eggestad (1964).

Method 5 gives the allowable bearing pressure for one inch of settlement, as opposed to a direct calculation of settlement. Methods 6, 7 and 8 are empirical methods not based on the original Terzaghi and Peck equation. These latter three methods have not been as well-studied in the literature as Methods 1 through 4. Method 9 involves correlation with relative density which can add uncertainty to the reults. The method is reported to underestimate settlements where sand thickness is small relative to the size of the loaded area.

Method 10 is based on the pressuremeter modulus from the Menard pressuremeter test. This test equipment is not widely used in the United States, and the disturbance of the sides of the borehole by the drilling operations prior to testing can have a significant influence on the pressuremeter modulus. The use of a self-boring pressuremeter may reduce disturbance of the sides of the borehole, but the empirical settlement method requires revision to reflect the use of the self-boring device.

The Schmertmann method, number 11, has gained considerable popularity over the past 10 years, especially in the southeastern U.S. The method is partially based on elastic theory, and so is not completely empirical. The elastic soil modulus used is calculated from the point resistance of the static cone penetrometer in a CPT. Although cone penetration testing is not routine for highway bridge projects, correlations between SPT and CPT resistance are available. However the error associated with such correlations adds additional uncertainty to settlement calculations using the method.

Elastic Methods

The D'Appolonia method (1968), one of the two procedures based on the theory of elasticity, is based on measurements of footing settlements on a large site in Indiana. D'Appolonia developed a correlation of elastic modulus vs. SPT resistance for both normally loaded and preloaded sands. One feature of the method is that it provides an explicit means to account for preloading of the sand in the settlement calculations. The data base for the correlations is relatively small, especially for preloaded sands. However, the method is relatively simple to use and accounts for the major factors affecting footing settlement.

The second elastic method is a relatively new approach by Oweis (1979). The method involves multiplying an initial elastic modulus by a reduction factor to account for the reduced stiffness of soils at higher shear stresses. The reduction factor is determined from an initial settlement calculation using the initial elastic modulus. The reduced modulus is then used in an elastic equation to calculate settlement of the footing. Potential advantages of this method are:

1. It models the expected behavior of increasing incremental amounts of footing settlement with increasing load.

2. It may be possible to use methods such as in situ shear wave velocity measurements to determine the initial elastic modulus. Oweis provides a correlation of SPT resistance with initial modulus based on plate load test results.

The main disadvantages of the method are that the calculations are more lengthy than for other settlement methods, and the method does not distinguish between normally loaded and preloaded sands.

One-Dimensional Compression Methods

One-dimensional compression methods are typically based on the assumption of a linear relationship between void ratio and the logarithm of effective vertical stress. This assumption is commonly made for consolidation settlement calculations for clay soils. The log-linear relationship implies that the incremental change in footing settlement decreases with increasing load, which is opposite to the observed settlement behavior of footings on sand. The decreasing incremental settlement behavior is more consistent with settlement under the center of large mat foundations or embankments than with bridge footings. These methods, however, have been used successfully in design calculations for bridge footings.

Two one-dimensional compression methods were identified in the literature. Hough's method (1959) utilizes SPT data to determine sand compressibility, while the Buisman-DeBeer method (1965) makes use of CPT results. The Buisman-DeBeer method was shown by DeBeer to overestimate settlement of bridge footings on sand by an average factor of two. Meyerhof proposed a revised sand compressibility equation to reduce the conservatism of the Buisman-DeBeer method.

Since the one-dimensional compression methods do not model the observed behavior of footings under increasing load, they are not considered as appropriate as other approaches for calculating settlement of footings. However, these methods can be used for calculating the settlement of bridge abutments as a result of the weight of the abutment backfill. These methods can also be used for estimating settlement caused by grade changes at a site. It should be noted that DeBeer and Martens (1956) have concluded that DeBeer's method also overestimates settlement under embankments or for mat foundations.

Selection of Methods for Preliminary Study

Methods were to be selected for further study using literature case history data, to include a variety of approaches and assumptions. Promising methods were chosen on the basis of documentation of their accuracy in the published literature, on the rationality of the approach, on their anticipated applicability to settlement of bridge footings, and consistency with bridge exploration and design procedures. Based on these criteria, four methods were selected for preliminary study:

- The Peck and Bazaraa method, an empirical method which utilizes SPT data to assess compressibility.
- The Schmertmann method, which is a semi-empirical approach, based on CPT data.
- D'Appolonia's method, an elastic approach which makes use of SPT results in the calculation of a soil modulus.
- Oweis'method, an iterative, non-linear elastic approach which uses SPT data to estimate the elastic modulus.

As indicated above, each of the four methods makes use of the relatively common SPT or CPT in situ tests.

The Peck and Bazaraa approach was chosen as one of the most promising of the SPT methods: 1) it uses a corrected N-value and an embedment correction factor to account for the effects of relative density and overburden pressure on sand compressibility, and 2) the method is widely used in practice.

The Schmertmann method was selected because: 1) it is being increasingly used in practice; 2) it is not strictly an empirical approach, but has some basis in elastic theory; 3) it was developed using CPT data, but may be used with SPT results through empirical correlations between SPT and CPT resistance; and 4) it can account for varying compressibility with depth and also for limited thickness of sand.

D Appolonia's method has the combined advantages of 1) being an elastic approach, 2) providing a means for accounting for preloaded sand, and 3) is a relatively easy method to use.

The Oweis method, although not well known or widely used, was considered promising due to its iterative approach to estimating the effective soil modulus. It is, however, more complicated and time-consuming to use than the other methods.

Case History Review

To evaluate the selected methods, published data on settlements of footings were compared to settlements predicted by the methods. The settlement calculations were performed based on data given in the published reports. The reported and calculated settlements were then reviewed and analyzed to provide an objective basis for comparing the methods.

A search of the published literature was performed to locate case studies reporting measurements of settlement of footings on sand. Although many potential case histories were located, most were not considered suitable for use in the study for a variety of reasons including:

- o No soil boring data or insufficient data were given. In many cases, only general descriptions of soil type and density were given.
- o Actual loads on the footings were not provided. In most cases only total design loads, not calculated dead and live loads, were reported.
- o Clay or clayey soils were present below the footing.
- o The case study described a large mat or tank foundation of dimensions much larger than typical bridge footings.

The limited amount of published data was considered to be a major obstacle to valid comparisons of the methods. Also, the relatively small magnitudes of measured settlements limited the applicability of the data. However, it was decided to continue with the comparison, recognizing these shortcomings.

Five case studies comprising a total of 10 footings, as presented in table 2, were selected from papers by Bergdahl and Ottosson (1982), Wennerstrand (1979), DeBeer and Martens (1956), Levy and Morton (1974), and DeBeer (1948). The cases involved bridge piers and abutments, with the exception of the Levy and Morton case study, which was a report on building spread footings. In each case, reported subsurface information consisted primarily of cone penetration resistance. These data were converted to standard penetration resistance N-values for use in the Peck and Bazaraa, D'Appolonia and Oweis methods, using Schmertmann's (1970) correlations. The relevant data used in the settlement calculations for each footing are presented in table 3.

Bergdahl and Ottosson describe a bridge pier supported on medium dense to dense silt and sand to a depth of about 12 ft, with an underlying deposit of medium dense sand. Two CPT tests were performed at the pier. The authors state that the penetration tests indicate inconsistent density of the sand, which they attributed to variations in grain size.

Wennerstrand analyzed a bridge on shallow foundations. The soil is described as being a loose, slightly organic fine sand to a depth of about 30 ft, with interbedded silt and clay below. CPT tests were performed subsequent to bridge construction in a test area located between two piers.

DeBeer and Martens provide data on several bridges, two of which were used in the case history study. Bridge XXIX in Loppem is supported on two abutments and a central pier. CPT data are provided, and the soils are described as fine sands and silty sands. The bridge in St. Denys - Westrem is supported on two abutments and two piers. The soil at this site is described as layered sand with silt inclusions and CPT data are presented in the paper.

Table 2. Case histories.

LOCATION AND STRUCTURAL ELEMENT	LITERATURE REFERENCE	ELEMENT DESIGNATION
Alvsbyn Bridge Pier (Sweden)	Bergdahl and Ottosson (1982)	C1
Sweden Support No. 23 (Sweden)	Wennerstrand (1979)	C2
Loppem Central Pier (Belgium)	DeBeer and Martens (1956)	C3
St. Denys-Westrem Brussels Abutment (Belgium)	DeBeer (1948)	C4
St. Denys-Westrem Central Pier (Belgium)	DeBeer and Martens (1956)	C5
3 Footings (England)	Levy and Morton (1974)	C6
Gentbrugge Pier A (Belgium)	DeBeer (1948)	C7
Gentbrugge Pier B (Belgium)	DeBeer (1948)	C8
Gentbrugge Brussels Abutment (Belgium)	DeBeer (1948)	C9
Gentbrugge Ghent Abutment (Belgium)	DeBeer (1948)	C10

Levy and Morton discuss the settlement of three (out of a total of eight) footings used to support twin twelve-story buildings. The subsurface soils were described as dense sands and gravels. Cone penetration tests were performed, and the results were converted to SPT N-values by Levy and Morton which were provided in the paper. Neither the actual values of cone penetration resistance nor the correlation used to calculate the SPT N-values were provided by the authors.

The second case history reported by DeBeer (1948), involved a 200 ft long highway bridge in Belgium. The bridge was supported by two abutments and two piers. The soil type was described as sand, with CPT data presented in the paper. Only loads and settlements due to the dead weight of the abutments and piers themselves were considered in order to avoid the effects of the embankments and other fill loads.

The settlements calculated using the four selected methods, Peck and Bazaraa, D'Appolonia, Schmertmann, and Oweis, are shown in table 4 together with the measured settlements for the ten footings. Calculated and

Table 3. Literature case history data.

ELEMENT DESIGNATION	q (ksf)	N_f (blows/ft)	N_c (blows/ft)	γ (kcf)	$\underline{\underline{\nabla}}$ (ft)	B (ft)	L (ft)	D (ft)	q_c (kg/cm^2)	H (ft)	$\bar{\sigma}_{vo}$ max
C1	3.80	21 to 24*	24	.120	8.2	16.4	28.0	8.2	85	54.3	2
C2	2.06	5*	7	.108	0	10.9	47.7	6.6	159	26.3	2
C3	4.82	40 to 42*	50	.120	5.2	9.8	32.9	9.7	130	48.5	2
C4	1.52	18*	17	.120	+1.6	19.0	78.9	8.2	70	53.0	2
C5	4.10	7*	9	.120	0	8.5	68.9	6.6	64	55.0	2
C6	10.60	38*	32	.120	21.6	13.0	23.0	16.4	210	29.5	2
C7	3.30	32*	42	.120	+3.6	19.7	52.5	9.2	120	50.0	2
C8	4.48	33*	42	.120	+3.6	19.7	52.5	11.8	120	50.0	2
C9	2.74	34*	42	.120	+3.6	23.0	118.0	7.6	120	39.0	2
C10	2.00	34*	42	.120	+3.6	17.0	92.0	7.6	120	39.0	2

* Converted from CPT data.

Definitions:

q = footing bearing pressure (average).
N_f = field SPT N-value (range of N-values is due to different depths of influence for different settlement calculation methods).
N_c = corrected N-value (corrected for overburden per Peck and Bazaraa, 1967).
γ = soil total unit weight (assumed); to kips per cubic foot; 1 kip equals 1000 lb.
$\underline{\underline{\nabla}}$ = depth to water table (below footing bearing elevation). (+) indicates water table is above footing bearing elevation.
B = footing width.
L = footing length.
D = depth of footing embedment below ground surface. (F) indicates footing is on new fill.
q_c = static CPT cone resistance (multiple values indicate that profile was subdivided into layers with corresponding q_c values).
H = depth below footing to (relatively) incompressible stratum. (H>2B indicates that incompressible stratum is located below the depth of influence.)
$\bar{\sigma}_{vo}$ max = indicates soil stress history

1 = soil is preloaded.
2 = soil is normally loaded.
3 = soil is partially preloaded.

Table 4. Calculated versus reported settlements - literature case histories.

ELEMENT DESIGNATION	CALCULATED SETTLEMENT (in) D'Appolonia	Oweis	Peck and Bazaraa	Schmertmann	REPORTED SETTLEMENT (in)
C1	0.55	1.37	0.67	1.57	0.47
C2	0.39	1.44	1.06	3.03	1.46
C3	0.51	0.80	0.39	0.94	0.83
C4	0.35	0.79	0.35	0.67	0.47
C5	0.79	2.30	1.54	2.05	1.30
C6	1.02	1.07	0.94	0.91	0.47 (avg)
C7	0.35	0.14	0.28	0.31	0.31
C8	0.31	0.11	0.24	0.24	0.16
C9	0.47	0.23	0.31	0.51	0.47
C10	0.31	0.10	0.20	0.31	0.39

Table 5 Ratio of calculated/reported settlements - literature case histories.

ELEMENT DESIGNATION	CALCULATED/REPORTED D'Appolonia	Oweis	Peck and Bazaraa	Schmertmann
C1	1.17	2.91	1.42	3.34
C2	0.27	0.99	0.73	2.08
C3	0.61	0.96	0.47	1.13
C4	0.74	1.68	0.74	1.43
C5	0.61	1.77	1.18	1.58
C6	2.17	2.28	2.00	1.94
C7	1.13	0.45	0.90	1.00
C8	1.94	0.69	1.50	1.50
C9	1.00	0.49	0.66	1.09
C10	0.79	0.26	0.51	0.79
Mean	1.04	1.25	1.01	1.59
Standard Deviation	0.60	0.88	0.50	0.74

Table 6. Calculated minus reported settlements - literature case histories.

ELEMENT DESIGNATION	CALCULATED-REPORTED ("Difference") D'Appolonia	Oweis	Peck and Bazaraa	Schmertmann
C1	0.08	0.90	0.20	1.10
C2	-1.07	-0.02	-0.40	1.57
C3	-0.32	-0.03	-0.44	0.11
C4	-0.12	0.32	-0.12	0.20
C5	-0.51	1.00	0.24	0.75
C6	0.55	0.60	0.47	0.44
C7	0.04	-0.17	-0.03	0
C8	0.15	-0.05	0.08	0.08
C9	0	-0.24	-0.16	0.04
C10	-0.08	-0.29	-0.19	-0.08
Mean	-0.13	0.20	-0.03	0.42
Standard Deviation	0.43	0.48	0.29	0.55
Using Absolute Values of "Difference":				
Mean	0.29	0.36	0.23	0.44
Standard Deviation	0.33	0.36	0.15	0.54

measured settlement are compared graphically in figure 22. Two parameters were used to evaluate the accuracy of the calculations: the ratio of calculated to measured settlement (hereinafter referred to as the "ratio"), and the difference between calculated and measured settlement (the "difference"). These parameters are shown in tables 5 and 6, and plotted in figures 23 and 24, for each footing.

Each of the methods had mean ratios greater than 1.0. The mean ratio was closest to 1.0 for the Peck and Bazaraa (1.01) and D'Appolonia (1.04) methods. The mean ratio for the Schmertmann method was the highest (1.59). Although the ratios exceeded 1.0, the difference values were greater than 0.0 for only the Oweis and Schmertmann methods. The difference values were near 0.0 for Peck and Bazaraa and D'Appolonia.

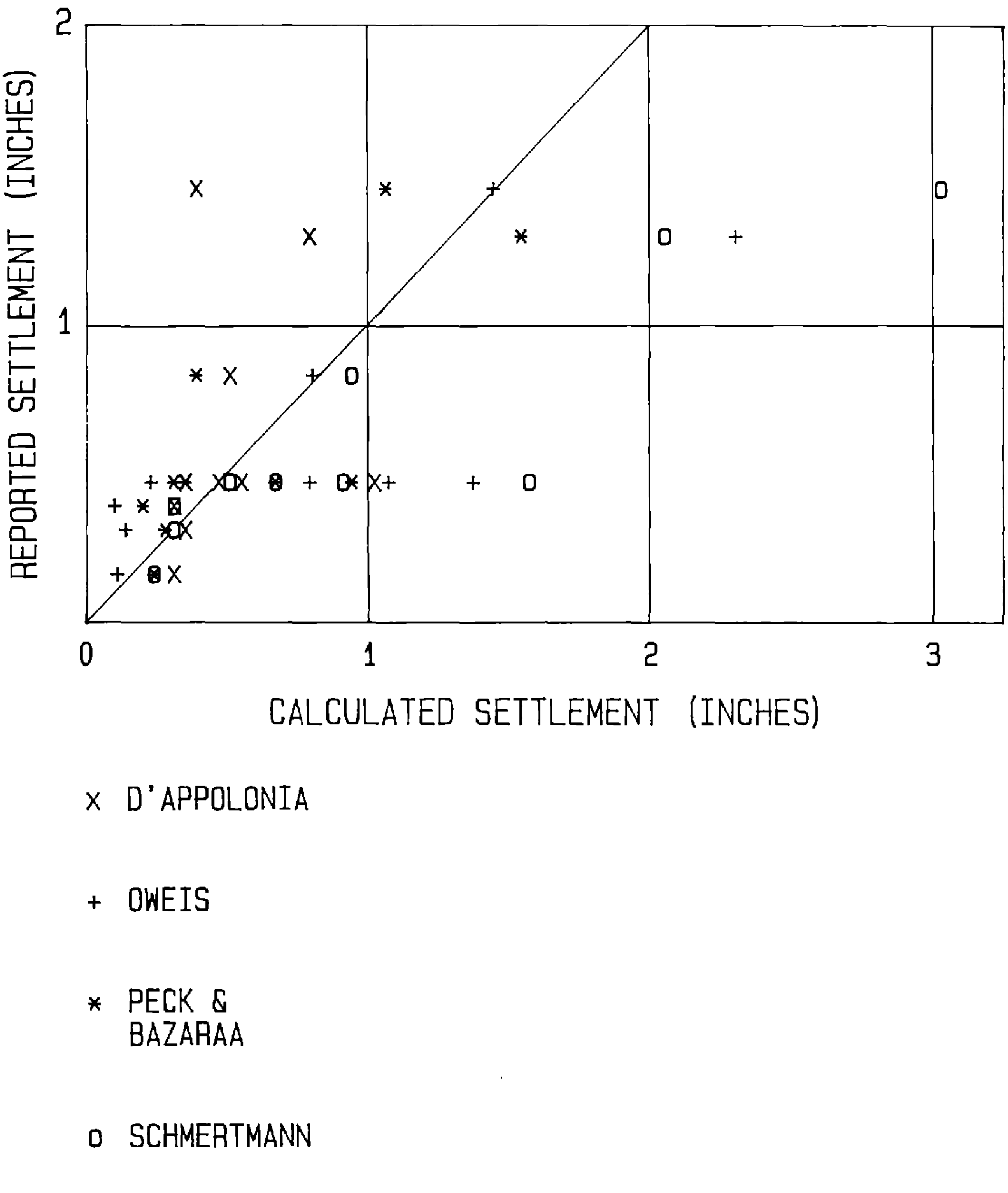


Figure 22. Calculated versus reported settlements - literature case histories.

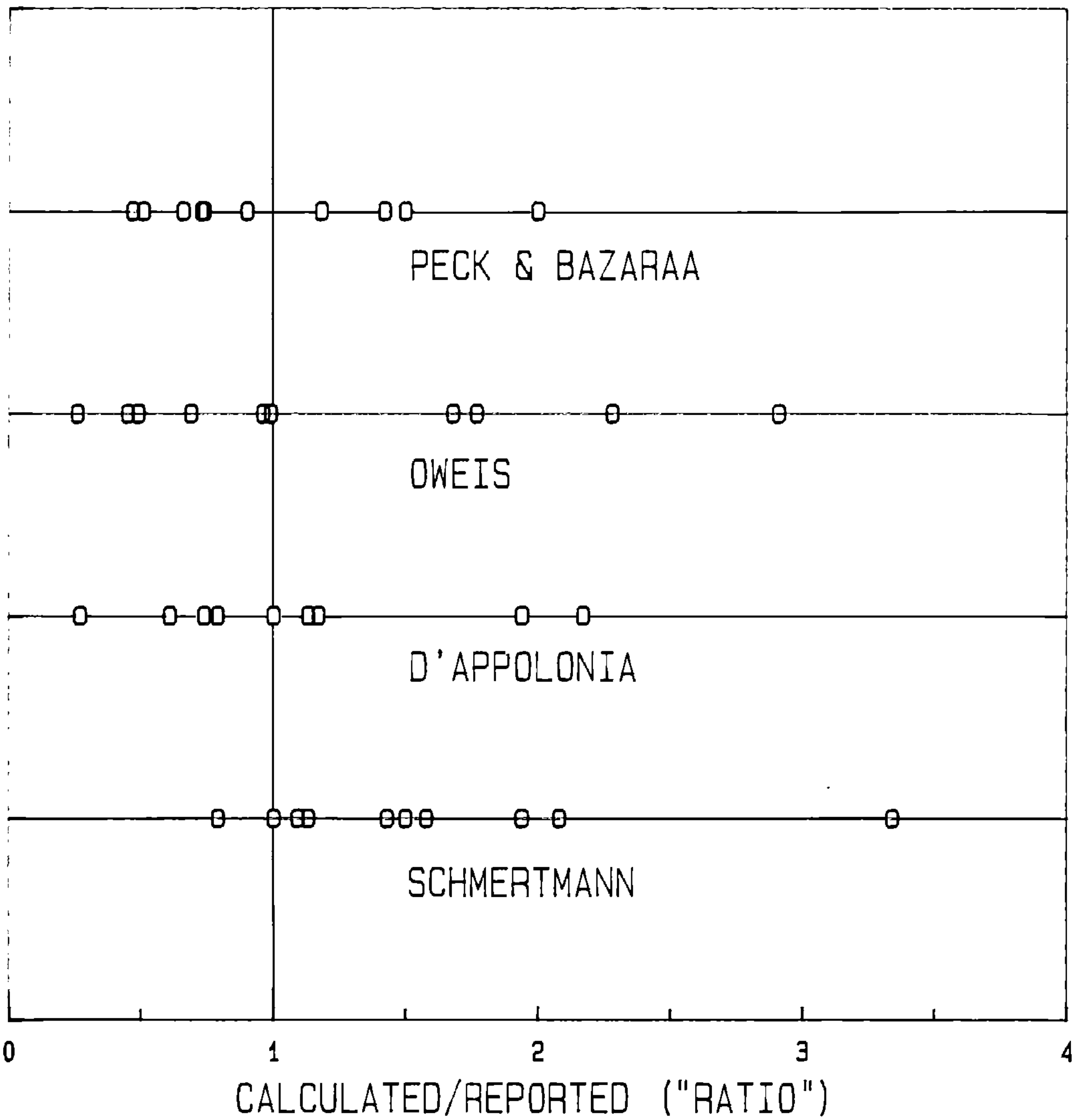


Figure 23. Ratio of calculated / reported settlements - literature case histories.

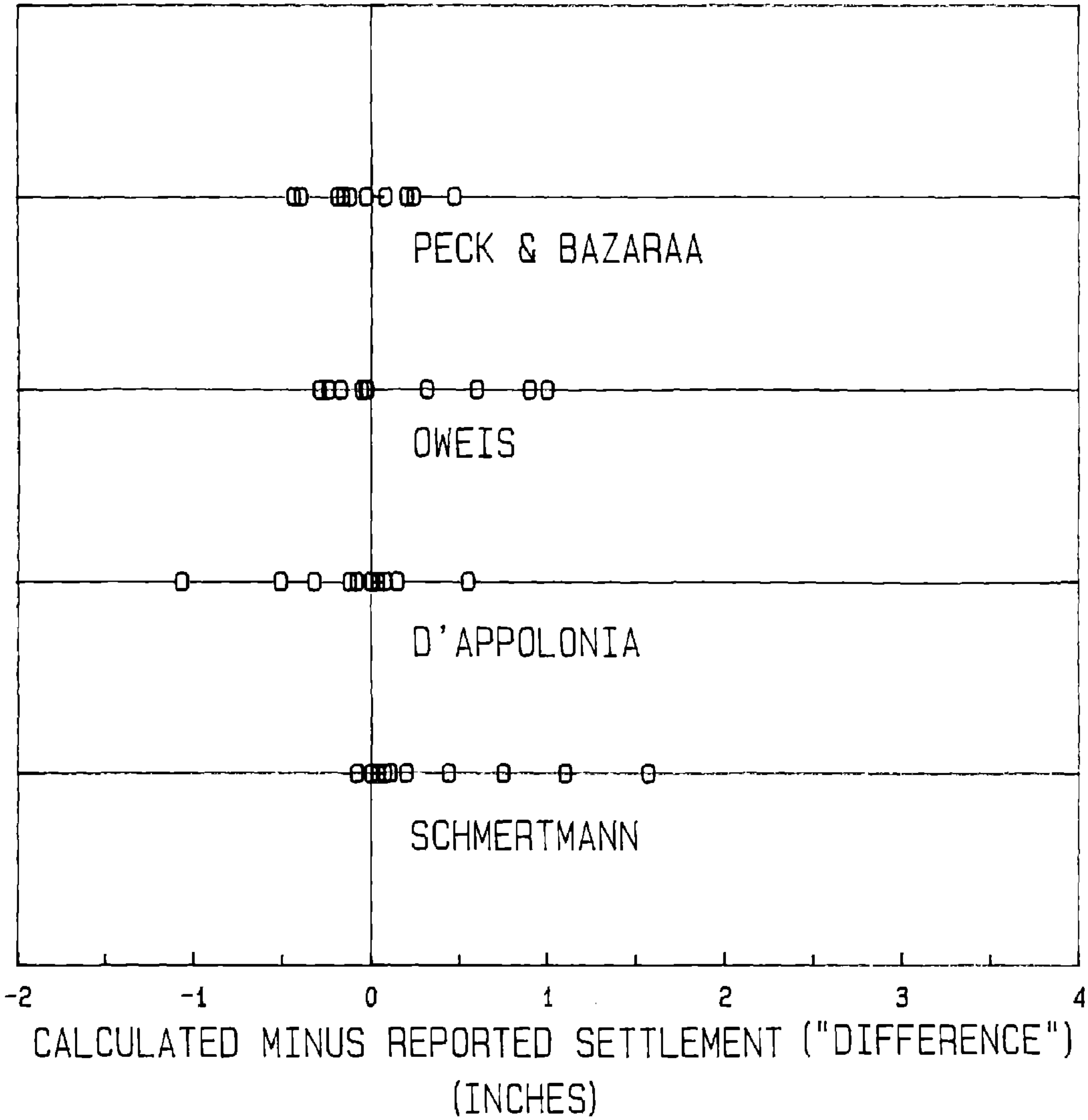


Figure 24. Calculated minus reported settlements - literature case histories.

The standard deviation of the parameters was calculated and is also shown in tables 5 and 6. The Peck and Bazaraa and D'Appolonia methods showed the smallest standard deviation of the ratio and the difference parameters. The mean and standard deviation of the absolute value of the differences was also calculated and is shown in table 6. Again, the Peck and Bazaraa and D'Appolonia results showed the smallest average absolute error and standard deviation of absolute error.

3.4 EVALUATION OF SETTLEMENT CALCULATION METHODS

Selection of Methods for Final Study

Based on the results of the preliminary study and additional assessments of the applicability of the available methods, five methods were chosen for final evaluation using the data from the instrumented bridges. Three of the four methods used in the preliminary study were included, Peck and Bazaraa, Schmertmann and D'Appolonia, as well as the following two additional methods:

- o Hough method: A one-dimensional compression method which has been in use by the FHWA and many State highway agencies, and described in the FHWA Foundation Workshop Manual (Cheney, 1983).

- o Burland and Burbidge method (1984): A recently-developed empirical method relating SPT data to sand compressibility, based on regression analysis of case studies.

The final selection of the five methods was based on several criteria including the following:

1. Ideally, the methods should have been accepted in practice and discussed in the literature over a period of time. This provides a valuable data base of experience, allowing for refinements of the technique to occur. The Burland and Burbidge method is an exception to this criterion. The method has not been widely publicized or used, in part due to the short time period since its introduction.

2. The method should have a logical basis, with results that follow expected trends. For example, the methods should predict increased incremental settlement as the footing size or bearing pressure is increased. Although the Hough method does not meet this criterion as discussed previously, the method was included in the final study because it is the method currently proposed by the FHWA.

3. Application of the method should not be overly complex or time consuming. The more straightforward the method, the less likely that errors will be made in its use, and the more likely it will actually be used. In large part, it was due to undesirable complexity that Oweis' iterative method was eventually discarded in favor of alternate, easier methods.

4. The procedure for assessing sand compressibility used by the methods should make use of readily-available in situ testing procedures and equipment. In particular, the field data should be readily obtainable by State highway engineers. In most States, Standard Penetration Testing is routinely performed and Cone Penetration Testing is becoming more commonplace. Other in situ testing procedures to assess soil relative density or compressibility (pressuremeter, dilatometer, etc.) are much less common. Consideration was also given to the type of subsurface information already available at each study bridge location,

and the availability of equipment for further exploration since further testing was to be performed during the study.

5. The selected methods should represent a cross section of method types (empirical, elastic, and one-dimensional compression).

<u>Description of the Selected Methods</u>

Summaries of the five selected methods are given in appendix D. Comments on the methods, based on this current research and evaluation, are provided below.

<u>Burland and Burbidge</u> - Burland and Burbidge established an empirical relationship between average SPT blow count, the width of the foundation B, and the modulus of subgrade compressibility. This relationship is based on a regression analysis of over 200 settlement records.

The method was developed for normally loaded sand. When the soil is known to be preloaded, the authors recommend reducing the compressibility index by a factor of three for the increment of applied stress which is less than the effective preloading stress.

SPT N-values are not corrected for overburden pressure, but are corrected if the material is either a very fine or silty sand below the water table, or if the material is a gravelly sand. The method also takes into account footing shape, thickness of compressible stratum, and creep (for time exceeding 3 years). The method assumes that no correction is necessary for the proximity of the water table.

<u>D'Appolonia</u> - This method is based on elastic theory, with the modulus of compressibility (M) backfigured from a limited number of measurements of footing settlement.

The modulus M is a function of uncorrected N-values, averaged over the depth of influence, taken as the width of the footing. No correction is made for presence of the water table.

Two influence factors based on elastic theory (μ_0 and μ_1) account for footing shape, depth of embedment and depth to incompressible material. Soils are assumed to be incompressible when the N-value exceeds 100 blows per ft.

Corrected values of μ_0 and μ_1 based on work by Christian and Carrier (1978) were used with this method during the current study. It is generally recognized that the revised values are more appropriate. However, D'Appolonia's modulus M was backfigured from case histories using the original Janbu (1966) influence values. A comparison of calculations using the original influence values and the modified values was made during this study. The comparison indicated that for cases where the ratio of footing length to width is less than about 5, and the footings bear at relatively shallow depth, (as is generally the case with bridge footings and abutments) use of the modified influence values results in little to no change in settlement prediction. Use of the modified values with D'Appolonia's modulus will result in the same or a slightly conservative (greater) settlement prediction compared to use of the original Janbu values.

Hough - Hough's method calculates settlements of sand using one-dimensional compression theory, similar to that commonly used for calculating consolidation settlement of clays. Hough provides an empirical chart relating SPT N-values to the "bearing capacity index" (C) for various soil types.

This method as presented in the FHWA "Soils and Foundations Workshop Manual" (1983) outlines a procedure for accounting for stress interaction between adjacent footings. A chart is provided for correcting the SPT N-values for overburden pressure. The case of preloaded soils is not addressed specifically by the method. As was the procedure for the Peck and Bazaraa method, calculated settlements were reduced by a factor of two during this study to account for preloading.

Peck and Bazaraa - This method is based on the original Terzaghi and Peck empirical equation with modifications made to address overconservatism of the original approach.

Standard Penetration Resistance N-values are corrected to account for overburden pressure. An embedment correction factor (C_D) is applied when the footing is constructed in an excavation and then backfilled to original ground surface. This correction reduces the calculated settlement. The method does not specifically address the case of footings placed on fill above original ground surface and judgement must be used in those situations. It has been assumed that C_D should be set equal to 1.0 if embedment results from filling above original ground surface.

The method uses a groundwater correction factor, the use of which is somewhat controversial. Some researchers suggest that the presence of groundwater is reflected in the N-values or CPT results.

The minimum width (B) is used to account for footing dimensions, with no allowance made for footing shape. Also, Peck and Bazaraa suggest no procedures for effects of adjacent footings.

No modifications are provided by Peck and Bazaraa if the sand is preloaded. For this study, however, settlements calculated by this method were reduced by 50 percent when the sand deposit was preloaded. The reduction is considered reasonable and consistent with other methods such as D'Appolonia and Schmertmann.

Schmertmann - This method incorporates a vertical strain influence factor in an attempt to model the strains occurring under the center of a loaded area. The method was developed using CPT data to determine the equivalent Young's Modulus for granular soils in compression. Although the method is somewhat empirical, it has more of a theoretical basis than do most of the empirical methods studied.

Schmertmann's method utilizes a procedure to account for increased stress due to adjacent footings by increasing the minimum width B used in the calculations (Schmertmann, 1970). The method also allows for the effect of footing shape in the calculation.

A correction factor is provided for time-related settlement (creep); however, it is regarded by some researchers as being overly conservative. It has not been applied in the calculations during this study.

The method is directly applicable only to normally loaded sands, and Schmertmann recommends reducing the predicted settlement by a factor of two if the sands are determined to be preloaded.

In cases where CPT data are not available, it is possible to use the method by converting SPT N-values to cone penetration resistance by empirical relationships. This will add to the uncertainty of the settlement predictions.

Calculation Procedures and Assumptions

The procedures summarized in appendix D for using the methods were followed in the study calculations. Example calculations illustrating the use of each method for footings under an abutment and a pier are given in appendix E. Some of the more significant assumptions used in the calculations are indicated below:

Estimating N-values in Structural Fill - When designing footings which will bear on compacted structural fill, SPT N-values or values of CPT resistance for the fill must be assumed. The gradation and density of the structural fill at the study bridges differed significantly from bridge to bridge. However for purposes of estimating settlement, a corrected N-value equal to 32 blows per ft has been used for each case of bridge footings on structural fill. This N-value was calculated assuming a relative density of 65 percent for the fill which corresponds closely to the normal compaction criteria for fill beneath bridge foundations. CPT resistance for 65 percent relative density varies with overburden pressure, but was approximated using Schmertmann's correlation (1978).

Converting SPT N-values to CPT Resistance - Correlations by Schmertmann (1970) and Robertson and Campanella (1983) were used.

Effect of Soil Preloading - The Burland and Burbidge and D'Appolonia methods account explicitly for soil preloading. Schmertmann's recommendation of a 50 percent reduction in settlement due to preloading was used for the other three methods. The only sands considered to be preloaded were those at bridge no. 3.

Effect of Nearby Footings - The Schmertmann and Hough procedures for accounting for stress interaction between nearby footings were used for those respective methods. Stress interaction was not considered for the other methods.

Other - In general, each method was applied adhering to the original authors'/ developers' procedures with the modifications indicated herein used for special conditions.

Comparison of Calculated Versus Measured Settlements

Settlements were measured on 24 bridge footings during this study, listed in table 7. Twenty-one of the footings (16 abutments and 5 piers) were considered suitable for use in comparisons of the settlement calculation methods and calculations of settlement were made using each of the selected methods for these footings. Construction problems caused significant disturbance to the soil subgrade at three footings, which is suspected of causing additional short-term footing settlement. The three footings, which were not used in the comparisons of the methods, are designated S12, S13 and S18 in table 7.

Data on footing and structure geometry were obtained from design plans and observations during construction. Loads on the footings were calculated as the tributary weight of the structure members. Subsurface soil and groundwater information was available from logs of original test borings taken at each bridge site. Supplemental test borings and cone penetrometer tests were conducted during this study to provide added SPT and CPT data and thereby improve the overall subsurface database on which the settlement calculations were based. The available subsurface data are available in plan view in appendix B.

The 21 footings represent a variety of bearing conditions and footing geometries. Footing geometries are shown in appendix B. Table 8 summarizes key information used in the settlement calculations for each footing. The data in the table indicate:

- The range in bearing pressures is relatively small, from 1.5 to 3.5 kips per ft^2.
- The footings represent a wide range in plan dimensions, with footing widths from 8 to 28 ft.
- Seven of the footings were constructed on compacted fill placed above natural soils. The remainder were placed directly on natural soil.
- Relative density of the bearing soils also differed widely, as evidenced by N-values ranging from 8 to 58 and CPT resistance ranging from 28 to 183 kg/ cm^2.

Table 7. Study bridges.

BRIDGE NO.	BRIDGE LOCATION	STRUCTURAL ELEMENT	ELEMENT DESIGNATION	NOTES
001	Highway VT127	Abutment 1	S1	
	Burlington, Vermont	Abutment 2	S2	
002	Dickerman Rd.	Abutment 1	S3	
	Cheshire, Connecticut	Abutment 2	S4	
		Center Pier	S5	
003	Branch Avenue	West Abutment	S6	
	Providence, Rhode Island	East Abutment	S7	
		Pier 1 North	S8	
		Pier 1 South	S9	
		Pier 2 North	S10	
		Pier 2 South	S11	
		Pier 3 North	S12	(1)
		Pier 3 South	S13	(1)
004	Route 28	South Abutment	S14	
	Colliersville, New York	North Abutment	S15	
005	Route 146	North Abutment	S16	
	Uxbridge, Massachusetts	South Abutment	S17	
006	VT Route 11	Abutment 1	S18	(1)
	Chester, Vermont	Abutment 2	S19	
007	Conrail over I-86	Abutment 2	S20	
	Manchester, Connecticut			
008	Tolland Turnpike	Abutment 1	S21	
	Manchester, Connecticut	Abutment 2	S22	
009	Route 84	Abutment 1	S23	
	Manchester, Connecticut	Abutment 2	S24	
010	Route 84			(2)
	Manchester, Connecticut			

Notes: 1. Construction problems at these footings resulted in disturbance to the subgrade soils and short term settlement was increased. These footings were not used in comparisons of the settlement calculation methods.

2. Total settlement was not measured.

Table 8. Study bridge data.(2)

BRIDGE NUMBER	STRUCTURAL ELEMENT	q (ksf)	N_f (blows/ft)	N_c (blows/ft)	γ (kcf)	$\underline{\nabla}$ (ft)	B (ft)	L (ft)	D (ft)	q_c (kg/cm^2)	H (ft)	$\bar{\sigma}_{vo}$ max	Notes
001	Abut 1	3.20	23 to 36	44	.120	12	17.0	63.7	F	NA	117	2(F)	
	Abut 2	2.67	60	58	.120	12	17.0	63.7	F	NA	117	2(F)	
002	Abut 1	2.32	32 to 44	43	.120	11.5	15.25	52.5	F	NA	35	2(F)	
	Abut 2	2.44	18 to 24	19	.120	9	16.75	52.5	4	NA	42	3	
	Ctr. Pier	1.88	12 to 13	12	.120	4	12.50	41.0	5	NA	40	3	
003	West Abut	1.70	18 to 20	34	.120	31	11.0	74.6	F	28,61, 90,125	155	1(F)	
	East Abut	2.34	22	22	.115	12	18.5	79.0	5	NA	130	1	
	Pier 1 N	2.10	18 to 19	18	.120	6	21.0	21.0	5	NA	150	1	
	Pier 1 S	1.50	18 to 19	18	.120	6	21.0	30.4	5	NA	150	1	
	Pier 2 N	2.34	16 to 17	20	.115	12	16.0	26.8	5	90,70,88	153	1	
	Pier 2 S	2.48	18 to 22	22	.115	12	16.0	18.5	5	74	155	1	
	Pier 3 N	1.48	13 to 14	15	.115	12	21.0	33.0	5	NA	120	1	(1)
	Pier 3 S	1.60	23 to 28	25	.115	12	21.0	30.0	5	NA	120	1	(1)
004	South Abut	3.30	21	21	.120	28	8.1	42.9	F	165	197	2(F)	
	North Abut	3.43	8	8	.120	26.5	8.1	42.9	F	53	>150	2(F)	
005	North Abut	2.40	34 to 37	42	.120	10	16.75	76.9	6	NA	52	2	
	South Abut	2.34	21 to 27	24	.125	8	15.25	76.1	6.5	NA	51	2	
006	Abut 1	1.88	37 to 53	55	.120	2	15.25	61.7	9	NA	10	2	(1)
	Abut 2	1.79	25 to 34	39	.120	2	15.25	67.3	9	NA	10	2	
007	Abut 2	2.14	19 to 22	24	.113,.115	44	28.0	28.0	0	62,131	>2B	2	
008	Abut 1	3.01	25 to 26	23	.115	0	20.0	100.8	22	NA	>2B	3	
	Abut 2	3.25	26 to 31	38	.115	1	20.0	100.8	5	NA	>2B	3	
009	Abut 1	3.51	33 to 40	39	.115,.120	17	21.75	44.4	F	NA	41	2(F)	
	Abut 2	3.37	37 to 38	49	.115	13	16.0	44.7	0	114,183	48	2	

Notes: 1. (same as table 7.)
2. Refer to table 3 for definitions.

Actual footing settlement was measured at each footing as described in chapter 2. The settlements attributed to compression of the sand (total settlement minus measured settlement of underlying compressible strata, if applicable) are shown on figure 25. In some cases, measurements were taken at multiple plan locations on the footings. In these cases, the settlement of the footing (for comparison with predicted values) was assumed to be the average of the values measured at the different locations. The measurements were taken after construction of the footings, and the weight of the footings was disregarded in the settlement calculations.

The results of the calculations are given in table 9. Calculated and measured settlements are compared graphically in figures 26A through 26F.

The ratios of calculated to measured settlement ("ratio") are listed in table 10 and plotted in figure 27. Values of calculated minus measured settlement ("difference") are shown in table 11 and figure 28. The standard deviation of these parameters is also shown on the tables.

The distribution of the difference values is illustrated in figures 29A through 29F. The combined distribution for all methods are approximately normal in shape, as was observed for measurements on other structures reported by Burland and Burbidge (1984).

Figure 30 shows the results of settlement calculations for footings on fill, using assumed N-values and using the measured N-values. The results using the assumed N-values are of comparable accuracy to those using measured N-values.

Figure 31 illustrates the results of using Schmertmann's method with actual CPT data compared to converting SPT data to CPT values. The calculations using the converted SPT overpredicted settlements to a greater extent than calculations using actual CPT data.

The following can be observed from the data in the tables and figures:

- o The measured settlements were small, exceeding one inch at only one footing. In the single case of settlement greater than one inch (2.3 in at abutment 1 at bridge no. 6), construction dewatering problems in silty sand soils are known to have disturbed the footing subgrade. Similar construction problems are believed to have increased the settlements of two footings which settled almost one inch (pier 3 at bridge no. 3). The mean settlement of the 21 footings was 0.49 in.

- o The mean values of the ratio of calculated to measured settlement ranged from 0.75 (Peck and Bazaraa) to 1.90 (Hough).

- o The average error of calculation with all field data, using the "ratio" as the indicator, ranged from 1 (D'Appolonia) to 90 percent (Hough).

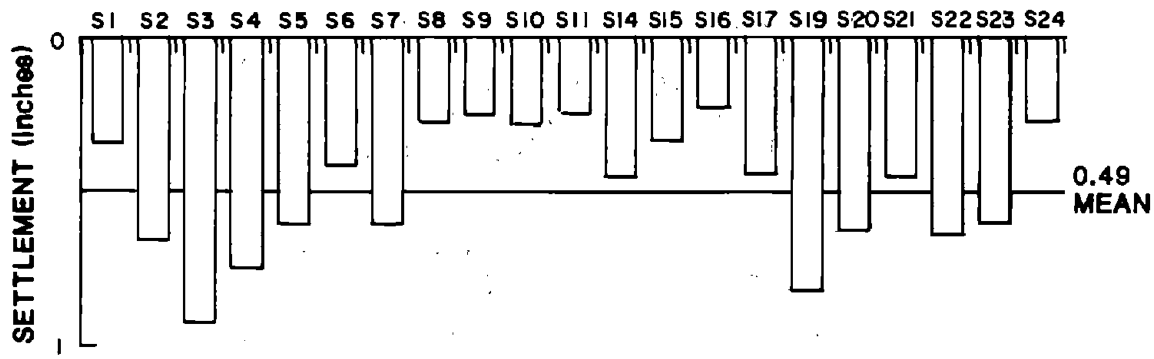


Figure 25. Settlements of study bridge elements used to compare settlement calculation methods.

Table 9. Calculated versus measured settlements - study bridges.

ELEMENT DESIGNATION	CALCULATED SETTLEMENT (in) Burland a Burbidge	D'Appolonia	Hough	Peck and Bazaraa	Peck and Bazaraa w/ Ladd	Schmertmann	MEASURED SETTLEMENTS (in)
S1	0.30	0.65	0.75	0.29	0.43	0.79	0.35
S2	0.12	0.39	0.94	0.16	0.16	1.85	0.67
S3	0.13	0.30	1.21	0.19	0.28	0.86	0.94
S4	0.39	0.58	1.46	0.36	0.53	0.46	0.76
S5	0.57	0.38	0.98	0.42	0.61	0.30	0.61
S6	0.34	0.50	0.61	0.17	0.24	0.52	0.42
S7	0.19	0.19	0.40	0.30	0.45	0.18	0.61
S8	0.14	0.26	0.60	0.16	0.24	0.30	0.28
S9	0.11	0.20	0.53	0.16	0.24	0.18	0.26
S10	0.09	0.23	0.40	0.16	0.24	0.29	0.29
S11	0.06	0.29	0.47	0.16	0.24	0.36	0.25
S12							0.97
S13							0.98
S14	0.40	0.57	1.27	0.50	0.70	0.41	0.46
S15	1.61	0.74	1.46	1.36	5.35	1.57	0.34
S16	0.17	0.39	0.74	0.17	0.25	0.26	0.23
S17	0.23	0.46	0.82	0.28	0.41	0.40	0.44
S18							2.26
S19	0.65	0.10	0.33	0.07	0.10	0.04	0.83
S20	0.54	0.49	1.05	0.21	0.32	1.21	0.64
S21	0.31	0.56	0.84	0.52	0.78	0.29	0.46
S22	0.64	0.61	1.39	0.34	0.51	0.54	0.66
S23	0.44	0.59	0.99	0.33	0.49	1.02	0.61
S24	0.36	0.36	0.61	0.25	0.37	0.64	0.28

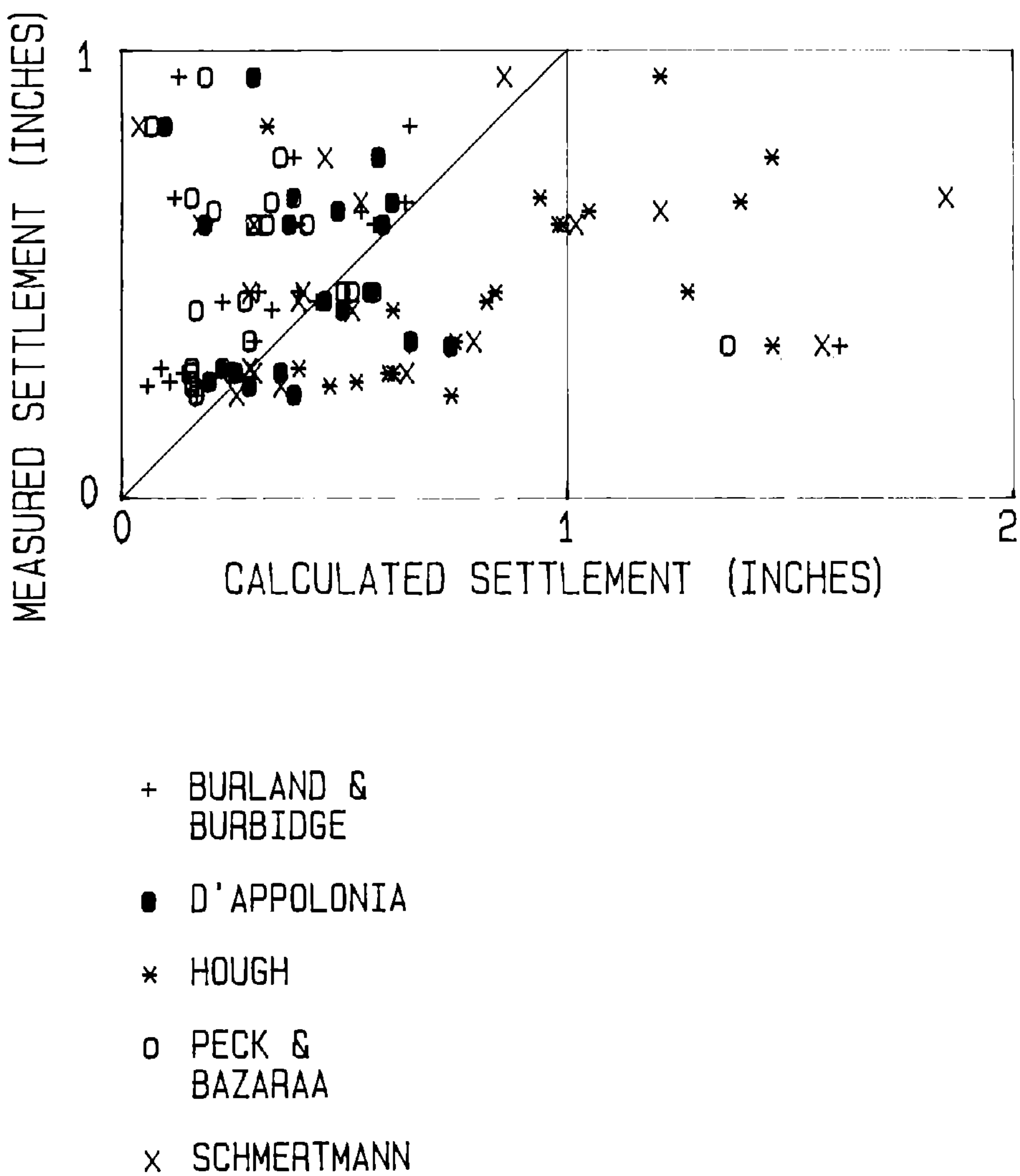


Figure 26. Calculated versus measured settlements - study bridges.

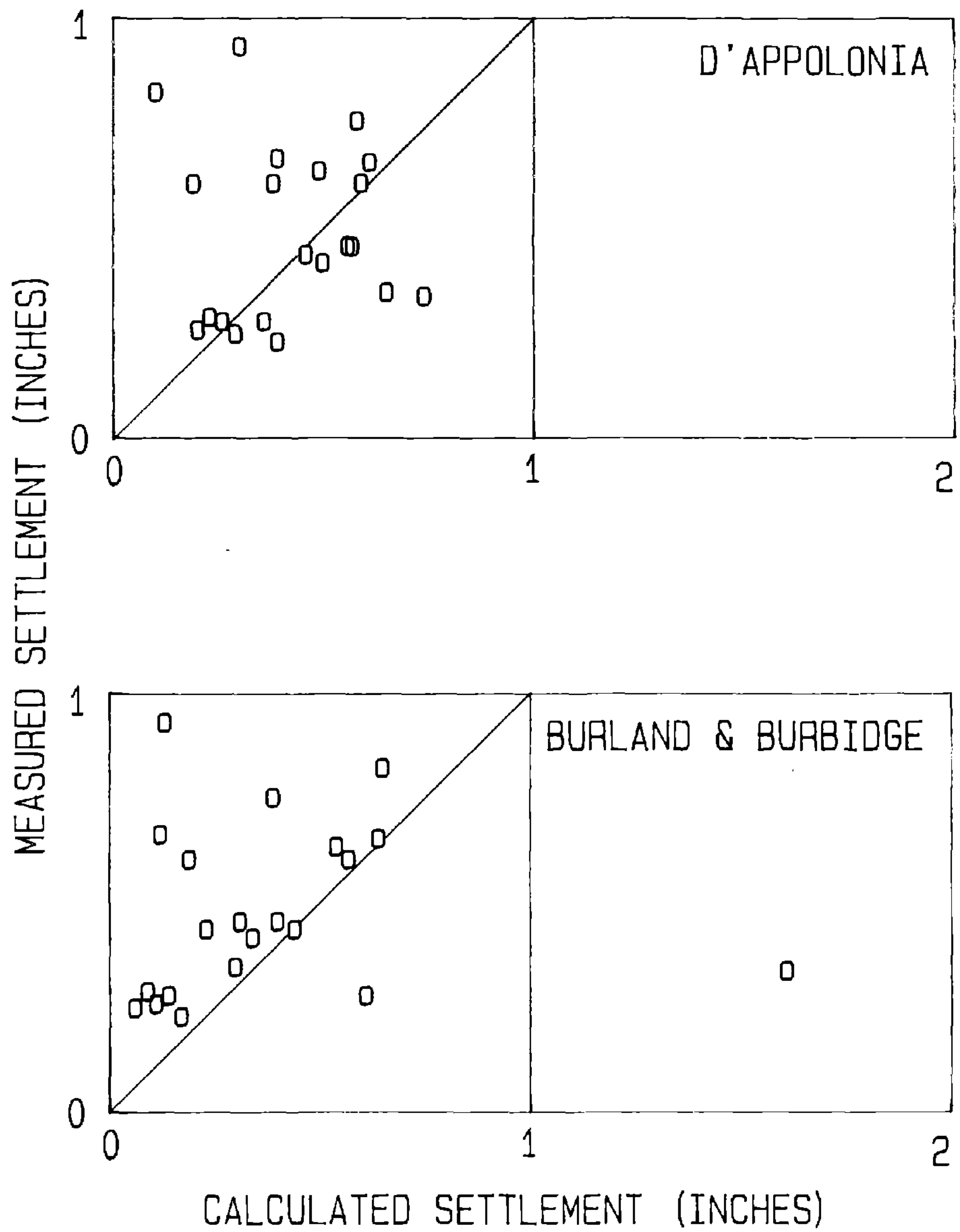


Figure 26. Calculated versus measured settlements - study bridges (Continued.)

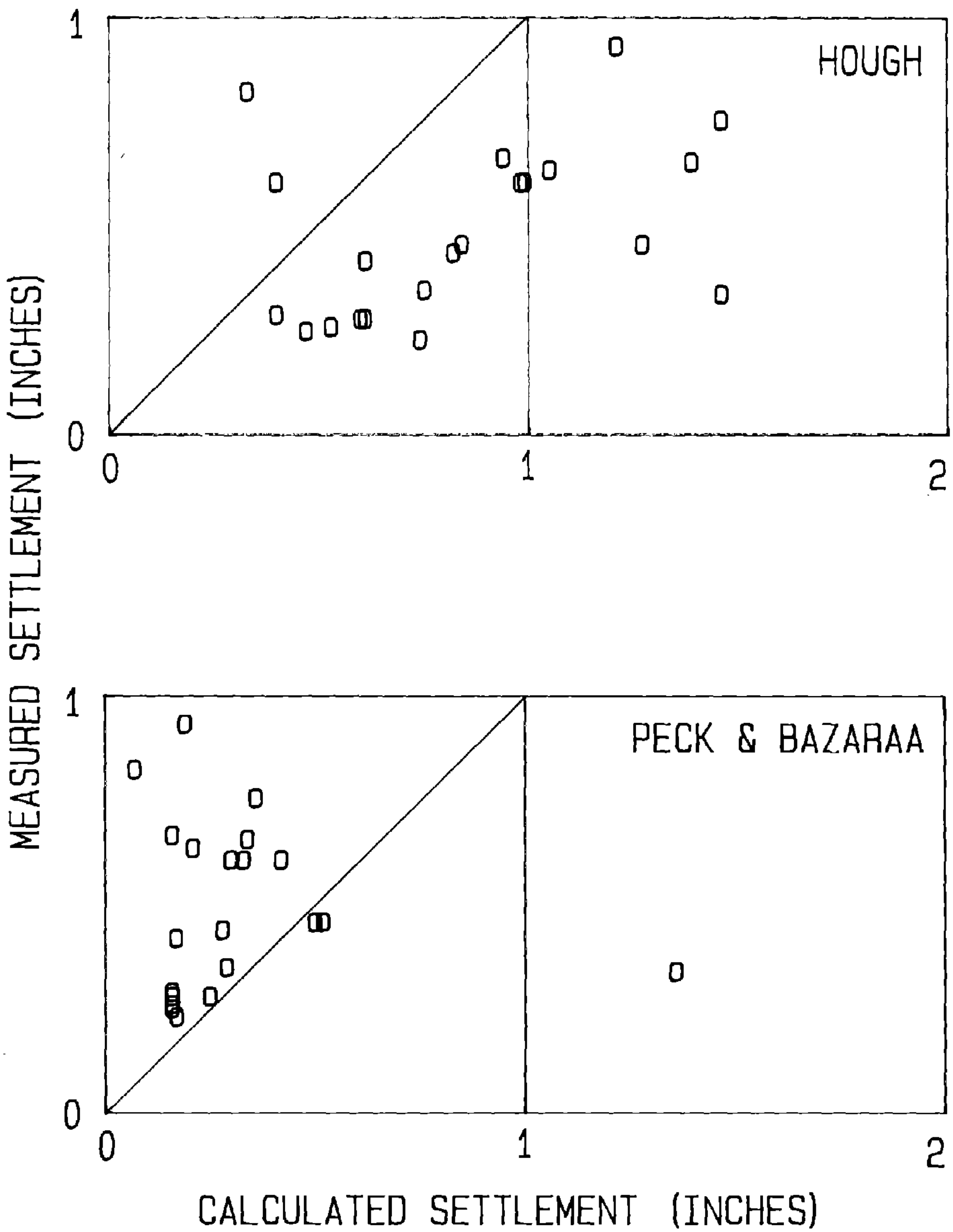


Figure 26. (Continued.)

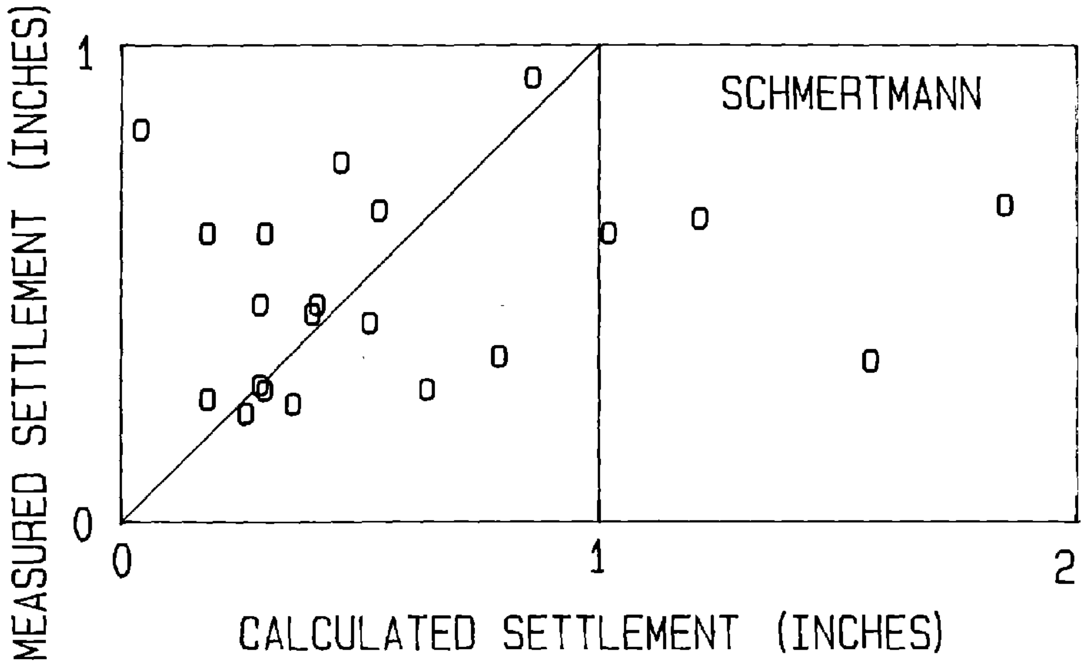


Figure 26. Calculated versus measured settlements - study bridges (Continued.)

- The mean of the absolute values of the "difference" for each method ranged from 0.20 in (D'Appolonia) to 0.42 in (Hough). This indicates that for the study bridge footings each of the selected methods was able, on average, to predict settlement within about 0.4 in. The difference exceeded one inch in only five calculated cases (out of a total 5x21=105 calculated cases) and in all five cases the calculated settlement exceeded the measured value, a conservative error.

- The D'Appolonia and Burland and Burbidge methods produced the smallest mean absolute differences, but underpredicted settlements by an average of 1 and 17 percent, respectively. The D'Appolonia method also produced the smallest standard deviation of the ratio of calculated to measured settlement (0.51).

  The Hough method produced the largest average ratio (1.90), and had the largest mean absolute difference (0.42).

- The Schmertmann method produced the largest standard deviation of the ratio of calculated to measured settlement (1.04).

Table 10. Ratio of calculated to measured settlements - study bridges.

ELEMENT DESIG-NATION	CALCULATED/MEASURED					
	Burland and Burbidge	D'Appolonia	Hough	Peck and Bazaraa	Peck and Bazaraa w/ Ladd(1)	Schmertmann
S1	0.86	1.86	2.14	0.83	1.23	2.26
S2	0.18	0.58	1.40	0.24	0.24	2.76
S3	0.14	0.32	1.29	0.20	0.30	0.91
S4	0.51	0.76	1.92	0.47	0.70	0.61
S5	0.93	0.62	1.61	0.69	1.00	0.49
S6	0.81	1.19	1.45	0.40	0.57	1.24
S7	0.31	0.31	0.66	0.49	0.74	0.30
S8	0.50	0.93	2.14	0.57	0.86	1.07
S9	0.42	0.77	2.04	0.62	0.92	0.69
S10	0.31	0.79	1.38	0.55	0.83	1.00
S11	0.24	1.16	1.88	0.64	0.96	1.44
S14	0.87	1.24	2.76	1.09	1.52	0.89
S15	4.74	2.18	4.29	4.00	15.74	4.62
S16	0.74	1.70	3.22	0.74	1.09	1.13
S17	0.52	1.05	1.86	0.64	0.93	0.91
S19	0.78	0.12	0.40	0.08	0.12	0.05
S20	0.84	0.77	1.64	0.33	0.50	1.89
S21	0.67	1.22	1.83	1.13	1.70	0.63
S22	0.97	0.92	2.11	0.52	0.77	2.33
S23	0.72	0.97	1.62	0.54	0.80	1.67
S24	1.29	1.29	2.18	0.89	1.32	2.29
Mean	0.83	0.99	1.90	0.75	1.56	1.39
Standard Deviation	0.94	0.51	0.82	0.79	3.27	1.04

Note: 1. Correction to footing size scaling factor is proposed by Ladd (1984). Refer to text.

Table 11. Calculated minus measured settlements - study bridges.

ELEMENT DESIG-NATION	CALCULATED-MEASURED ("Difference") Burland and Burbidge	D'Appolonia	Hough	Peck and Bazaraa	Peck and Bazaraa w/ Ladd(1)	Schmertmann
S1	-0.05	0.30	0.40	-0.06	0.08	0.44
S2	-0.55	-0.28	0.27	-0.51	-0.51	1.18
S3	-0.81	-0.64	0.27	-0.75	-0.66	-0.08
S4	-0.37	-0.18	0.70	-0.40	-0.23	-0.30
S5	-0.04	-0.23	0.37	-0.19	0	-0.31
S6	-0.08	0.08	0.19	-0.25	-0.18	0.10
S7	-0.42	-0.42	-0.21	-0.31	-0.16	-0.43
S8	-0.14	-0.02	0.32	-0.12	-0.04	0.02
S9	-0.15	-0.06	0.27	-0.10	-0.02	-0.08
S10	-0.20	-0.06	0.11	-0.13	-0.05	0
S11	-0.19	0.04	0.22	-0.09	-0.01	0.11
S14	-0.06	0.11	0.81	0.04	0.24	-0.05
S15	1.27	0.40	1.12	1.02	5.01	1.23
S16	-0.06	0.16	0.51	-0.06	0.02	0.03
S17	-0.21	0.02	0.38	-0.16	-0.03	-0.04
S19	-0.18	-0.73	-0.50	-0.76	-0.73	-0.79
S20	-0.10	-0.15	0.41	-0.43	-0.32	0.57
S21	-0.15	0.10	0.38	0.06	0.32	-0.17
S22	-0.02	-0.05	0.73	-0.32	-0.15	-0.12
S23	-0.17	-0.02	0.38	-0.28	-0.12	0.41
S24	0.08	0.08	0.33	-0.03	0.09	0.36
Mean	-0.12	-0.07	0.36	-0.18	0.12	0.10
Standard Deviation	0.38	0.28	0.34	0.36	1.15	0.48
Using Absolute Values of "Difference":						
Mean	0.25	0.20	0.42	0.29	0.43	0.32
Standard Deviation	0.30	0.20	0.24	0.27	1.07	0.36

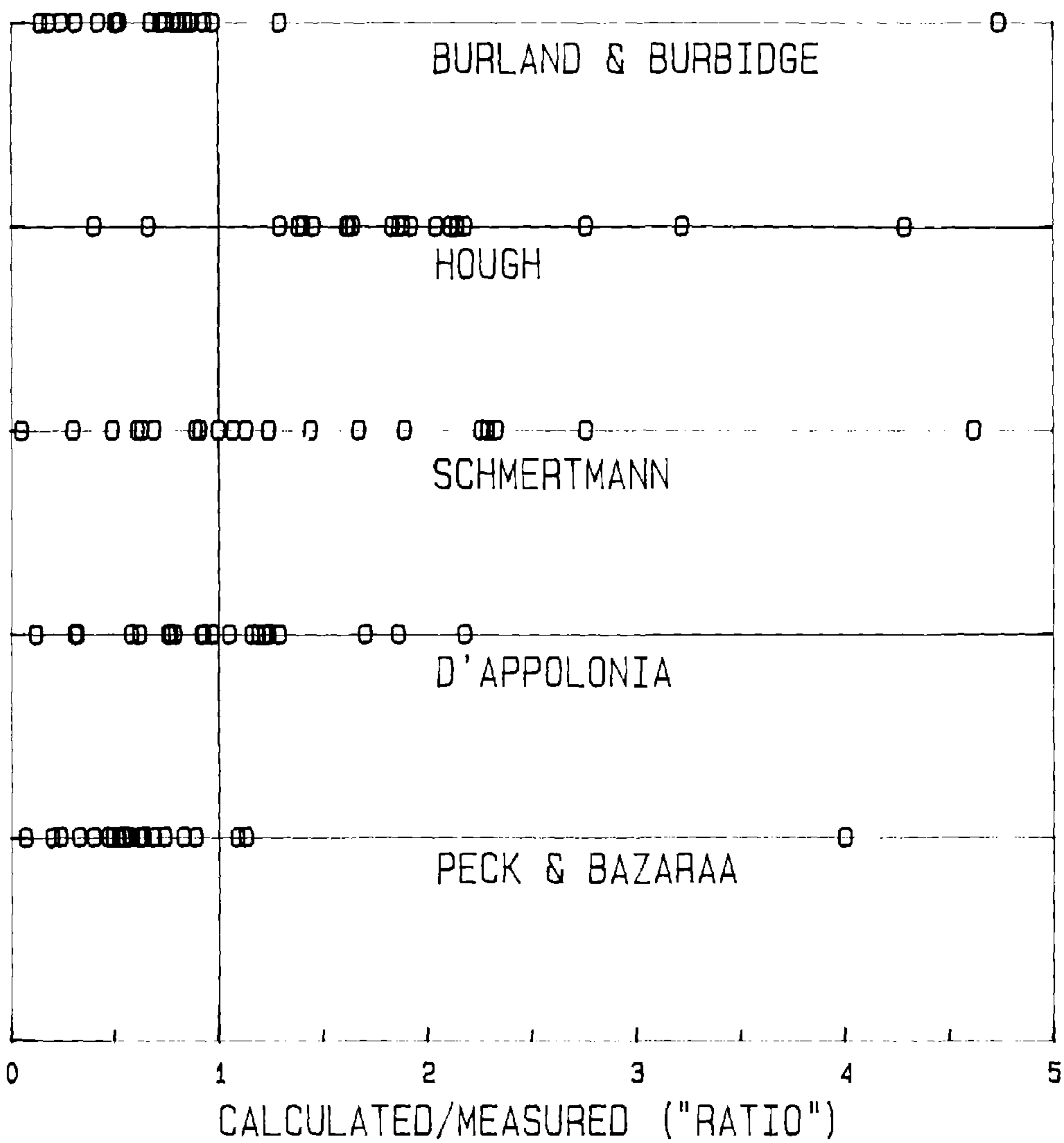


Figure 27. Ratio of calculated / measured settlements for all footings - study bridges.

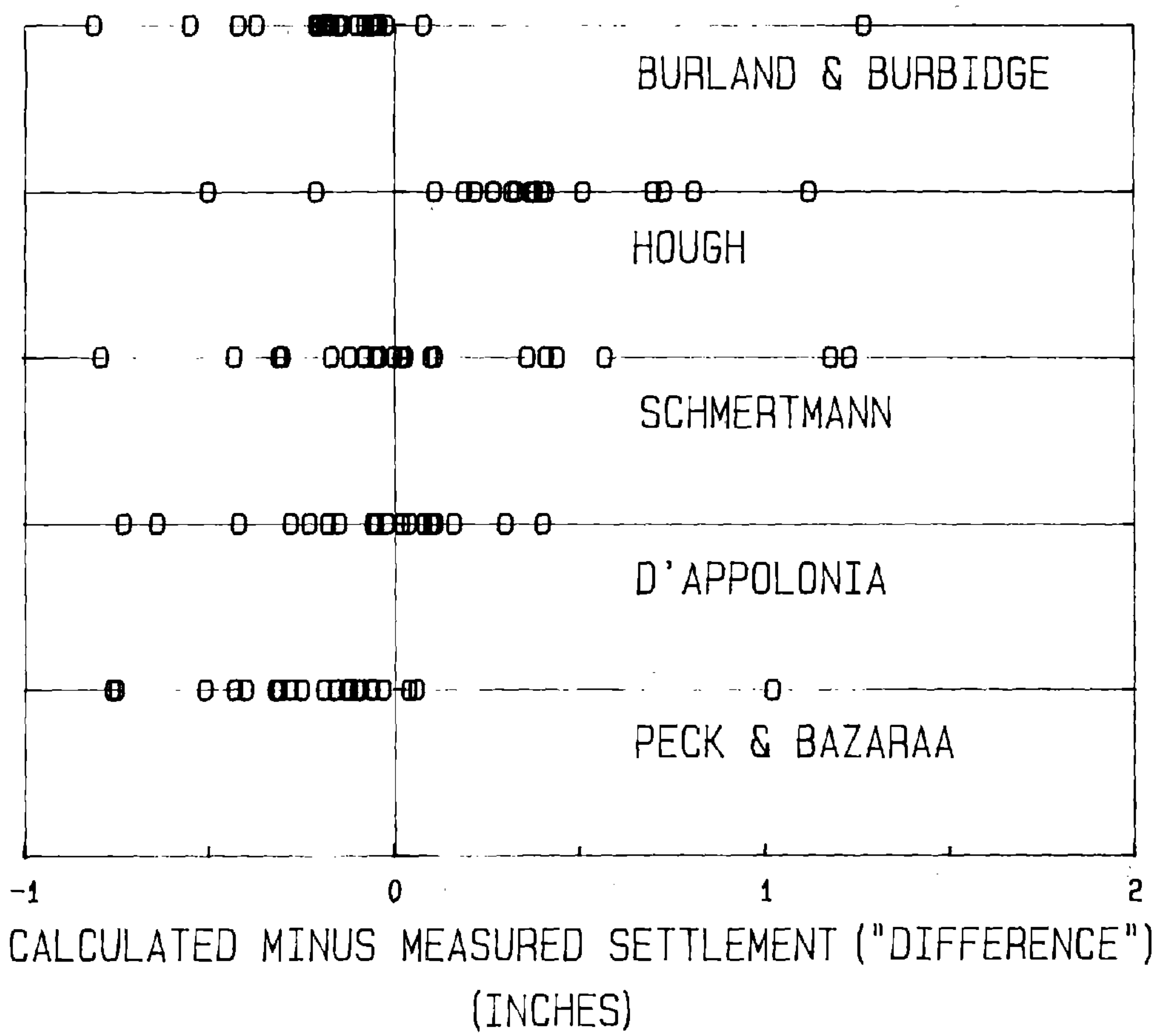


Figure 28. Calculated minus measured settlements - study bridges.

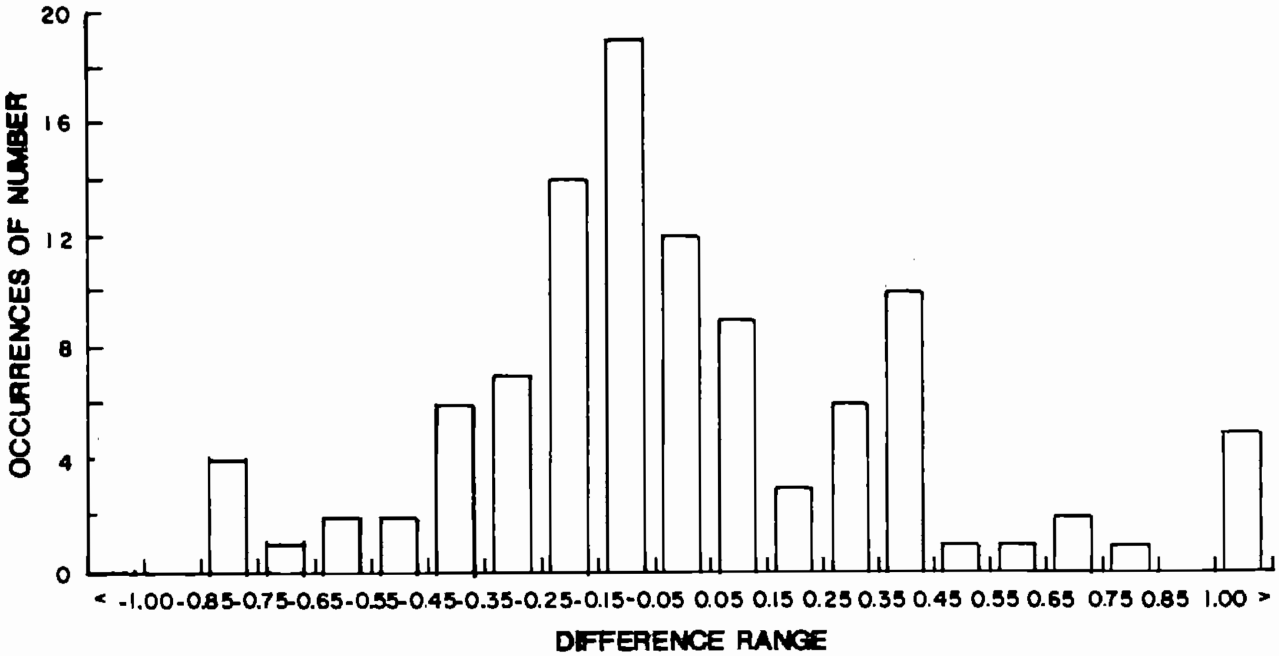


Figure 29. Distribution of "Difference" (calculated minus measured settlements).

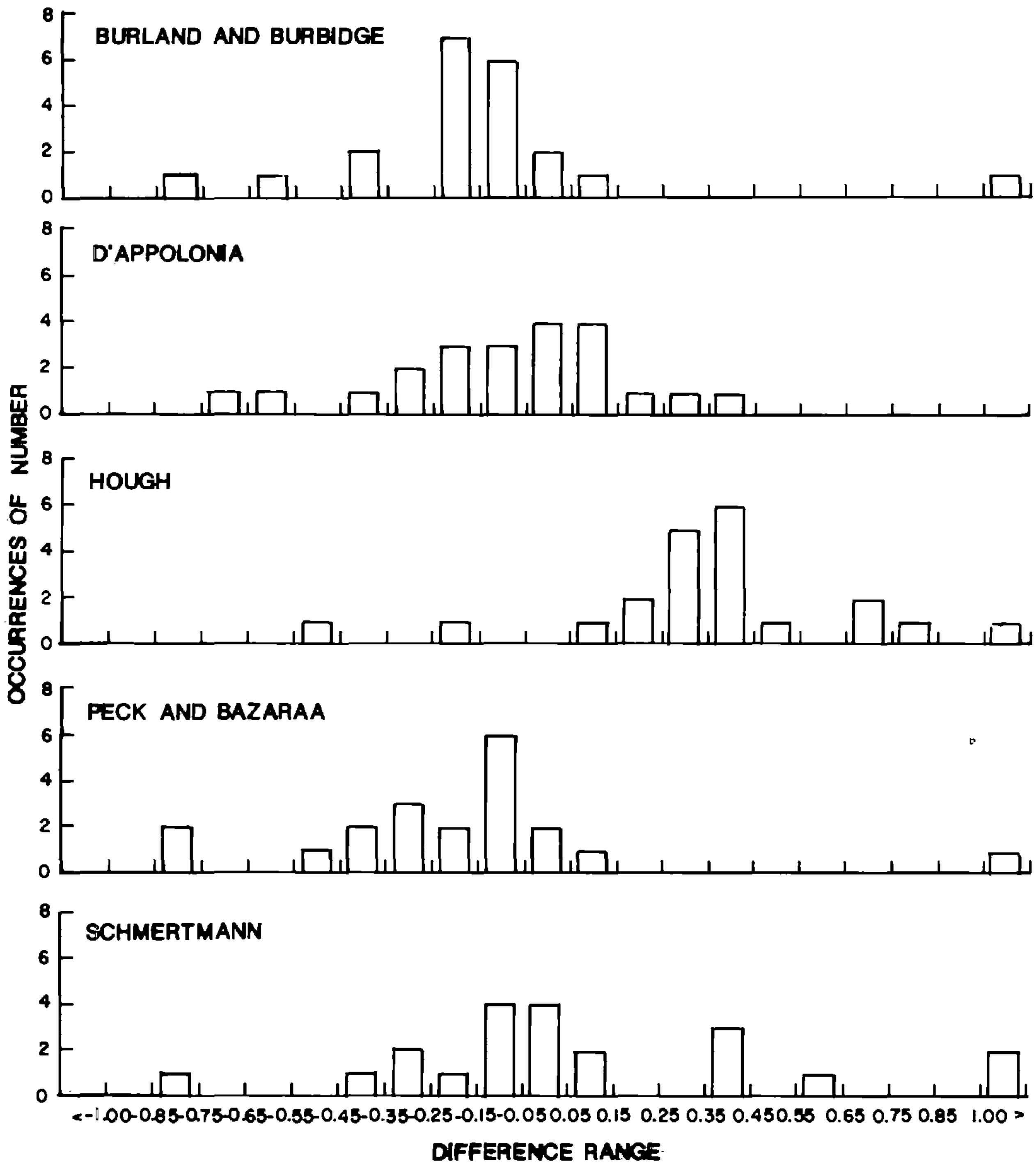


Figure 29. Distribution of "Difference" (calculated minus measured settlements)(continued.)

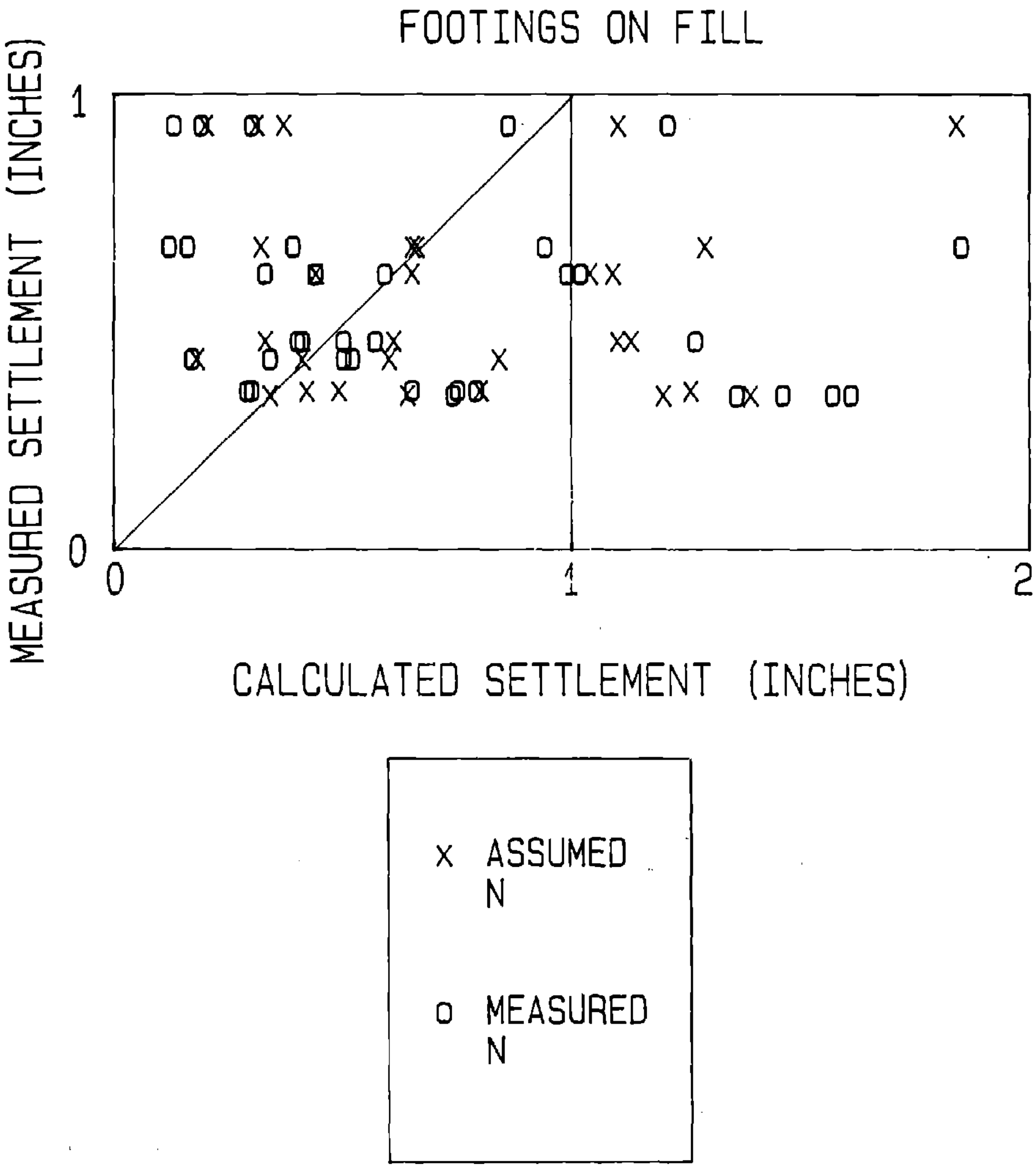


Figure 30. Calculated versus measured settlements - study bridges on fill.

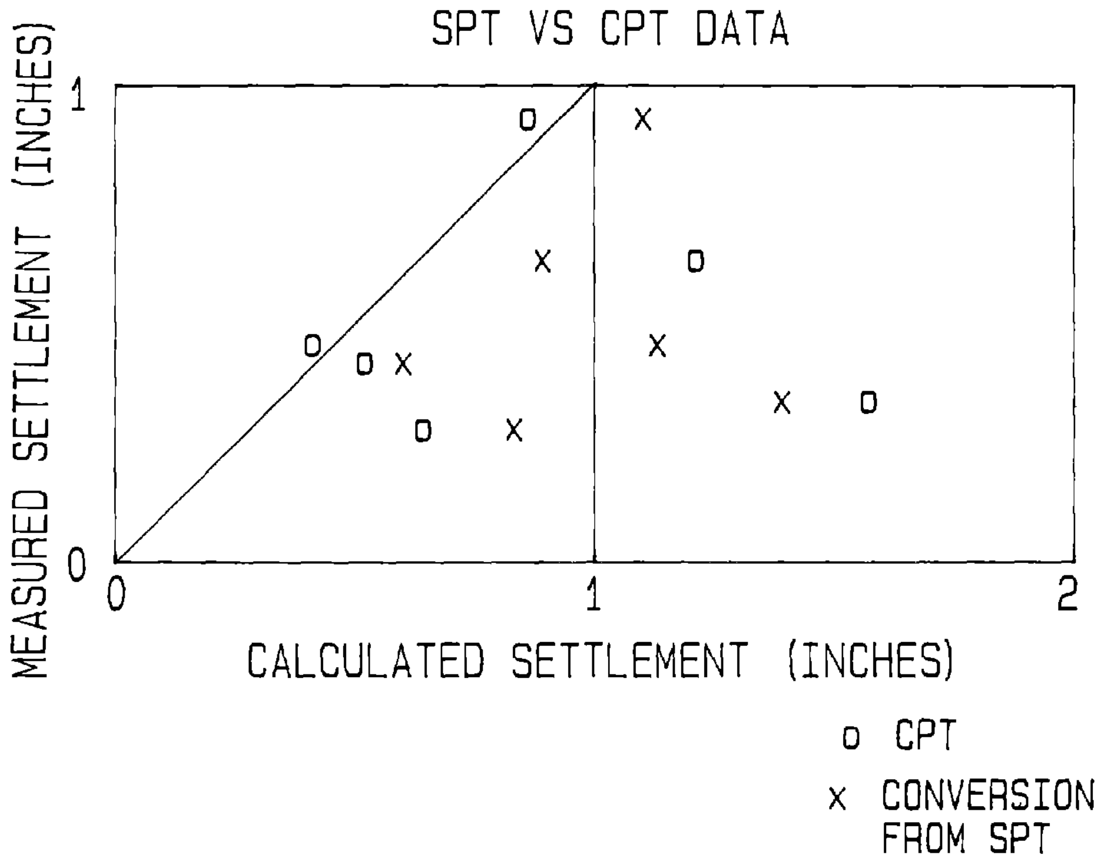


Figure 31. Calculation results for Schmertmann Method - CPT versus SPT data.

3.5 CONCLUSIONS

General

As is the case for buildings and most other structures, footings can be a viable alternative to piles for bridge foundations, both technically and economically, at sites underlain by sand soils. Most bridges can be successfully supported on footing foundations, at sites having medium dense to dense sand bearing soils without underlying compressible soils (clay, plastic silt, etc.). This conclusion is supported by the successful performance of the study bridges and many other bridges throughout the United States.

This study has compared the calculated and measured settlements for 21 bridge footings, using five settlement calculation methods. Each of the bridges was constructed and put into service, and has performed very well, during the study period. Measured total footing settlements of the study bridge footings were typically less than one inch. At three footings where the soil subgrade was disturbed by construction activities, settlements of 0.97, 0.98 and 2.26 in were measured. Based on final total settlement measurements, differential settlements between adjacent bridge elements ranged from 0.0 to 1.43 in. Only in one case (bridge no. 6) did the differential exceed 0.7 in. These data are given in table 12.

However, it is important to recognize the time rate of settlement in relation to construction of the bridges. Settlement which occurs prior to construction of the decks will not adversely affect the bridge unless the magnitude is so great that the vertical alignment of the piers or abutments is impaired. For continuous span bridges, only post-deck settlement can cause bending moments and stresses in the structural frame. Therefore analysis of predicted settlement and potential effects on the bridge should account for the relative timing of the settlement versus the construction stage of the bridge.

Most of the measured settlement of the study bridges occurred prior to construction of the bridge decks. This is demonstrated in figure 14, which shows that post-deck settlement of the study footings typically ranged from 0.02 to 0.35 in, while one pier footing which had the soil subgrade disturbed during construction settled 0.85 in. As indicated in table 12, post-deck differential settlements were always less than 0.7 in, and typically less than 0.2 in.

In a study for the FHWA, Moulton et. al. (1982) demonstrated that bridges can tolerate differential settlements and still perform well. Based on field studies of 314 bridges as well as theoretical analyses, they concluded the following:

- Angular distortions (differential settlement divided by span length) of 0.004 and 0.005 could be tolerated for continuous and simply-supported bridges, respectively. Using these criteria, a continuous bridge having a span length of 50 ft can tolerate a differential settlement of 2.4 in between supports.

- Settlements of those bridges that performed acceptably averaged 2.0 in.

Measured angular distortions for the instrumented bridge footings are also given in table 12. The measured distortions are well within the criteria described by Moulton and typically are smaller than Moulton's criteria by a factor of 10.

Conclusions

The five settlement calculation methods differ in their basic approach, assumptions, complexity and ease of use. The accuracy of the calculations for each method for the study bridge footings, as measured by their average absolute error, is summarized in table 13. Any interpretations of the results of the study should be made recognizing that the number of footings studied, as well as the range of bearing pressures, footing geometries and other conditions, are statistically small. Considering these limitations,

Table 12. Settlements at study bridges.

BRIDGE NO.	STRUCTURAL ELEMENT	MEASURED SETTLEMENT (in)	SPAN (ft)	DIFFERENTIAL SETTLEMENT (in)	DISTORTION $\Delta\rho/l$*
001	Abut 1	0.35 (0.20)			
			140	0.32 (0.15)	0.00019
	Abut 2	0.67 (0.35)			
002	Abut 1	0.94 (0.20)			
			110	0.33 (0.09)	0.00025
	Ctr. Pier	0.61 (0.11)			
			110	0.15 (0.07)	0.00011
	Abut 2	0.76 (0.18)			
003	West Abut	0.42 (0.15)			
			135	0.15 (0.15)	0.00009
	Pier 1	0.27(ave.)(0.0)			
			120	0.0 (0.14)	0.0
	Pier 2	0.27(ave.)(0.14)			
			125	0.70 (0.66)	0.00047
	Pier 3	0.97(ave.)(0.80)			
			125	0.36 (0.54)	0.00024
	East Abut	0.61 (0.26)			
004	South Abut	0.46 (0.0)			
			195	0.12 (0.15)	0.00005
	North Abut	0.34 (0.15)			
005	North Abut	0.23 (0.10)			
			135	0.21 (0.10)	0.00013
	South Abut	0.44 (0.20)			
006	Abut 1	2.26 (0.32)			
			110	1.43 (0.05)	0.00108
	Abut 2	0.83 (0.27)			
009	Abut 1	0.61 (0.01)			
			140	0.33 (0.01)	0.00020
	Abut 2	0.28 (0.0)			

( ) indicates data taken after deck was constructed.

* Where $\Delta\rho$ = differential settlement, in.
l = span length, in.

Table 13. Accuracy of calculation methods.

METHOD	AVERAGE ABSOLUTE DIFFERENCE (in)	STANDARD DEVIATION OF AVERAGE ABSOLUTE DIFFERENCE (in)
Burland and Burbidge	0.35	0.39
D'Appolonia	0.33	0.47
Hough	0.50	0.36
Peck and Bazaraa	0.41	0.47
Schmertmann	0.44	0.52
Using the average of all methods for each footing	0.29	0.45

the following conclusions related to the calculation methods are offered, based on the results of the study:

- On average, the selected methods when used separately can predict footing settlement within about 0.4 in, on average. Larger error can occur for individual calculations.
- Based on the ratio of calculated/ measured settlements, three of the five methods (Burland and Burbidge, D'Appolonia, and Peck and Bazaraa) typically underpredicted settlement, while the other two (Hough and Schmertmann) typically overpredicted.
- The mean of the "ratio" for all 105 calculations is 1.17 and the mean absolute difference for all the calculations is 0.30.
- The D'Appolonia method was the most accurate, on average, with Burland and Burbidge next. The Hough method provided the least accurate predictions.
- If the results of each method are averaged on a footing by footing basis (as though all five methods were averaged for each footing), the mean absolute difference for all 105 calculations is 0.30. This mean compares to the range of means of 0.20 to 0.42 calculated separately for each method (table 11).

- The accuracy of each of the methods is similar to the accuracy of other analysis techniques used in geotechnical engineering, including methods used to predict pile foundation settlement.

Based on experience, case history results and the performance of the instrumented bridges, it can be expected that the total settlement of properly-designed and constructed footings on medium dense or dense sand would typically be on the order of one inch or less due to compression of the sand, and post-deck settlements of bridges in such cases would be expected to be even smaller.

Recommendations

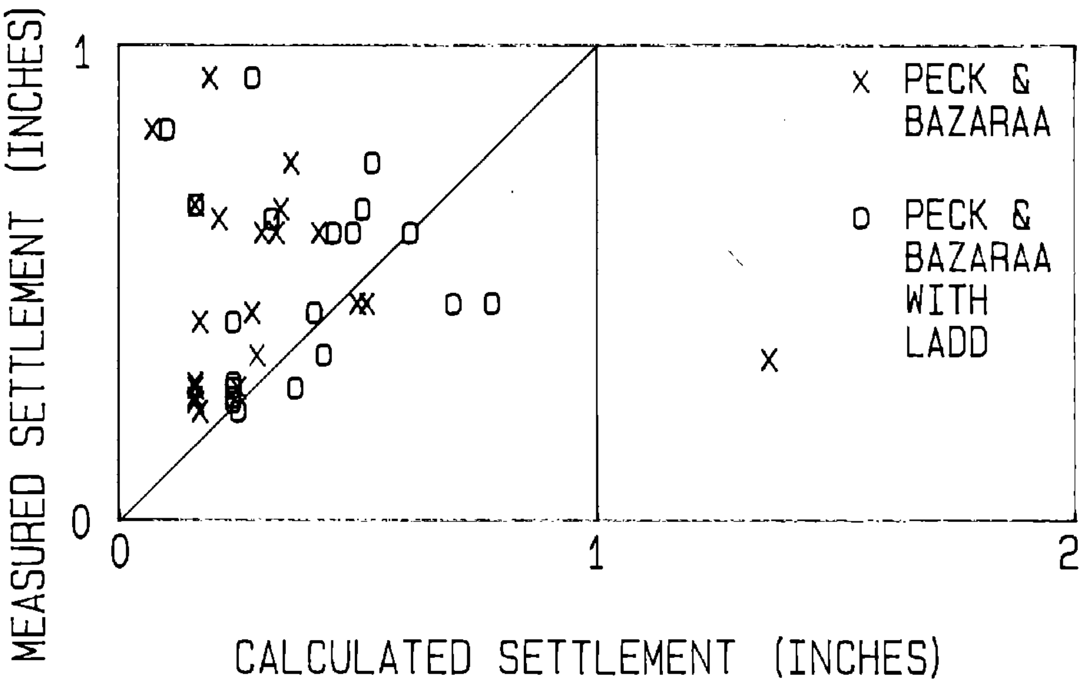
In general, it is believed that the selected settlement calculation methods are sufficiently accurate for use in design of footing foundations for bridges. The methods should be used with an understanding of their limitations and assumptions. Design of footing foundations should be performed by engineers knowledgeable of geotechnical engineering principles.

The methods provide a means to calculate settlement under a given set of conditions, which existed in the cases used to develop the methods. Many details of performing such calculations in actual design situations, such as how to handle soil preloading or interaction between adjacent footings, have not been explicitly explained by the developers of most of the methods. Many such details are addressed by two calculation examples given in appendix E.

Based on the observations made during this study, it is recommended that the settlement calculation procedures be used in the manner illustrated by the calculation examples in appendix E. The procedures outlined in the examples and herein represent the consultant's interpretation of appropriate use of the methods.

For use in practice, calculated settlements could be increased by a factor to reduce the likelihood that actual settlements might exceed the calculated values. The magnitude of the factor would depend on the calculation method and the desired reduction of "risk". As can be seen on figure 27, if a factor of 1.3 was multiplied times the calculated settlements using Burland and Burbidge, all but one of the measured settlement would have been less than the factored calculated value. Factors could be applied in a similar manner to the other methods. However, considering the relatively small magnitude of settlements measured at the study bridges, it would seem overly conservative to design footings based on calculated values which are arbitrarily increased in this way.

One possible modification to the Peck and Bazaraa method involves revised scaling factors for footing size to account for soil relative density. Ladd (1984) has proposed the modified factors shown on figure 32 to account for increased scale effects for looser sands. Calculations using the modified factors are summarized in tables 10 and 11, and on figure 32. The results indicate an increase in the mean calculated/ measured ratio (0.75 to 1.56),



Range of Corrected Blow Count*	Peck and Bazaraa Scale Factor	Ladd Scale Factor
$N_c<10$	$(\frac{2B}{B+1})^2$	$(\frac{5B}{4+B})^2$
$10<N_c<50$	same as above	$(\frac{2.5B}{B+1.5})^2$
$N_c>50$	same as above	$(\frac{2B}{B+1})^2$

* N_c is N corrected for overburden as per Bazaraa 1967.

Figure 32. Peck and Bazaraa Method results with Ladd Modification.

an increase in the mean absolute "difference" (0.29 to 0.43) and an increase in the standard deviation (0.79 to 3.27). However, one footing greatly influenced the calculation (S15), possibly because the footing is bearing on fill above relatively loose soil. The fill possibly limited settlement to a much smaller value than was predicted by the calculations which were influenced by low N-values in the soil below the fill. If S15 is not considered, the mean "ratio" is 0.85 and the mean absolute "difference" is 0.20, both significantly improved over the results without the correction.

Based on these results, it appears that in general Ladd's correction improves settlement predictions of the Peck and Bazaraa method. Subject to verification with a greater number of measurements, use of the modification can be recommended.

Because the five methods account for the important parameters in different ways, it appears appropriate to use two or more of the methods for design of any particular bridge footing foundation. The magnitude of anticipated settlement should be selected based on the range of the calculated values, the level of confidence in the quality of the subsurface data and experience with similar structures under similar conditions.

Adequate data on soil stratigraphy, compressibility, preloading and variability must be obtained and interpreted. If possible, subsurface explorations should be performed at each potential footing location to obtain soil samples, groundwater levels, and SPT and/or CPT values. It is recommended that the testing in the explorations be concentrated on the depth range from the bottom of the footing to a depth equal to the width of the footing. In this zone, continuous SPT and/or CPT data with depth will provide a greater data base on which to estimate soil compressibility.

4. RISK-BASED METHODS IN GEOTECHNICAL DESIGN

4.1 INTRODUCTION

This section briefly summarizes the current state of the art of risk-based methods for the design of shallow footings. More complete discussion is presented in a companion report:

- o "Geotechnical Risk Analysis: The State of the Art," FHWA Technical Report No. 87/010, 1987.
- o "Geotechnical Risk Analysis: A User's Guide," FHWA Technical Report No. 87/011, 1987.

The second of these reports is a step-by-step procedures manual for using risk-based methods in geotechnical design.

4.2 STATE OF THE ART OF RISK-BASED GEOTECHNICAL DESIGN

Probabilistic and statistical methods in geotechnical engineering are of several distinct forms, having different purposes. As a matter of convenience, they may be divided into four groups by the methods they use and the questions they answer:

- Probabilistic techniques,
- Statistical methods,
- Risk assessment, and
- Economic optimization (decision analysis).

Probabilistic Techniques and Reliability

Probability theory is a mathematical theory which can be used to characterize uncertainties about engineering parameters and to describe the relations among such uncertainties. Probability theory is used in geotechnical engineering to translate uncertainties about engineering parameters or variables through engineering models to draw conclusions about the uncertainty in the predictions of those models. For example, given information about the uncertainty in soil conditions, probability theory can be used to calculate the uncertainty in bearing capacity predictions calculated by Terzaghi's formula.

The application of probabilistic models to geotechnical systems to assess safety is called reliability analysis. A reliability analysis replaces conventional safety indices such as the factor of safety FS with indices based on probability. The most common probabilistic index is the "probability of failure" p_f. This is the area under an estimated probability distribution of facility performance lying, for example, beneath FS=1. Other common probabilistic indices are based just on averages and standard deviations of predicted performance. The most important of these is the first-order second-moment reliability index, β, described below.

Statistical Methods

Statistical methods are a set of techniques for drawing inferences from observations. These methods use probability theory as a means for describing variability and uncertainty, but they are more an ad hoc collection of methods than a consistent theory.

Statistical methods are used in geotechnical engineering primarily to analyze data on site conditions and environmental loads. To some extent they are applied to validating model predictions against observed performance. The intent of statistical analysis in geotechnical applications is to make efficient use of data and to provide the probabilistic characterization of uncertainty necessary for reliability modeling or risk analysis. Increasingly, statistical methods are also being used to plan efficient "scientific" sampling plans for gathering information or validating models.

Risk Assessment

Risk assessment in its meaning here is the effort to bring all relevant uncertainties together in an analysis to assess the aggregate uncertainty facing a designer. That is, it combines a number of probabilistic analyses each addressing different modes of performance, expert opinion, statistical analyses, and so on in an attempt to comprehensively analyze a proposed design. A proper risk assessment leads to predictions of rates of failure and a quantification of the uncertainty in the rates.

To date, the use of risk assessment in geotechnical engineering has been limited and often proprietary, for example, in evaluating risks for insurance underwriting. An increasing area of use is in regulatory licensing and evaluation of siting for hazardous facilities. It appears likely that risk analysis will also become more widespread in the design of dams and other large projects.

Decision Making and Optimization

Optimization of design or project decisions by balancing risk against cost requires not only risk assessment but also an analysis of the costs of failure. In many cases such failure costs involve only economic attributes, but in others they also involve nonmonetary costs: life loss, environmental degradation, social disruption. Decision analysis and optimization attempt to quantify the consequences of facility failures, combine these quantifications with assessments of their associated probabilities, and identify design options that balance conservatism against the cost and likelihood of failure.

In geotechnical engineering, decision analysis approaches have been often discussed, but seldom implemented in a comprehensive way. Applications have tended to emphasize either careful assessment of consequences or careful assessment of probabilities, but seldom both. The better applications of decision analysis in geotechnical engineering for the most part have dealt with regulatory problems, such as power plant siting, in which the principal uncertainties and concerns do not deal with soil or rock mechanics problems.

4.3 GEOTECHNICAL RISK-ANALYSIS METHODOLOGY

The purpose of risk-based design is to improve but not fundamentally change traditional practice. Specifically, the improvements involve two things,

- the selection of design parameters and
- the economic rationalization of design.

In the first area, conservative soil properties are replaced with best estimates and measures of uncertainty. This provides a repeatable criterion for choosing parameters, and allows uncertainties in more than one parameter to be combined. In the second area, factors of safety are replaced with measures of confidence. This provides an explicit statement of uncertainty, and allows design cost to be balanced against performance.

The term "risk-based design" as used here is not what many now call "probabilistic design." In "probabilistic design," as that term is commonly used, one attempts to predict actual rates of failure to be observed in the field. Geotechnical engineering, however, involves many uncertainties only some of which are explicit. Therefore, probabilities resulting from analysis are not predictions of rates of failure in the field. Studies of failures show that the majority are attributable to unanticipated loads, gross errors, inadequate maintenance, and other factors that are not analyzed. Uncertainty in an analytical prediction, as for example factor of safety, has to do with the chance that, if the proper analysis had been made and if the proper parameter values had been chosen, then the calculated prediction of performance would have shown the design to be inadequate (i.e., the correctly calculated FS would have been less than 1.0). Risk-based design is an attempt to identify uncertainties in performance predictions and quantify their effect.

Describing Uncertainty

Evaluations of soil properties are conveniently expressed by a best estimate and a measure of uncertainty. The mean and standard deviation, respectively, are used to express these two attributes. The mean of a set of measurements $x=[x_1,\ldots,x_n]$ is the arithmetic average,

$$m_x = \frac{1}{n}\sum x_i = \text{mean.} \qquad \text{(Eq. 2)}$$

The standard deviation is the square root of the moment of inertia of the data about their mean,

$$s_x = \frac{1}{n-1}\sum (x_i - m_x)^2 = \text{standard deviation.} \qquad \text{(Eq. 3)}$$

The square of the standard deviation (i.e., the moment of inertia) is called the variance. The ratio of the standard deviation to the mean, or proportional uncertainty, is called the coefficient of variation,

$$\Omega_x = s_x/m_x = \text{coefficient of variation.} \qquad \text{(Eq. 4)}$$

In dealing with two or more soil properties, not only may the individual means and standard deviations be important but also the association among different properties. The strength of such association is measured by the covariance, or cross-product moment,

$$C_{x,y} = \frac{1}{n-2}\sum (x_i - m_x)(y_i - m_y) = \text{covariance.} \qquad \text{(Eq. 5)}$$

Components of Uncertainty

Uncertainty in a soil property x is expressed quantitatively by the standard deviation s_x. Four contributing factors must be considered in assessing the magnitude of the standard deviation,

1. Spatial variability of the soil deposit.
2. Random measurement error or noise.
3. Measurement bias.
4. Statistical error due to limited testing.

These are related to one another as shown in figure 33. Spatial variability is real differences in soil properties from one location to another at too detailed a scale to be accounted for in engineering calculations. Measurement noise is random variability in measurements caused by operator or instrumental effects. Together spatial variability and measurement noise combine to produce data scatter.

Measurement bias is a consistent error in measured soil properties caused, for example, by sample disturbance or the assumptions used in analyzing test data. Statistical error is the estimation inaccuracy caused by limited numbers of measurements and variations among test results. Together, measurement bias and statistical error constitute systematic error in test data. The distinction between data scatter and systematic error is shown in figure 34.

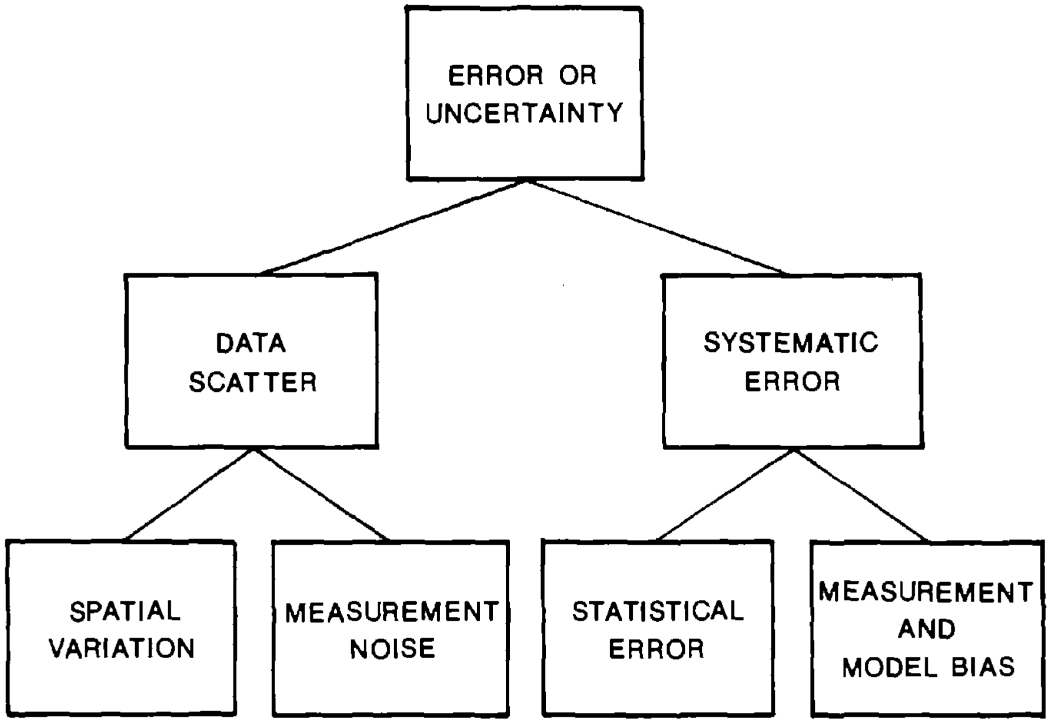


Figure 33. Sources of error or uncertainty in soil property estimates.

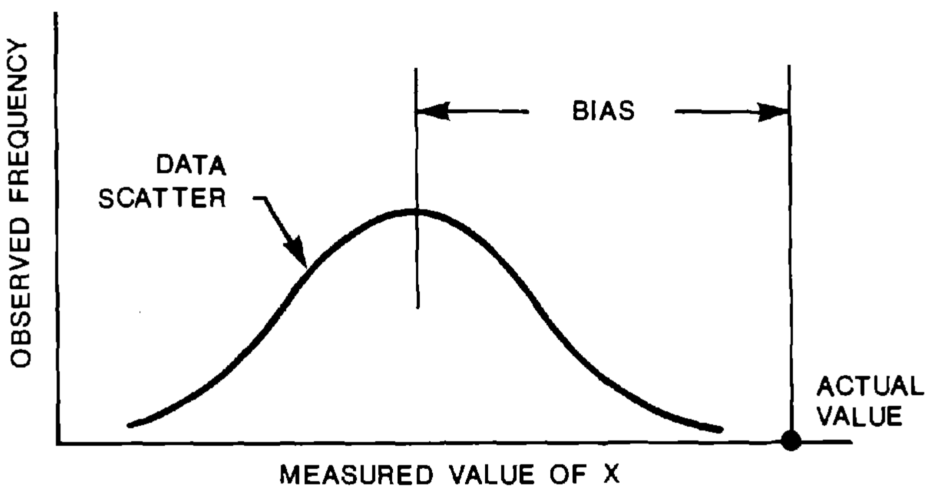


Figure 34. Distinction between data scatter and bias error.

In practical applications, separating the four components of uncertainty is important for two reasons. First, each of the four components arise differently from the others and is translated differently through an engineering calculation. Second, the four components affect engineering predictions in different ways and need to be accommodated differently from one another in site investigations and in design. For example, spatial variation of soil properties leads to variation among footing settlements at the same site. Systematic errors lead to a difference between predicted settlement and the average settlement realized in the field. Measurement noise merely increases the imprecision with which footing settlements can be predicted and cost-effective effort can usually be invested in removing such noise from data.

The statistical procedures discussed in the report, "Geotechnical Risk Analysis: A User's Guide," can be used to quantify the contribution of each source of uncertainty in soil property estimates. With simplifying assumptions these procedures lead to an expression,

$$s_x^2 = s_{1_x}^2 + s_{2_x}^2 + s_{3_x}^2 + s_{4_x}^2 \qquad \text{(Eq. 6)}$$

for combining the four sources of uncertainty, in which s_x = standard deviation of the estimated soil property x at any arbitrary location defined as follows,

$$\begin{aligned} s_{1_x} &= \text{Standard deviation due to spatial variation,} \\ s_{2_x} &= \text{Standard deviation due to measurement noise,} \\ s_{3_x} &= \text{Standard deviation due to measurement bias,} \\ s_{4_x} &= \text{Standard deviation due to statistical error.} \end{aligned} \qquad \text{(Eq. 7)}$$

Uncertainty in Engineering Calculations

The best estimate and uncertainty of a soil property x are translated through an engineering calculation to obtain a corresponding best estimate and standard deviation on a performance prediction y. For example, x might be Standard Penetration Test blow count and y might be the settlement of a shallow footing on sand. The best estimate (mean) and uncertainty (standard deviation) of x (SPT blow count) are translated through the engineering model used to calculate y (footing settlement) to obtain a mean and standard deviation of y.

Operationally, the mean and standard deviation of x are translated through a calculation by using a linear approximation. A performance variable y is predicted from a soil property x using an engineering model g(x),

$$y = g(x) \qquad \text{(Eq. 8)}$$

The model g(x) is approximated by its tangent at the mean of x (figure 35). Probability theory then yields the approximate results

$$m_y \approx g(m_x) \qquad \text{(Eq. 9)}$$

$$s_y^2 \approx \left(\frac{dy}{dx}\right)^2 s_x^2 \qquad \text{(Eq. 10)}$$

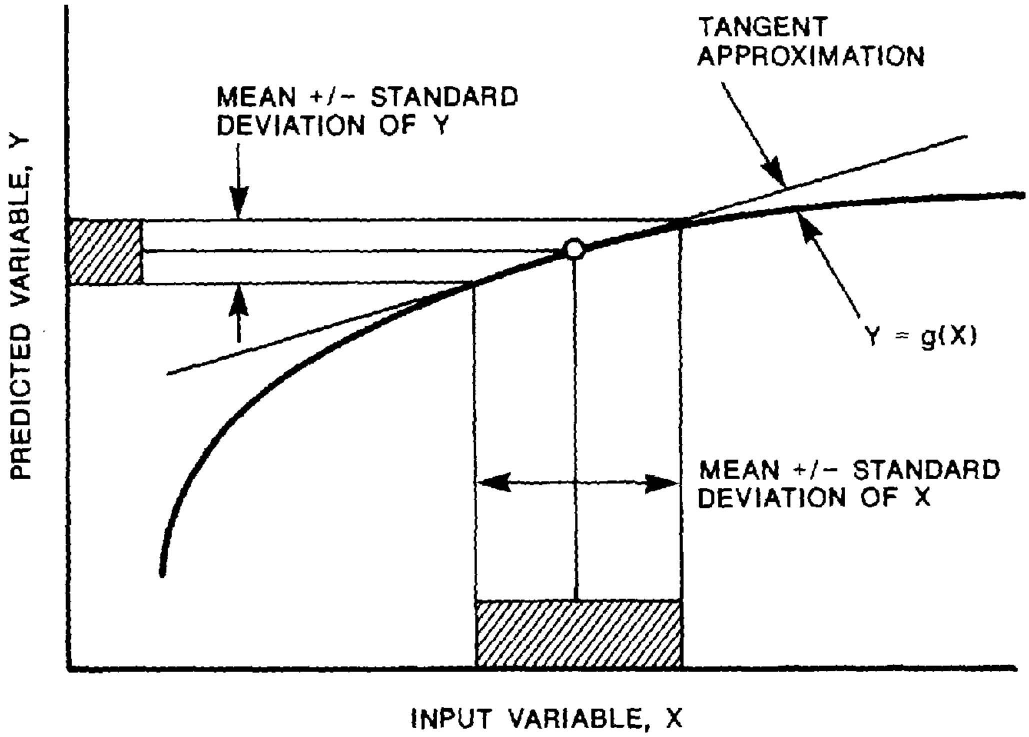


Figure 35. Variability function approximated by tangent.

In words, the mean of y is calculated by using the mean of x as input to the model. This is the common engineering solution using the best estimate of x as input. The variance (standard deviation squared) of y is calculated from the variance of x by multiplying $s_x{}^2$ by an influence factor equal to the square of the derivative of y with respect to x. When y depends on more than one soil property, $\underline{x} = [x_1, \ldots x_n]$, the corresponding forms of equations 9 and 10 are

$$m_y \approx g(m_{x1}, \ldots, m_{xn}) \qquad \text{(Eq. 11)}$$

$$s_y^2 \approx \sum_i \sum_j \left(\frac{dy}{dx_i}\right)\left(\frac{dy}{dx_j}\right) C_{x_i,x_j} \qquad \text{(Eq. 12)}$$

Note that, when x_i and x_j are independent, their covariance equals zero. Since the covariance of an estimate with itself equals the variance, when all the soil property uncertainties are independent equation 12 reduces to

$$s_y^2 = \sum_i \left(\frac{dy}{dx_i}\right)^2 s_{x_i}^2 \qquad \text{(Eq. 13)}$$

In risk-based design the individual component uncertainties on x due to spatial variability (s_1^2), measurement noise (s_2^2), measurement bias (s_3^2), and statistical error (s_4^2) are individually translated via equation 10 through the calculation $y = g(x)$ to find corresponding component uncertainties on y. This is shown schematically in figure 36.

The individual component uncertainties on x are separately translated through the calculation $y = g(x)$ because they each have a different effect on the corresponding uncertainty of y. For example, the systematic errors summarized in s_3^2 and s_4^2 translate directly through a calculation. If a calculated result y depends proportionately on x, then the relative error in x due to s_3^2 and s_4^2 produces the same relative error in y. On the other hand, the influence of spatial variability s_1^2 does not translate directly through an engineering calculation; it is affected by scale. The larger the volume of soil mobilized, in general, the more the spatial variability averages out. A size effect factor R is introduced to equation 10, to accommodate this scale dependence,

$$s_{1_y}^2 = R \left(\frac{dy}{dx}\right)^2 s_{1_x}^2 \qquad \text{(Eq. 14)}$$

For most geotechnical problems $0<R<1.0$, and can be read from graphs or tables. The derivation of R is beyond the scope of this summary, but is considered in the User's Guide.

Unlike spatial variability and systematic error, measurement noise is primarily a nuisance. To the extent possible one would like simply to filter it out of an analysis. It affects the precision with which soil properties can be estimated, but it has nothing to do with the true variability of soil properties. Thus, if the magnitude of measurement noise can be assessed, it can be removed to lessen the magnitude of data scatter. Experience has shown that measurement noise often contributes 50 percent or more of the variance of data scatter. Thus, statistically removing noise can substantially decrease the uncertainty of a performance prediction. Statistical techniques for assessing the magnitudes of measurement noise in data are also discussed in "Geotechnical Risk Analysis: A User's Guide."

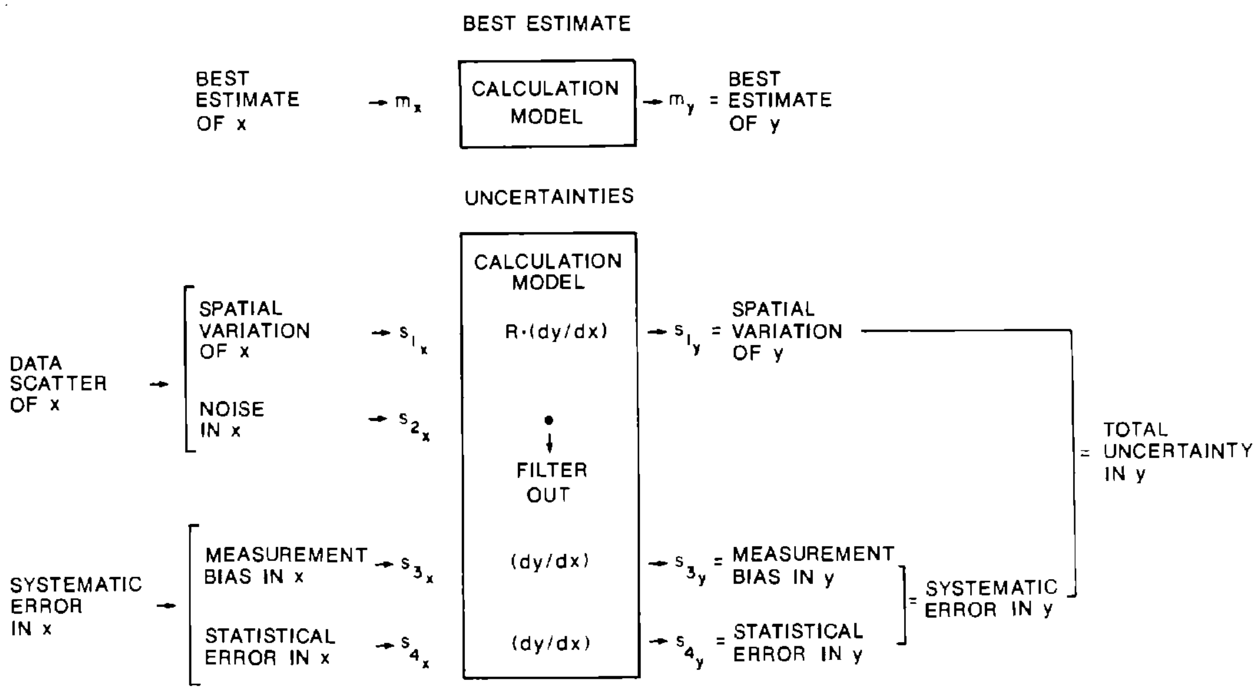


Figure 36. Schematic of error propagation through model.

Reliability Index

In assessing reliability, both the mean prediction and its standard deviation play a role. Reliability is inversely related to the probability that the performance y actually realized in service fails to meet some specified value y_0. The value y_0 might be a failure condition, a standard, or so forth. The mean m_y by itself is insufficient to judge reliability, as can be seen in figure 37. The probability that y lies beneath y_0 equals the corresponding area under the probability distribution. As can be seen, a calculation which yields a high mean but also a high standard deviation (e.g., calculation 1) can lead to a lower reliability than another calculation (e.g., calculation 2) which has a lower mean but also a lower standard deviation.

To overcome this problem, the mean m_y and standard deviation s_y are combined in the reliability index

$$\beta = \frac{m_y - y_0}{s_y} \qquad \text{(Eq. 15)}$$

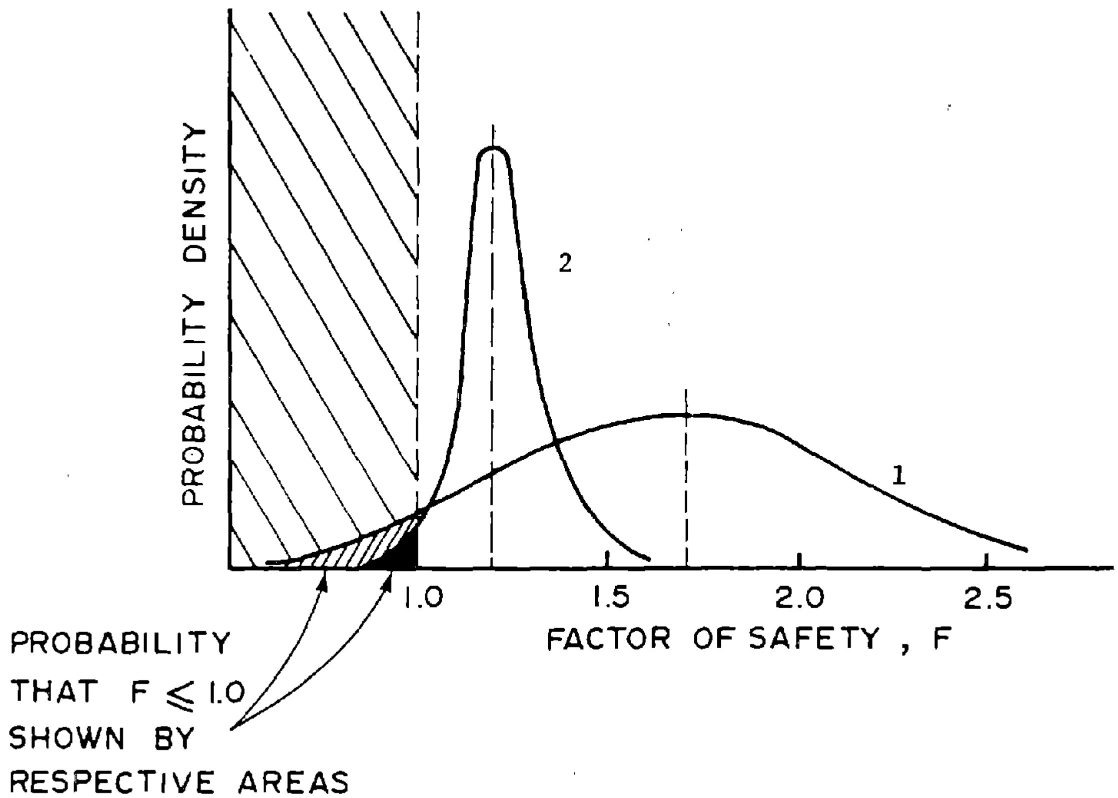


Figure 37. Comparison of factors of safety with different distribution patterns.

This index measures the number of standard deviations separating the best estimate of y from the nominal failure value y_0. The advantage of using beta as a measure of safety is that it captures the difference in reliability illustrated in figure 37.

4.4 RISK-BASED DESIGN OF SHALLOW FOUNDATIONS

In this section the procedure outlined above is applied to settlement calculations for the design of shallow footings on sand.

Site Conditions

The site is underlain by fine dry sand to a depth of 10m. Fifty SPT borings were made across the site and a limited number of laboratory tests were performed to correlate blow count with friction angle. The trend of depth-averaged blow counts was corrected by Gibbs and Holtz's method, and the autocovariance function used to estimate the contribution of noise to total data scatter, as described in "Geotechnical Risk Analysis: A User's Guide." For the upper levels of the profile which most strongly influence the settlement of shallow footings, the average blow count is 16.6 blows/ft (bpf) and the average corrected blow count is about constant with depth at a value of 25 (figure 38).

The standard deviation of the vertically averaged corrected blow count in the upper levels is about s_N=11bpf. Thus, the coefficient of variation is Ω_N=(11bpf/25bpf)=0.44. Using the technique to estimate noise developed in the Users' Guide suggests that noise contributes about 50 percent of this data scatter, measured in variances. Laboratory tests on specimens recompacted to the in situ relative density led to an average friction angle of 36.4°, and a standard deviation of 1.1°.

Best Estimate of Footing Settlement

The results of applying the methods described in this section to predictions of settlement and bearing capacity are shown in figure 38.

The footing is 10 ft wide and embedded 5 ft, with a design load of 3 TSF, as shown. Settlement is predicted by the Peck and Bazarra formula, ignoring correction factors for groundwater level, etc., as

$$\rho \approx \left(\frac{2\Delta q}{N}\right) \left(\frac{2b}{1+b}\right)^2 \left[1- \frac{1}{4} D/b\right] \qquad \text{(Eq. 16)}$$

in which,

$$\begin{aligned} \Delta q &= \text{applied stress (tons per square foot)} \\ b &= \text{footing width (feet)} \\ \rho &= \text{settlement (inches)} \\ D &= \text{embedment depth (feet)} \\ N &= \text{depth averaged blow count} \end{aligned} \qquad \text{(Eq. 17)}$$

The mean settlement m_ρ is found by substituting mean values of all the parameters in equation 16, in the same way that the deterministic solution would be obtained. In the present case, the only uncertain parameter is SPT blow count, N, for which m_N is substituted. The other parameters are assumed to have negligible uncertainty. Inserting m_N into equation 16 gives the best estimate of settlement, m_ρ = 0.7 inch.

Uncertainty of Settlement

The uncertainty in the settlement prediction is represented by the standard deviation s_ρ. This is calculated by propagating the four sources of uncertainty in the input parameter N through equation 16, using equation 10, and then recombining the output according to an equation of the form equation 6.

The first step in calculating s_ρ is assessing the magnitude of the four contributions to uncertainty in N: spatial variability, measurement noise, measurement bias, and statistical error. The first two appear as data

CALCULATION SHEET

PROBLEM: SPT data analysis CALCULATED BY: ____________

DATE: ____________ CHECKED BY: ____________

DESIGN PROFILE: SPT data in a clean, wind-deposited sand.

DATA SCATTER

Raw Data:

n = 50 measurements

m_N = 16.6 between elevations 590' and 610'

m_N^c = 25 bpf, corrected blow count approximately constant with depth for first 20'

s_N = 11 bpf (total data scatter of vertically averaged blow counts)

Measurement Noise (from autocorrelation analysis):

$s_{2_N} \approx$ 7.8 bpf

Spatial Variability:

$$s_{1_N}^2 = s_N^2 - s_{2_N}^2$$

$$\approx (11\text{bpf})^2 - (7.8\text{bpf})^2$$

$$\approx (7.8\text{ bpf})^2$$

SYSTEMATIC ERROR

Statistical Error $S_{3_N}^2$:

$$S_{m_N}^2 \approx S_N^2/n$$

$$\approx (11\text{bpf})^2/50$$

$$\approx (1.6\text{bpf})^2$$

Measurement Bias S_4^2:

<ignored>

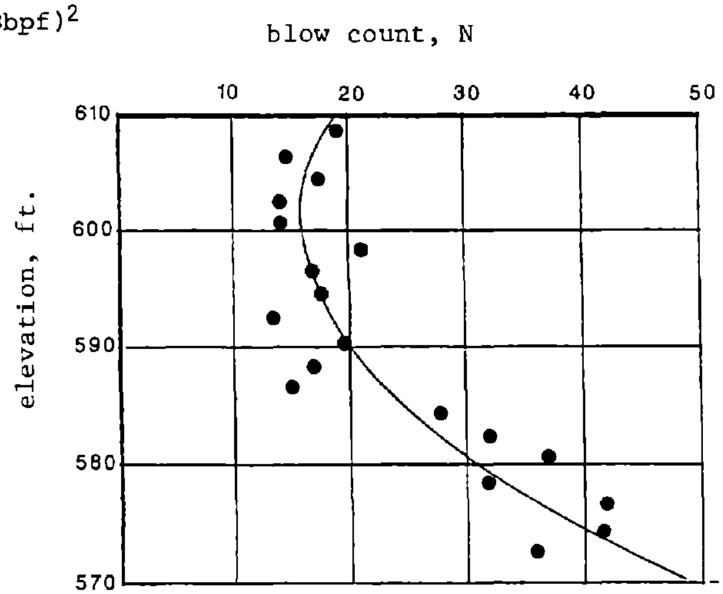


Figure 38. Sample calculation of error propagation.

CALCULATION SHEET

PROBLEM: footing settlement CALCULATED BY: GBB
DATE: CHECKED BY:

[(b) UNCERTAINTY OF SETTLEMENT con't]

Systematic Error (statistical only, model bias neglected)

n=50 borings thus the statistical error on the mean blow count at any elevation is,

$$s_{4_N} = \sqrt{\frac{(11\text{bpf})^2}{50}} = 1.6\text{bpf}$$

$$s_\rho^2 = (d\rho/dN)^2 \, s_N^2$$

$$= (1/m_N^2)^2 \left[(2\Delta q) \left(\frac{2b}{1+b}\right)^2 (1 - \tfrac{1}{4} D/b) \right]^2 s_N^2$$

$$= (1/25^2)^2 \left[(2.3) \left(\frac{2 \cdot 10}{1+10}\right)^2 [1 - \tfrac{1}{4} 5/10] \right]^2 (1.6)^2$$

$$= (0.04'')^2$$

(c) RELIABILITY INDEX

$$s_p^2 = (0.21^2 + 0.04^2) = 0.23''^2$$

$$\beta = \frac{m\rho - \rho_o}{s_\rho} = \frac{|0.70'' - 1''|}{0.23''} = \underline{\underline{1.3}}$$

(d) OBSERVED SETTLEMENT

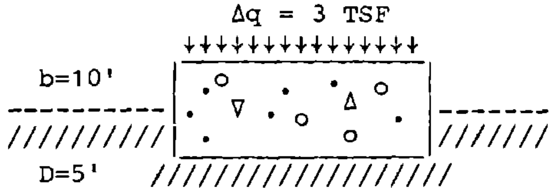
	Observed	Predicted
m_ρ =	0.35"	0.70"
S_ρ =	0.12"	0.23"
Ω_ρ =	0.34	0.33

Figure 38.(Continued.)

CALCULATION SHEET

PROBLEM: footing settlement ____________ CALCULATED BY: ____________
DATE: ____________ CHECKED BY: ____________

PROBLEM



allowable ρ = 1"

SOIL PROPERTIES

N = vertically averaged corrected SPT blow count

m_N = 25 bpf

s_N = 7.8 bpf real

s_N = 11 bpf data scatter

n = 50 measurements

(a) BEST ESTIMTE (MEAN) OF SETTLEMENT

$$m_\rho \doteq \left(\frac{2\Delta q}{m_N}\right)\left(\frac{2b}{1+b}\right)^2 [1-\frac{1}{4}\,D/b] \;=\; \left(\frac{2\cdot 3}{25}\right)\left(\frac{2\cdot 10}{1+10}\right)^2 [1-\frac{1}{4}\,5/10]$$

$$= \underline{\underline{0.70"}}$$

(b) UNCERTAINTY (STANDARD DEVIATION) OF SETTLEMENT

Spatial Variability

$$s_\rho^2 = (d\rho/dN)^2\, s_N^2$$

$$= (1/m_N^2)^2\, \left[\, (2\Delta q)\left(\frac{2b}{1+b}\right)^2 (1-\frac{1}{4}\,D/b)\, \right]^2 s_N^2$$

$$= (1/25^2)^2\, \left[\, (2.3)\left(\frac{2\cdot 10}{1+10}\right)^2 [1-\frac{1}{4}\,5/10]\, \right]^2 (7.8)^2$$

$$= (0.22")^2$$

Figure 38.(continued.)

scatter and must be separated from one another. The second two are systematic errors and can only be estimated by calculation (i.e., they do not appear in data scatter or in any other explicit form).

The scatter in SPT data for the site, by empirical observation, has a standard deviation of 11 bpf. Since about half the data scatter measured as a variance appears to be noise, the standard deviation of the spatial variability alone is

$$s_{1_N}^2 = s_N^2 - s_{2_N}^2 \qquad \text{(Eq. 18)}$$

$$\approx (11 \text{ bpf})^2 - (7.8 \text{ bpf})^2$$

$$\approx (7.8 \text{ bpf})^2$$

Based on an analysis of the structure of spatial variability at the site, the conclusion was reached that $R \approx 1.0$ (the analysis of this case is detailed in the User's Guide). As a result

$$s_\rho^2 = (d\rho/dN)^2 \, s_N^2 \qquad \text{(Eq. 19)}$$

$$= (1/m_N^2)^2 \left[(2\Delta q) \left(\frac{2b}{1+b}\right)^2 \left(1 - \frac{1}{4} D/b\right) \right]^2 s_N^2$$

$$= (1/25^2)^2 \left[(2.3) \left(\frac{2.10}{1+10}\right)^2 \left[1 - \frac{1}{4} 5/10\right] \right]^2 (7.8)^2$$

$$= (0.22'')^2$$

In the calculations of figure 38 the measurement bias s_{3_N} is ignored, because blow counts are measured directly rather than being inferred through a model or set of calculations. Thus, it is assumed no error of interpretation is introduced by the way measurements are analyzed.

As a general approximation, the statistical error in the mean value of any N, expressed as a variance, is approximately equal to the data scatter variance divided by the number of independent measurements, n. In the present case, there are n=50 blow count measurements at any depth, so the standard deviation of the statistical error is approximately

$$s_{4_N} \cong \frac{s_N^2}{n} = \frac{(11\text{bpf})^2}{50} = 1.6\text{bpf} \qquad \text{(Eq. 20)}$$

The total error in the settlement prediction is found by combining the sources of uncertainty according to an equation of the form of equation 4.4. The main causes of uncertainty are spatial variation and statistical error. Measurement noise has been statistically removed from the prediction, and measurement bias is ignored. This gives,

$$s_\rho^2 = s_{1_\rho}^2 + s_{4_\rho}^2 \qquad \text{(Eq. 21)}$$

Dividing both sides by $m_\rho{}^2$ gives the same expression in terms of coefficients of variation, often a more useful form:

$$\Omega_\rho{}^2 = \Omega_{1_\rho}^2 + \Omega_{4_\rho}^2 = 0.31^2 + 0.06^2 = 1.00 \qquad \text{(Eq. 22)}$$

$$\Omega_\rho = 0.33$$

The reliability index is calculated from equation 15 as,

$$\beta = \frac{m_\rho - \rho_0}{s_\rho} = \frac{0.70" - 1"}{(0.33)(0.7)} = 1.3 \qquad \text{(Eq. 23)}$$

The mean and standard deviation of the actually observed footing settlements is shown in figure 38. The mean settlement was about half that predicted, but the variability among footing settlements was close to the spatial variability predicted. The differences between mean predicted settlement and mean observed is due to two factors. In service, the footings were subject to less than the design loads, and the settlement model of equation 15 itself contains bias. This latter bias could be accounted for by regression analysis, and incorporated in the uncertainty analysis as an s_3 term.

4.5 CONCLUSIONS

The method described in this section allows a quantified assessment to be made of the degree of error in settlement calculations, based on the amount and scatter of data obtained and on the confidence one places in the type of tests used. This assessment can be used as a guide in deciding upon the level of exploration and testing that is economically beneficial. If combined with considerations of the financial consequences of excessive settlement, these assessments of error can be used as the basis for risk analysis.

5. DESIGN OF SPREAD FOOTINGS ON ROCK

5.1 INTRODUCTION

General

Usually rock is regarded as the best bearing material for structural foundations. Pile foundations through soft overburden soils are often driven to a refusal resistance. This refusal is often equated with rock. Thus, in the mind of the bridge designer, rock is a very desirable bearing material. There are instances, however, where rock can present problems such as in areas where sinkholes in limestone may make a footing foundation impractical or in other areas where swelling or air slaking of rock or decomposition of the rock exposed to frost or weathering may represent a problem. Normally, however, the bearing capacity of the rock in its undisturbed and protected state is greater than the adjacent soil materials.

Objectives and Scope

The objective was to produce "practical design" recommendations for spread footings on rock. These recommendations have been included in section 5.5. Practical design means that the design procedures should not require a detailed, analytical rock mechanics approach but should be something consistent with standard practice. The methods should more clearly highlight rock types with unusual engineering properties or rock types likely to have significant defects. The objectives include:

- Review of the current design procedures for various types of spread footings on rock. Refer to section 5.2.
- Identify the rock properties needed to complete the spread footing design on rock. Refer to sections 5.3 and 5.4.
- Prepare a recommended design procedure for spread footings on rock. For design recommendations refer to section 5.5.

5.2 REVIEW OF CURRENT DESIGN PROCEDURES

General

If the rock beneath a footing were free of defects, then the allowable bearing pressure could be taken, conservatively, as the average compressive strength of unconfined rock core samples. Real rock masses, however, are virtually never free of imperfections. There is usually one or more sets of fractures which divide the mass into blocks, as illustrated in figure 39. When load is applied, settlement may result from compression of individual blocks, slippage between blocks, volume decrease of materials that fill the spaces, and closing of open fractures. It should also be noted that blasting effects may create new fractures or at least cause existing fractures to open.

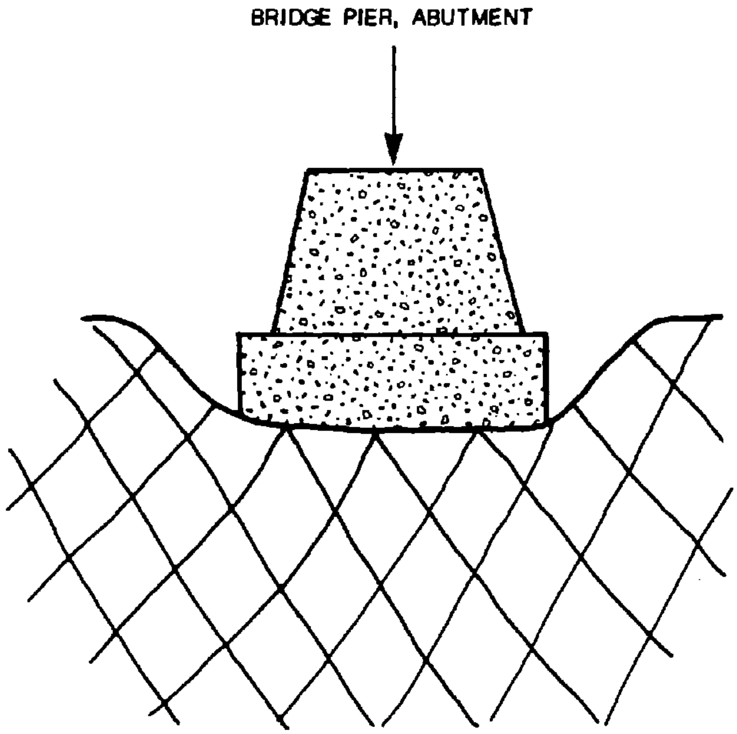


Figure 39. Typical footing on discontinuous rock mass.

The potential for settlement under applied foundation loads is nearly always the governing factor for design. Therefore, it is necessary to restrict the allowable bearing pressure to a value considerably less than the unconfined compressive strength.

Rational selection of an allowable rock bearing pressure should, therefore, be based primarily on the in situ compressibility of the rock mass and not the strength of intact rock core.

Bearing Capacity

Presumptive Bearing Values

In many localities, the allowable contact pressure for foundations on rock is specified by building codes. Typical values are shown in table 14.

Many codes are not designed to meet local conditions. The allowable bearing values are based only on highly generalized rock descriptions. There are significant differences in allowable pressures for rocks of the same general description.

Table 14. Presumptive bearing stresses (tsf) for foundations on rock (after Putnam, 1981).

CODE	YEAR	BEDROCK	SOUND FOLIATED ROCK	SOUND SEDIMENTARY ROCK	SOFT ROCK	SOFT SHALES	BROKEN SHALES
Baltimore	1962	100	35	---	10	---	(4)
BOCA	1970	100	40	25	10	4	1.5
Boston	1970	100	50	10	10	---	(4)
Chicago	1970	100	100	100	---	---	---
Cleveland	1951/1969	---	---	25	---	---	---
Dallas	1968	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Detroit	1956	100	100	9600	12	12	---
Indiana	1967	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Kansas City	1961/1969	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8	---	---
New York State	----	100	40	15	---	---	---
Ohio	1970	100	40	15	10	4	---
Philadelphia	1969	50	15	10-15	8	---	---
Pittsburgh	1959/1969	25	25	25	8	8	---
Richmond	1968	100	40	25	10	4	1.5
St. Louis	1960/1970	100	40	25	10	1.5	1.5
San Francisco	1969	3-5	3-5	3-5	---	---	---
Uniform Bldg Code	1970	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
NBC Canada	1970	---	---	100	---	---	---
New South Wales	1974	---	---	33	13	4.5	---

(1 tsf = 96 kN/m^2)

1 - Year of code or original year and date of revision.
2 - Massive crystalline bedrock.
3 - Soft and broken rock, no shale.
4 - Allowable bearing stress to be determined by appropriate city official.
5 - q_u = unconfined compressive strength.

Presumptive values may be quite conservative relative to the actual capacity of the rock. Nevertheless, an applicable code may govern unless there is provision for a variance based on site-specific data.

Empirical Design Approach

Peck, Hanson and Thornburn (1974) present a correlation of allowable bearing pressure with Rock Quality Designation (RQD). The RQD value is a modified computation of percent rock core recovery that reflects the relative intensity of jointing and hence the compressibility of the rock mass. Their suggested values are shown below.

Table 15. Suggested values of bearing capacity from Peck, Hanson and Thornburn (1974).

RQD	ALLOWABLE PRESSURE (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

Note: See table 18 for recommended values based on rock type and RQD.

Peck, Hanson and Thornburn recommend that if RQD is reasonably uniform within one footing width below the footing, an allowable pressure may be selected based on the average RQD. If, as is often the case, RQD tends to increase with depth, then selection should be based on the RQD of rock within about 1/4 the footing width below the footing. In no case, however, should the allowable pressure be taken greater than the average unconfined compressive strength of intact rock core samples. No increase in bearing pressure is allowed for embedment, since the criteria is based on limitation of settlement, not available strength.

Peck, Hanson and Thornburn estimate that settlement of foundations using the allowable bearing pressures tabulated above would be no more than 1/2 inch.

The Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1978) suggests estimating the allowable bearing pressure with the formula:

$$Q_a = K_{sp}\, Q_u\text{-core} \qquad \text{(Eq. 24)}$$

where

Q_a = Allowable bearing pressure

Q_u-core = Average unconfined compressive strength of rock cores, as determined from ASTM D2938-N,

K_{sp} = Empirical coefficient depends on the spacing of discontinuities and includes a factor of safety of 3 as follows:

SPACING OF DISCONTINUITIES	K_{sp}
10 ft	0.4
3 to 10 ft	0.25
1 to 3 ft	0.1

where spacing is the perpendicular distance between parallel discontinuities as discussed under Spacing of Discontinuities in section 5.3.

The guidelines are intended for use with a rock mass with favorable characteristics; that is, the rock surface is perpendicular to the foundation load, the load has no tangential component, and there are no open discontinuities.

Burman and Hammet (1975) proposed a simplified design method whereby the allowable bearing capacity for jointed rock masses would be taken as the Brazilian or minimum nonuniform compressive strength of intact rock material. They reviewed laboratory tests which indicated that nonuniform load distribution within a jointed rock mass can lead to tensile fractures of individual elements, and thus the jointed mass can have a strength considerably less than the individual elements. The diametral point loading used in a Brazilian test represents this mode of failure and is used to define the lower limit of rock mass strength. The test can be conducted on typical rock core samples. For further discussion refer to Intact Rock Properties under section 5.3.

Field Load Test

Full scale field load tests, such as that shown on figure 40, are the most reliable method of determining a design bearing pressure. However, these tests are expensive and are generally not warranted unless very high stresses are anticipated, such as for piers for a long span or arch bridge.

Plate jacking tests can be used under similar circumstances. They have the advantage of being less expensive than full scale load tests, although interpretation of results is more difficult due to scale effects. Test apparatus and procedures for plate jacking tests are described by Deere et al. (1967).

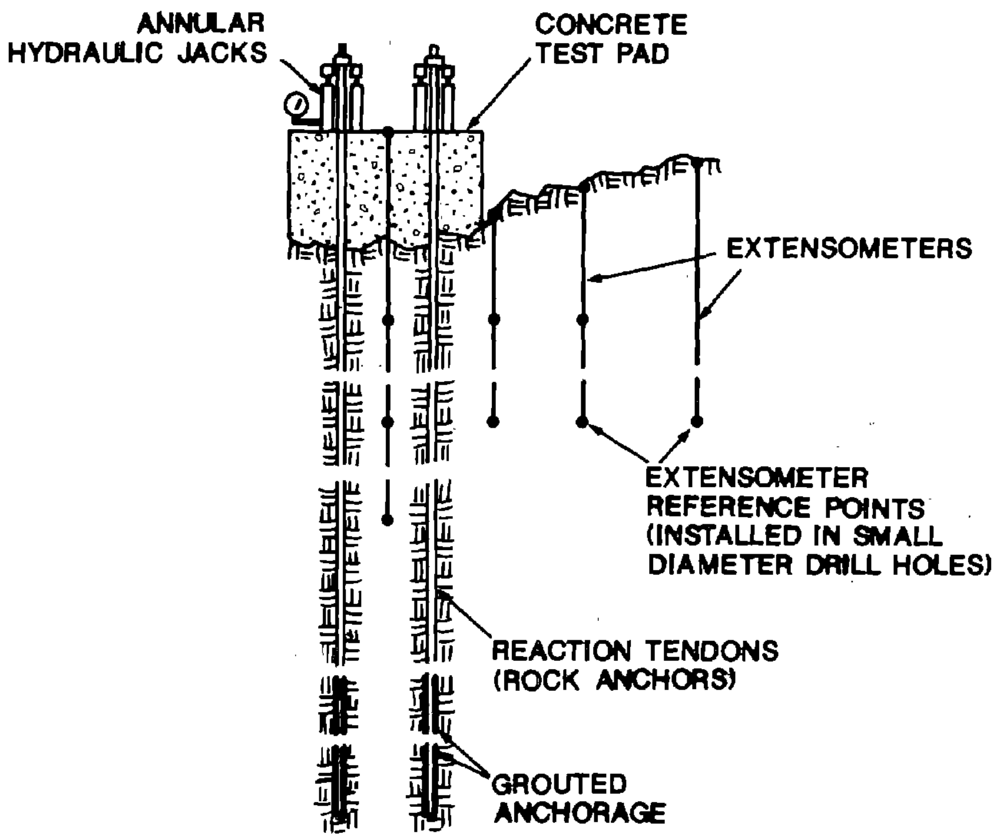


Figure 40. Schematic sketch of field load-test arrangement for determining bearing capacity and compressibility of rock. Concrete test pad is loaded by hydraulic jacks reacting against steel tendons anchored at great depth. Vertical strains are measured between extensometer reference points anchored to rock walls of small drill-holes at suitable locations. (From Peck, Hanson and Thornburn, 1974.)

Comparison and Evaluation of Methods

Although it is the most reliable test to determine allowable bearing capacity, field load tests are generally not warranted unless very high stresses are anticipated, and settlement is a potential problem, as for piers for a long-span or arch bridge. If settlement is a real problem, the best estimates will require large scale field tests, as described in the following section on Settlement, Field Load Test. In these cases, it is best to have field explorations, testing, and analysis performed by a geotechnical engineer familiar with the exploration, testing, and design procedures.

Presumptive bearing values for codes can vary considerably in their recommendations from one region to another. For the relatively low bearing pressures typical of most bridge abutment footings in unweathered rock, the code values are reasonable to use, since little or no economy can be realized by increasing the values of allowable bearing pressures. For piers where large loading is imposed, the code values may be overconservative, leading to overly expensive foundations.

The Peck, Hanson and Thornburn approach is more rational, in that it takes into account the intensity of jointing (RQD), which is an indicator of rock mass compressibility. The method is intended to limit settlement to less than 0.5 in, and allows bearing values for good quality rock (RQD 75 percent) which are greater than most presumptive values found in codes.

Figure 41 compares the Peck, Hanson and Thornburn values to permissible bearing stress allowed by two typical building codes. It should be noted that the allowable bearing pressure corresponding to RQD greater than about 75 percent exceeds the maximum bearing stress permitted for most unreinforced concrete (f_c = 4000 psi).

Settlement

General

As noted previously, when load is applied to footings on rock, settlement may result from compression of individual blocks, slippage between blocks, volume decrease of joint infill materials, and closure of open joints. Design loads selected for bridge footings on rock are generally several times less than the ultimate bearing capacity of the rock, and settlements under the design loads for most bridge abutment footings would be very small.

However, on some heavily loaded foundations, such as piers for long-span bridges or rigid arch bridges, settlement limits may be much smaller and estimated settlement may be an important design consideration.

In order to estimate foundation displacement, it is first necessary to estimate the deformation modulus of the rock mass. The modulus generally varies with depth, so estimates of modulus of deformation with depth are often necessary to determine the depth and size of foundations in order to keep within tolerable settlement limits.

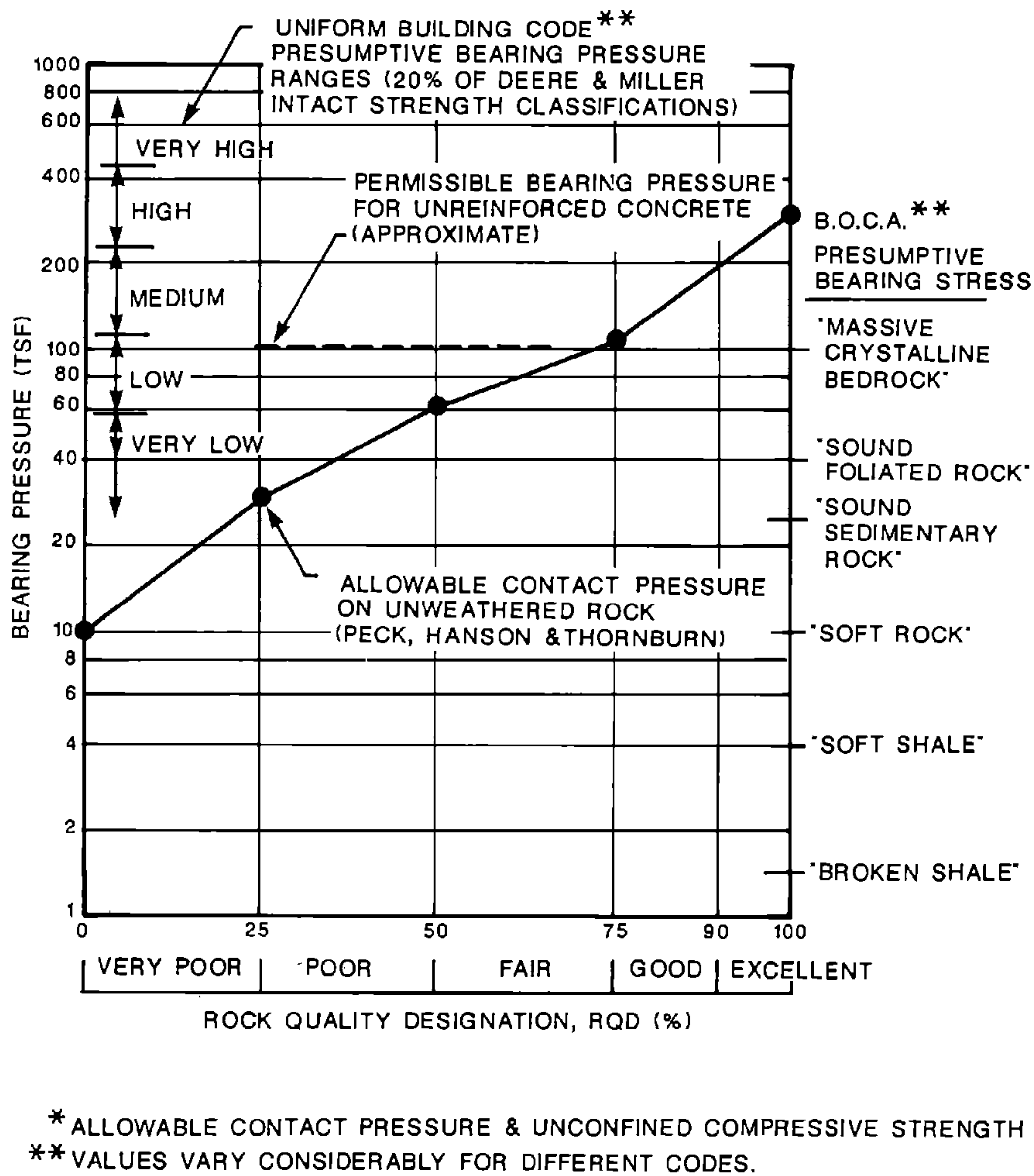


Figure 41. Comparison of allowable bearing pressure criteria.

In the following paragraphs, three methods of modulus determination and settlement estimates will be discussed: Empirical Design Approach, Field Load Test, and Finite Element Studies.

Empirical Design Approach

It is possible to make a crude estimate of settlement using elastic theory and an empirical determination of in situ rock modulus. An expression for vertical compression beneath the center of a uniformly-loaded circular pier on a homogeneous elastic half-space is as follows (Kulhawy, 1978):

$$\rho = \frac{P\,(1-\nu^2)}{B\,A^{1/2}E_m} \cong \frac{P}{A^{1/2}E} \qquad \text{(Eq. 25)}$$

for the following typical values

ρ = vertical deformation (inches)
ν = poisson's ratio (about 0.20)
B = shape factor (about 1.0)
A = footing area (in^2)
E_m = elastic modulus of rock mass (psi)
P = column load (lb)

The main difficulty in applying the above equation (or other numerical analyses) is in selection of an appropriate modulus for the rock. Modulus values for intact core are not representative of the rock mass because they do not account for the presence of fractures.

One method for estimating in situ rock modulus is to use a modulus reduction factor E_m/E_L or E_m/E_{seis} to estimate the relationship of the elastic modulus of the rock mass, E_m, to either the intact rock core modulus, E_L, measured in the laboratory or the seismic modulus, E_{seis}. Deere et al (1967) have suggested a modulus reduction factor related to RQD and velocity ratio, which is shown on figure 42. The velocity ratio is defined as the seismic velocity of the in situ rock mass, measured in the field (V_F), divided by the sonic velocity of an intact specimen measured in the laboratory (V_L). A high quality, massive rock with high RQD would be expected to have a velocity ratio approaching one, while poorer quality rock, with lower RQD, would have decreasing values. Deere et al. found an approximate one to one correlation between the square of velocity ratio and RQD for several sites, and therefore used the two indices of rock quality interchangeably on figure 42. Bieniawski (1981) has also proposed a modulus reduction factor related to his geomechanics rock classification rating system.

An example settlement computation for a 5-ft-diameter-pier, bearing on a rock mass having an intact modulus of 10^6 psi, is presented in figure 43. As indicated, the settlements would theoretically be less than 1/2 in for contact pressures up to the allowable value suggested by Peck, Hanson and Thornburn (1974) or table 15.

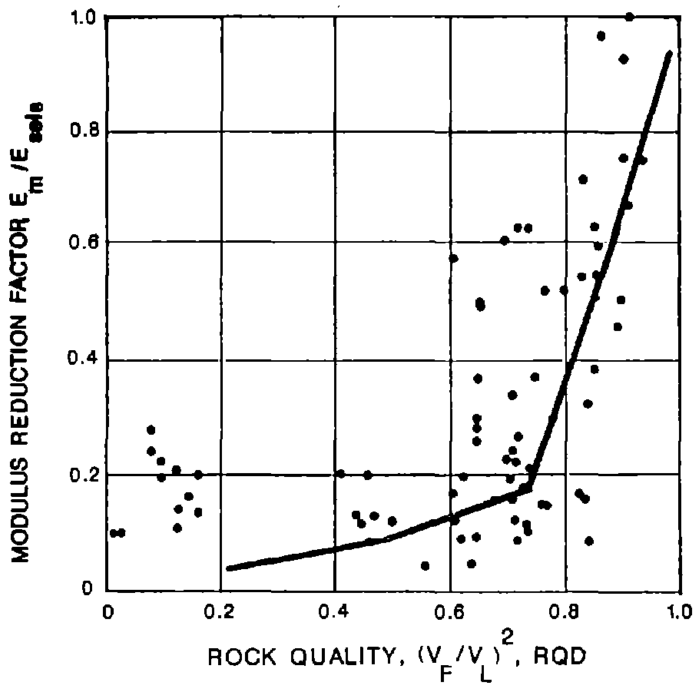


Figure 42. Variation of modulus reduction factor with RQD and Velocity Ratio.

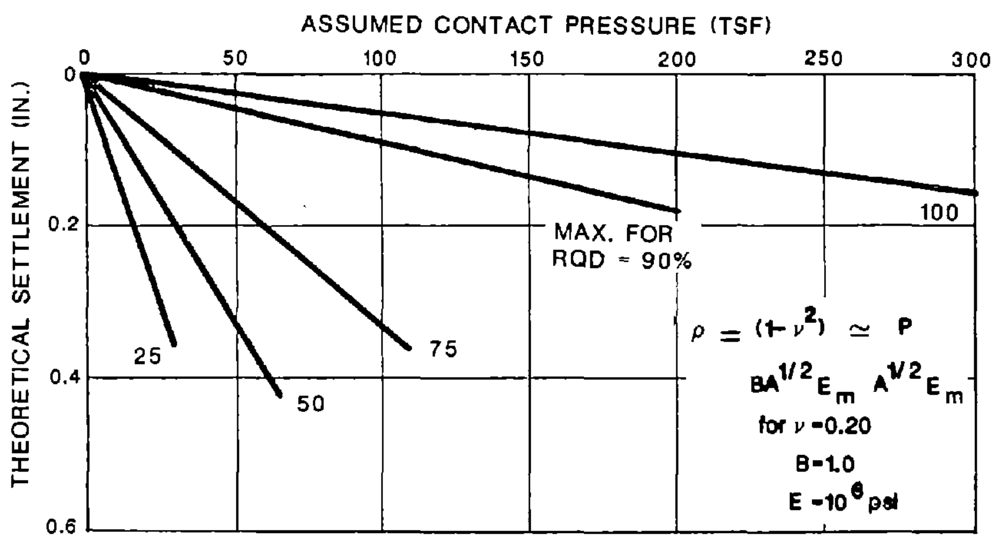


Figure 43. Computed elastic settlements for 5-ft diameter bearing surface.

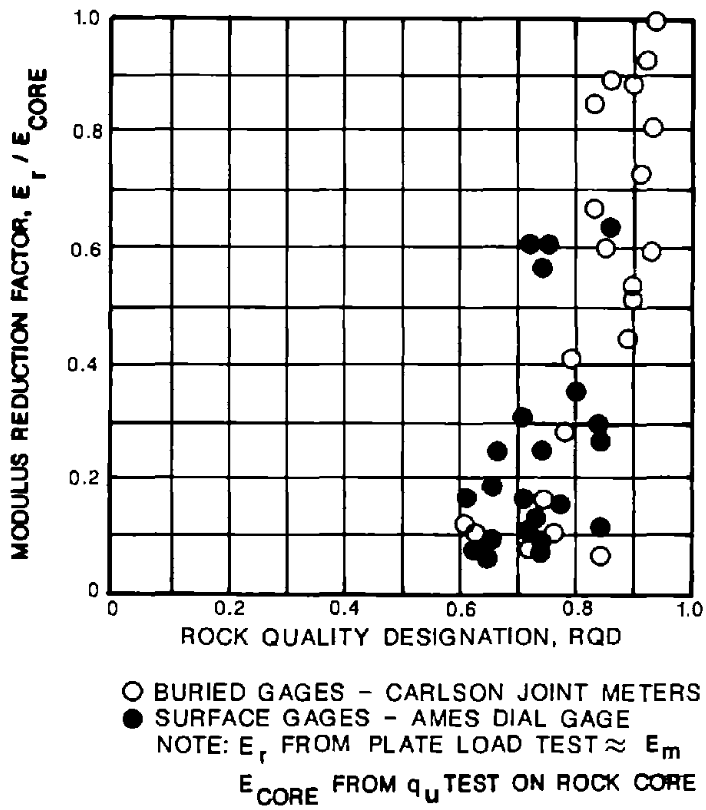


Figure 44. Example of site-specific variation of modulus reduction factor with RQD.

Field Load Test

It should be emphasized that actual settlements of a particular footing will depend on site-specific rock conditions. The only reliable way to estimate in situ rock modulus, and arrive at meaningful estimates of settlements is to conduct large scale field load tests (see figure 40). The loaded area must be the same area as a typical footing, or larger than the discontinuity spacing so as to stress a representative volume of the rock mass. Deere et al. (1967) describe the two most common types of field measurements of the static in situ modulus of deformation: plate jacking tests and pressure chamber tests.

In the case of plate jacking tests, the results can often be used directly to estimate settlement. For variable rock conditions, the results of several field tests can be correlated with intact rock core modulus determined in the laboratory, and with RQD, to generate a curve for the site similar to figure 44. Settlements for various contact stresses and rock qualities can then be calculated.

Finite Element Analysis

The deformation below a footing on rock can also be estimated using the theory of elasticity together with a finite element method of numerical modeling. As indicated previously, the results of these techniques are only as good as the estimated in situ modulus of deformation. Finite element methods are generally expensive and not warranted for most footing designs on rock. However, for heavily loaded footings with odd shapes or eccentric loading, the finite element methods may be instructive in identifying relative stresses and displacements beneath the footing and identifying and understanding important factors which govern the behavior of the loaded rock mass.

5.3 ROCK QUALITY PARAMETERS

General

Rock masses are complex, homogeneous media of intact rock materials and naturally occurring discontinuities such as joints, shears, faults, bedding planes and cleavage planes. The rock mass exhibits the characteristics of both the rock materials and the discontinuities. Since most intact, unweathered rock generally is stronger and less compressible than concrete, the influence of these discontinuities should govern foundation design. During the geotechnical investigations, it is important to determine the spacing, attitude, thickness, amount of weathering and filling of all discontinuities within the zone of influence of the proposed foundations.

The description of rock masses and their discontinuities is generally done using drill core and/or outcrop mapping. The following sections review important parameters used to classify intact rock and rock masses.

Geologic Classification

General

Rock may be defined as a consolidated or coherent and relatively hard, naturally formed mass of mineral matter. Rock constitutes an essential and appreciable part of the earth's crust. Rock cannot normally be excavated by manual methods alone. For the purpose of differentiating between soil and rock for payment purposes in contract specifications, rock is often defined as solid rock or rock in place, the removal of which requires drilling and blasting. For the purpose of design of footings, the latter definition is more meaningful. There are many naturally occurring materials which may be defined geologically as rock but which should be treated as soils. Examples are: very weak rock or weakly cemented rocks with unconfined compressive strength lower than 125 lb/in^2; and highly weathered or crushed rock or other rock which can be excavated by hand using a shovel or pneumatic spade.

There are three classes of rock based on geologic origin as defined below:

Igneous Rocks

Igneous rocks were formed by solidification of molten material termed magma. There are two kinds of igneous rock: intrusive (sometimes called

Plutonic) rock, which was formed by slow cooling of molten magma at great depth within the earth's crust, with resultant large crystals; and extrusive (or volcanic) rock, which was formed by the rapid cooling of magma at the earth's surface, with consequent very small crystals.

Examples of coarse grained intrusive igneous rocks are granite, syenite, diorite and gabbro. The fine grained extrusive igneous rocks include rhyolite, trachyte, andesite, basalt, and diabase.

Some noticeable features of igneous rocks are their uniformity of structure and the presence of interlocking crystals. Because of the lack of stratification or cleavage planes found in sedimentary and metamorphic rock, igneous rocks are generally a competent footing bearing material when unweathered.

Sedimentary Rocks

Sedimentary rocks are the products of the disintegration and decomposition by weathering of pre-existing rocks. They may also derive from calcareous constituents of lake and ocean waters. The rocks are consolidated by mechanical cementation and by chemical precipitants, sometimes accompanied by pressure. Examples of sedimentary rock are sandstone, limestone, dolomite, shale, and chert.

Some noticeable features of sedimentary rocks include rounded grains, stratification in relatively thick layers, and abrupt changes in color from layer to layer. The various layers, or beds, may vary in texture, color, composition, and thickness. Some compressible materials, such as bentonite or soft shale, may be present in layers within a hard rock. Also, the bedding planes are often inclined due to tectonic upheaval and they form planes of weakness along which rock blocks may move when loaded.

Metamorphic Rocks

Metamorphic rocks were formed from igneous or sedimentary rocks which have been altered physically or chemically by intense heat, pressure and attendant gasses and liquids below the earth's surface. Examples are quartzite, marble, slate, gneiss, and schist.

Some noticeable features of metamorphic rocks include a separation of crystals into approximately parallel layers, as contrasted to the uniformity of structure of igneous rocks; and the facility with which parallel layers will break into slabs along these foliation planes. Depending on the degree of lamination or foliation and its attitude, these planes of weakness can have an impact on foundation design.

Intact Rock Properties

The primary intact rock property of interest in foundation design is the unconfined compressive strength. Although it is known that the strength of a jointed rock mass is generally less than that of the individual units of intact rock composing the mass, the unconfined compression test on intact rock cores is often used to:

- Provide an upper limit of the allowable bearing capacity of a rock mass.
- Provide an index to assist in classifying the rock.
- In very weak or highly weathered rock, to provide an estimate of the ultimate bearing capacity and also to determine if the rock is so weak it should be treated like a soil (see Geologic Classification, General, above).

Another test proposed for use by Burman and Hammet (1975) in assessing the strength of a jointed rock is the Brazilian compressive strength. They reviewed results of laboratory tests on simple block-jointed materials under uniform compressive loading and suggested that the strength reduction below that of unjointed material arises from nonuniformity of load distribution within the jointed medium. This nonuniform loading can induce brittle tensile fracture in the rock blocks. The Brazilian test uses a diametral point loading to determine the compressive strength of rock samples under extreme conditions of nonuniform loading, as shown in figures 45 and 46.

Burman and Hammet indicate that the strength of a jointed rock mass will lie between the limits defined by the Brazilian and unconfined compression tests.

Rock Mass Structural Features

Structural Discontinuities

Structural discontinuities are present in almost all near surface rock masses. They are defined as the geologic features which separate intact blocks of rock, such as joints, shears, faults, bedding planes, and foliation planes. These discontinuities represent planes of weakness which can reduce significantly the bearing capacity of a rock mass.

Nature and Orientation of Discontinuities

In assessing the impact of discontinuities on foundation design, it is important to know the width (or aperture) of the opening, the character of any infill materials, and the degree of weathering along the rock faces of the discontinuity. In addition, the orientation of discontinuities should be known with respect to the applied load.

Open, near vertical joints are often encountered in foundation excavations in rock. Sometimes the joints will be soil filled. Below footing foundations for bridge abutments such joints do not usually present problems. If necessary, they may be cleaned out and filled with grout or dental concrete.

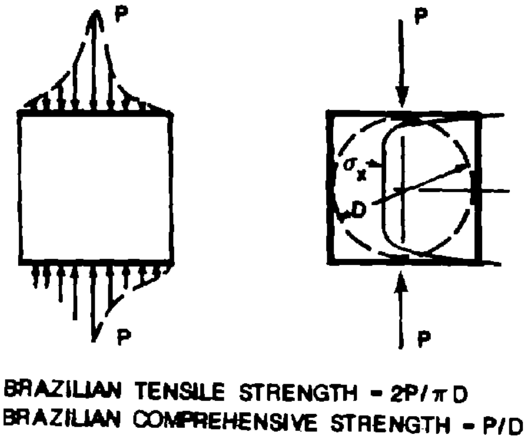


Figure 45. Brazilian compressive strength test: Nonuniform compressive loading of blocks.

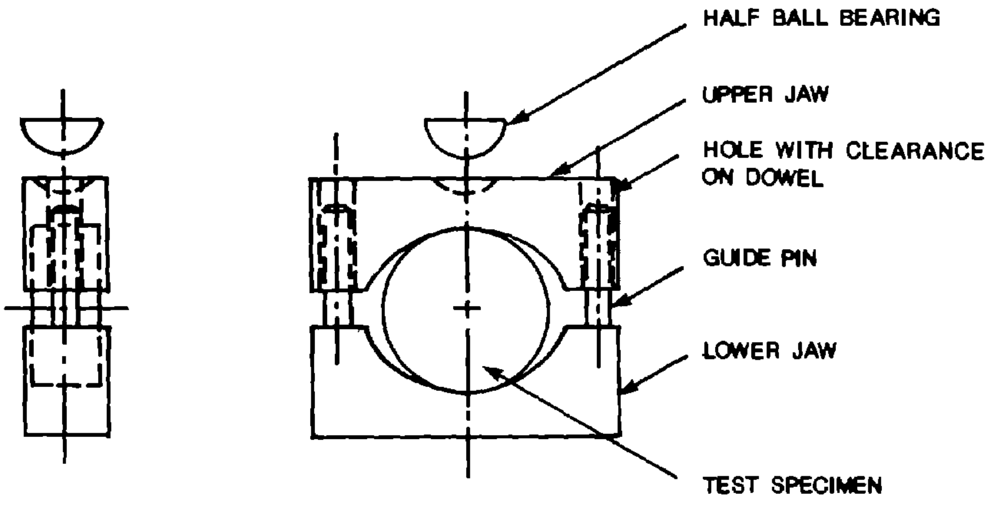


Figure 46. Apparatus for Brazil test (after Bieniawski, 1981).

Intersecting vertical joints, together with near surface weathering, can create very large openings near the top of rock. In the case of a heavily loaded bridge pier, the opening may constitute a significant fraction of the bearing area of the pier, and require overexcavation until the joints narrow, close, or until they are no longer within the base.

Nearly horizontal joints can also be a problem in foundation design. Horizontal joints are often open due to relief of vertical stress from past erosion of overlying rock. These joints are often filled with clay or other compressible material. When loaded by a footing, settlement will occur, and often the settlement will be uneven and possibly very sudden.

Faults and shear zones generally have an area of intense fracturing and crushing from a few inches to several feet thick. Often the fault or shear has been healed and the discontinuity may not impact the foundation design. On the other hand, the fractured zone may consist of a fault gouge which has low strength and high compressibility. In this case, the properties of the gouge must be determined in order to make estimates of deformation under design loads.

Spacing of Discontinuities

The spacing of discontinuities in a rock mass is an indication of the overall rock quality and thus indirectly affects the allowable bearing capacity of the rock. The spacing of discontinuities should be compared to the proposed footing width, particularly for smaller footings or piers. If the spacing approaches the footing width and the joints are tight and unweathered, the rock properties below the footing should approach those of intact rock.

Spacing of discontinuities can be defined using the terminology recommended by the International Society for Rock Mechanics (1981) as described in table 16.

Table 16. Definition of spacing of discontinuities.

Description		
Extremely close spacing	< 20 mm	(0.8 in)
Very close spacing	20 to 60 mm	(0.8 to 2.4 in)
Close spacing	60 to 200 mm	(2.4 to 7.9 in)
Moderate spacing	200 to 600 mm	(0.7 to 2.0 ft)
Wide spacing	600 to 2000 mm	(2.0 to 6.6 ft)
Very wide spacing	2000 to 6000 mm	(6.6 to 20.0 ft)
Extremely wide spacing	> 6000 mm	(20.0 ft)

Spacing is measured as the perpendicular distance between parallel discontinuities. Measurement can usually be easily accomplished for rock outcrops, however, for inclined discontinuities measurement of spacing from vertical drill cores can be difficult. If the rock has foliation or bedding planes, the core pieces can be matched and oriented and the spacing measured by:

$$S = L \sin \theta \qquad \text{(Eq. 26)}$$

where:

S = Spacing

L = Length measured along the core axis between adjacent discontinuities of the same orientation

and θ = The acute angle these features subtend with the core axis.

If the rock has no foliation or bedding features and it is difficult to match and align core pieces, a simple method of estimating the in situ rock mass quality is to determine the fracture frequency, which is simply the number of natural discontinuities (of any orientation) per unit length of drill core. Drill cores with one fracture or less per foot would indicate a good quality rock mass with properties approaching those of intact rock. High fracture frequencies (four to six fractures per foot) would indicate a poorer quality rock and the rock mass would be considerably weaker and more compressible than the intact rock.

Rock Quality Designation (RQD)

RQD, or Rock Quality Designation, was developed by Deere (1968) as a quantitative index of rock mass quality derived from drill core. RQD is based on a modified core recovery procedure which indirectly takes into account the number of fractures and the amount of weathering and softening in the rock. RQD is determined by measuring and summing all the pieces of sound core four inches and longer in length in a core run, and dividing this modified recovery by the core run.

An example of determining RQD is given in figure 47. For a core run of 60 in, the total core recovery is 50 in, giving a core recovery of 83 percent. The modified core recovery was 34 in, giving an RQD of 57 percent. The RQD should be computed using NX (2-1/8 in diameter) or larger size core. Smaller BX core may fracture and break due to the drilling operation and thus lead to incorrect values of RQD.

RQD provides a good preliminary estimate of the variation in properties of the in situ rock mass from the intact rock core. From this, a general assessment can be made as to the overall rock mass quality and engineering behavior. Table 17 gives the relationship between RQD and rock mass quality. An RQD of 100 percent would represent an excellent quality rock mass whose engineering properties under footing loading would be similar to that of an intact specimen. An RQD between 0 and 50 percent would represent a poor quality rock mass whose engineering properties under footing loading would be very much lower than those of an intact specimen. Nevertheless, even poor quality rock with low RQD values can provide very satisfactory foundation support. Refer to the settlement computations in figure 43 for example where RQD of 25 results in settlement of less than 0.4 in for

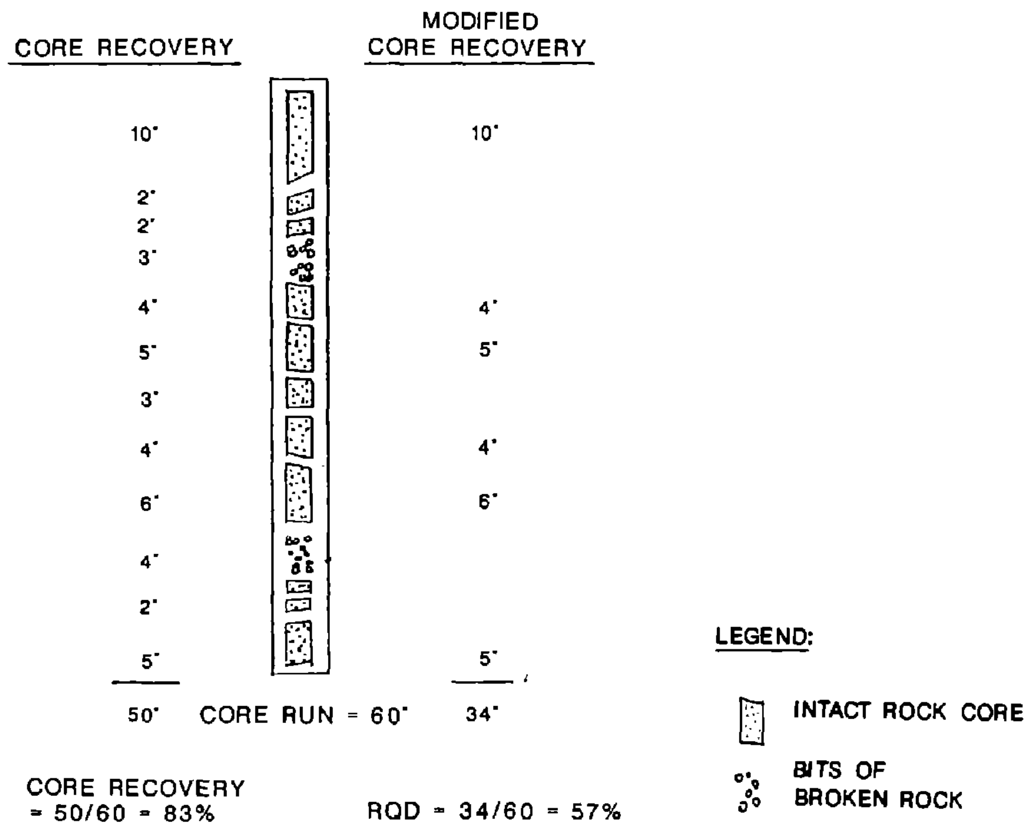


Figure 47. Determination of modified core recovery (RQD).

Table 17. RQD as an index of rock quality.

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0- 25	VERY POOR
25- 50	POOR
50- 75	FAIR
75- 90	GOOD
90-100	EXCELLENT

applied stresses of 30 tsf. For typical bridges with loads less than 10 tsf the settlement of poor quality rock would be tolerable.

Some problems arise in the use of RQD. One is drilling technique and equipment. Poor drilling technique and equipment will "penalize" the rock quality by lowering the recovery and causing fresh breaks which are not related to the quality of the rock mass. For these reasons, it is important to use drillers experienced in rock coring, to use double-tube core barrels of at least NX size (2-1/8 in diameter), and to have monitoring of core drilling by a geologist or geotechnical engineer experienced in rock coring procedures and equipment.

Another potential problem in using RQD is in determining tightness of individual joints. It is not possible to determine joint aperture, or opening width, from rock core unless expensive overcoring techniques or borehole photography are used. Overcoring techniques are described in Thompson et al. (1980), and borehole television techniques are described by Ellis et al. (1977). RQD determination is not sensitive to joint aperture, except in the case of very wide openings, whereas in some cases, such as for horizontally jointed rocks, the in situ deformation modulus may be strongly affected by the average joint aperture.

5.4 PROBLEM ROCKS AND CONDITIONS

General

A number of special problems and conditions can strongly influence the design of rock foundations. Described below are four problem rocks and conditions: weathering of rock, solution cavities, swelling rocks, and foundations on rock slopes. In addition to these, other conditions or defects, which may not be as common, can also impact design.

One of these conditions is creep in salt, gypsum, and in some cases in clayshale and claystone. Creep is a continued, long-term deformation under constant loading which can lead to long-term settlement problems. Another condition is that of rock formations above abandoned mineral mines. Long-term subsidence or even surface collapse can result, and often special investigations and expensive treatment are required.

Weathering

Weathering may be defined as the group of processes whereby rocks, on exposure to the weather, change in character, decay, and finally crumble into soil. The processes of weathering may include the chemical action of air and rain water and of plants and bacteria, and the mechanical action of changes in temperature. Generally, the effects of weathering increase as temperature and humidity increase.

Weathering in rock can result in a great variety of physical properties, from a discoloration of hard, intact rock to a resultant soil-like material which crumbles easily. There is often a zone of weathered rock between overlying soils and underlying unweathered rock. This weathered zone may have a great variety of physical properties over short horizontal and vertical distances, and may be the controlling factor in foundation design and construction.

Peck, Hanson and Thornburn (1974) give a detailed description of various types of weathering and the influence on design. Although few generalizations can be made concerning design of footings on weathered rock, the designer should be aware of the possible presence of weathered zones; should look for them in the exploration program; and be prepared to overexcavate and lower footings, or even change the foundation design during construction.

Solution Cavities

Solution features, such as irregular bedrock surface, open vertical joints, clay seams, cavities and sinkholes, can pose serious design and construction problems and require detailed attention during the exploration phase. These features are encountered in gypsum and salt, but most commonly in karstic limestone.

Because of the uneven surface and unpredictable rock quality, it may be very difficult to predict the foundation elevation and allowable bearing pressure. In addition, rocks with solution features or open structures are generally very pervious, so groundwater control can be a problem where bearing levels are below water levels. Groundwater levels can be irregular, and because of the varied and unpredictable nature of the rock quality and groundwater levels, a conservative approach is necessary during design. Additional borings and probes are generally warranted in order to find and define cavities, sinkholes, or other rock defects which might exist below proposed foundations. The design should be flexible enough to allow for changes in bearing elevation or allowable pressure during construction, and grouting of cavities may be included in the design.

Swelling Rocks

Foundation problems relating to swelling and heave can occur in rocks with expansive or unstable minerals such as some clayshales with montmorillonite, basalts with monzonite, and other rocks with pyrite, myrmekite, and marcasite. Swelling in shales is most common. The unloading effect of excavation can release locked in stresses and result in significant swelling over prolonged periods of time. In addition, some "alum shales," which are black, carbonaceous shales of the Paleozoic era, contain pyrite which oxidizes and forms crystals of gypsum or jarosite. Significant swelling can occur due to the growth of these crystals.

Foundations on Rock Slopes

Often bridge footings are required to be constructed on a rock slope or near the top of a natural rock slope or rock cut. This is illustrated by the photograph of figure 48 and schematically in figure 49. Figure 49 indicates that discontinuities in the rock mass can form a wedge beneath a footing which may be unstable due to the combined loading of the rock itself and the footing. Procedures for the analysis of the stability of the rock cut are shown on figure 49 and given in more detail in a Corps of Engineers Rock Reinforcement manual (U.S. Army Corps of Engineers, 1980). In such a case, rock dowels or rock anchors are frequently used as rock reinforcement to stabilize the rock wedge.

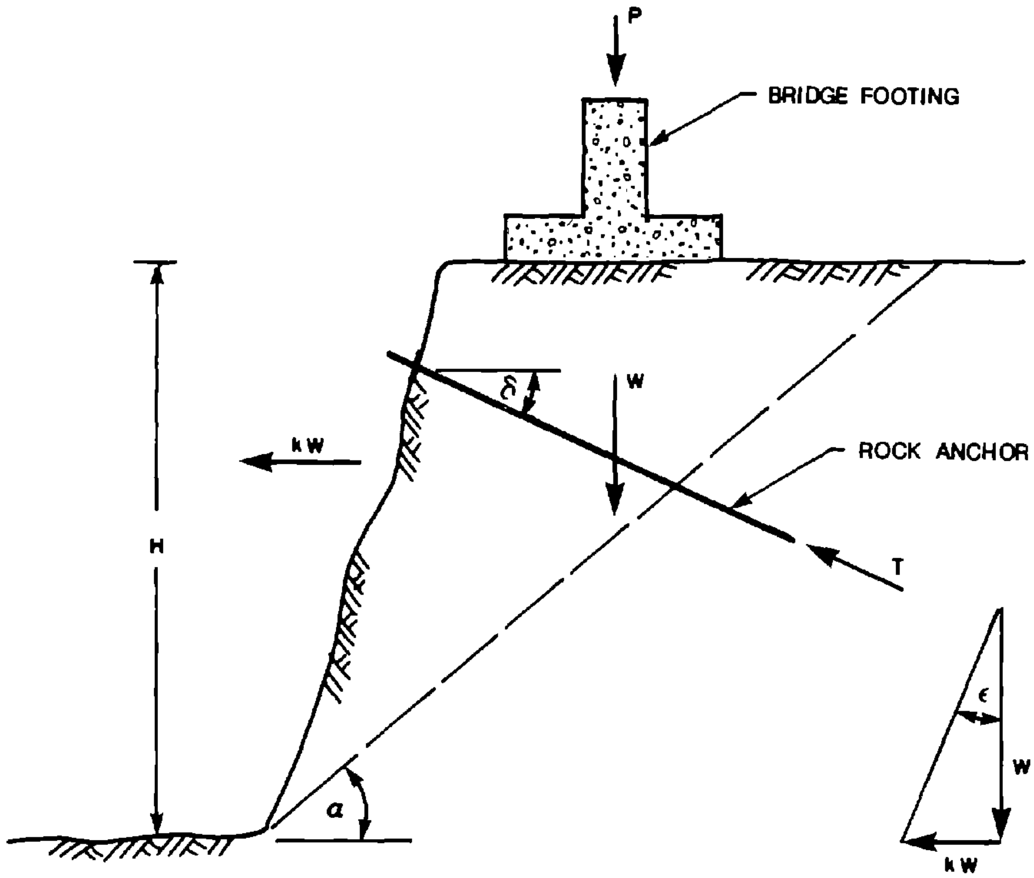
Figure 48. Photograph of bridge abutment on rock slope.

Figure 50 illustrates a situation of a footing on an inclined rock surface. Grouted bars or dowels are often placed in drill holes and cast into the footing to provide resistance against sliding of the footing or an underlying slab of rock.

Both cases serve to point out the need for an exploration program to define the orientation of discontinuities and to define the nature and variations in elevation of the top of bedrock in the area of proposed footings.

Rock Excavation

When rock excavation must be completed by drilling and blasting, the final bearing surface will be irregular and often intentionally 1 to 3 ft lower than the design grade. This overexcavation is accomplished for practical reasons; rock will not reliably break on a flat plane between drill holes. Therefore some overexcavation is planned to be sure that no high spots of intact rock are left between drillholes. High spots would interfere with placement of steel reinforcing and concrete for the footing. Removal of high spots is typically too costly and time consuming, therefore overexcavation becomes a tolerable design and field procedure. If the overbreak zone is filled with concrete, after removal of loose blast rock, then footing design is controlled by the rock properties. If the overbreak is backfilled with compacted fill or crushed stone then footing design is controlled by the soil or backfill properties.



$$F.S. = \frac{1}{W_T \sin(\alpha+\epsilon)}\left\{cH \operatorname{cosec}\left[\alpha + W_T \cos(\alpha+\epsilon) + T\sin(\alpha+\delta)\right] \tan\phi + T\cos(\alpha+\delta)\right\}$$

WHERE: W_T = W + P = WEIGHT OF ROCK + FOOTING LOAD

kW = EARTHQUAKE OR BLASTING FORCE WHERE k IS USUALLY ASSUMED TO BE 0.1

T = TOTAL FORCE DUE TO ROCK ANCHOR

ϕ = FRICTION ANGLE ALONG DISCONTINUITY

c = COHESION OF ROCK ALONG DISCONTINUITY

Reference: Corps of Engineers,
Rock Reinforcement,
EM1110-1-2907,
February 1980

Figure 49. Corps of Engineers method of analysis of rock cut stability.

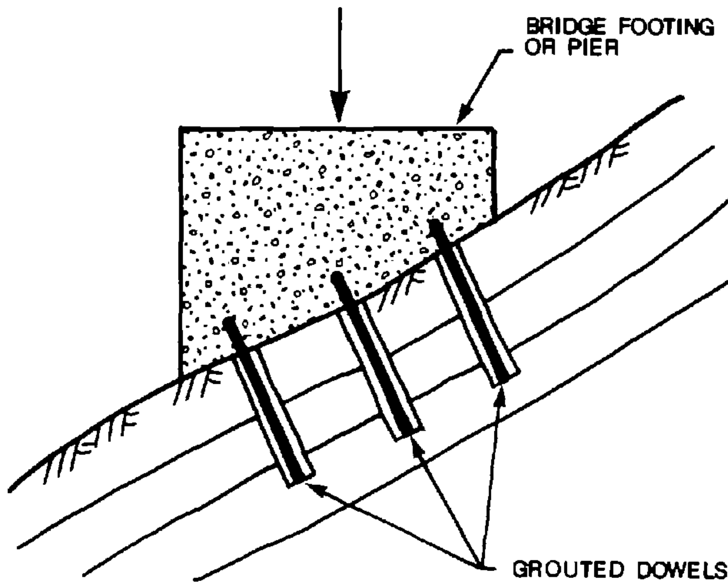


Figure 50. Treatment of rock footing on sloped surface.

Embedment of a footing into rock can thus result in extra cost and a loss in bearing capacity. Since minor embedment of 6 in to 2 ft into rock does not materially alter bearing capacity, foundation bearing design should eliminate unnecessary embedment procedures. Where bonding to the rock is required, use alternate methods such as the rock dowels shown in figure 50.

5.5 RECOMMENDED DESIGN PROCEDURES

Investigation of Rock

General

The exploration phase may often be the most important part of the design and construction of footings on rock. The primary purpose of the exploration program is to determine the location of the rock on which the footings will bear. Information must be gathered on which to base a decision of an allowable bearing pressure to use in design. Also, if there are weathering effects at the top of rock, a determination must be made as to the required depth of excavation into the rock.

Geologic Mapping

If rock outcrops exist in the vicinity of the proposed bridge footings, valuable information can be gained as to rock type, discontinuities, and other defects. Geologic mapping of outcrops should be done to determine orientation, spacing, and aperture (joint width) of major joint sets. Often evidence of shear zones or faults is apparent at the ground surface or at outcrops, and the orientation can sometimes be determined.

The International Society for Rock Mechanics (1981) gives criteria for classification of rock masses as observed in outcrops. The spacing, orientation, and aperture can be easily determined for observations of outcrops, while it is difficult or impossible to do so with core borings without the use of expensive borehole photography or core orientation techniques.

Kulhawy and Goodman (1980) have correlated the frequency of discontinuities with RQD as shown in figure 49. Thus, a correlation can be made of RQD using outcrop measurements of joint spacing. In this fashion, the allowable bearing pressure can be estimated from the RQD as described in Settlement Evaluation below.

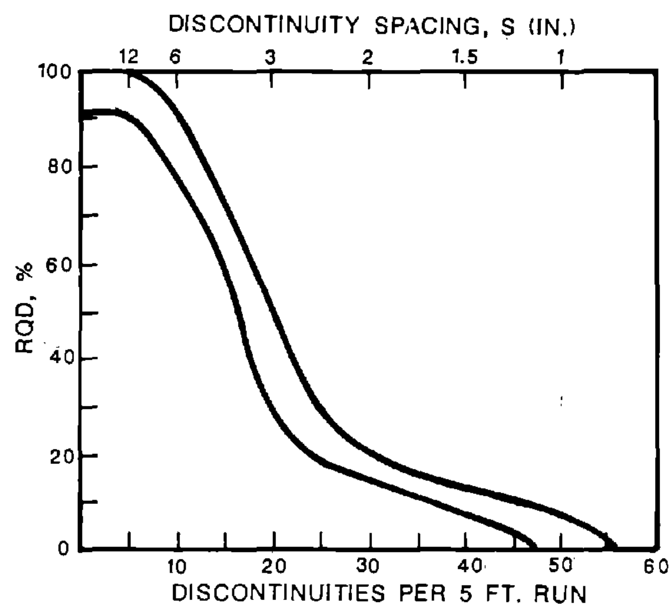


Figure 51. Correlation between frequency of discontinuities and RQD.

Rock Drilling

Rock core drilling should be done with equipment equal to or exceeding "NX" size double tube core barrels with diamond bits. If rock is highly fractured, triple tube core barrels and/or split inner liners may be required to get good recovery. Drilling procedures are important in obtaining good recovery. Experienced crews should be used, and a geologist or geotechnical engineer who is experienced in rock drilling techniques should monitor borings in order to vary drilling depths and procedures based on conditions encountered in the field. Changes in drilling noise, vibrations, bit pressure, drill water loss, and advance rate should be noted and carefully recorded.

For most bridge footings on rock, where imposed bearing pressures are relatively low, core drilling should generally be extended to a depth about 10 ft into unweathered rock, to establish that there are no major rock defects (i.e., faults, shear zones, solution cavity, etc.) below the footing. Where high footing bearing pressures are required or where settlement is a potential problem, core drilling should be extended to a minimum depth below anticipated bearing elevation of about two times the least footing dimension, to be sure that all rock is investigated within the zone of influence below the footings.

If a highly irregular rock surface is encountered, such as a karst or highly weathered rock, it may be more important and economical to obtain a lot of information describing depth to rock rather than obtaining a few core samples. This can often be done with air track rotary percussion drills, which can quickly drill many holes to determine the top of bedrock. This system uses the driller's "feel" for increased resistance to drilling to determine the top of weathered and sound rock and changes in rock type.

An acoustic sounding technique, described by Stimpson, Brierly, and Liu (1976) combines the use of a percussion drill with a sensitive noise level indicator to better define the top of sound rock. Because of the different sound transmission characteristics of overburden and bedrock, it is possible to distinguish, by sound intensity and experience, when the drill bit penetrates from overburden into bedrock. Figure 52 shows the technique whereby a listening hole is drilled into sound bedrock and filled with water. A geophone is lowered into this "listening hole" and connected to a noise level indicator. The operator "listens" to the drilling noise and can thereby distinguish whether the drill bit has encountered bedrock (a loud noise) or a boulder above the rock (a muffled noise of reduced intensity).

Both these percussion drill techniques can drill many holes economically in order to assist in determining top of rock. However, the technique is very sensitive to the overlying soil conditions and to the experience of the operators. Several core borings should always be used for correlation with the air track data, and to obtain samples and RQD determination of the rock mass below footing bearing elevation.

Core Evaluation

Classification

Rock core can be classified as described in section 5.3. Classification of rock core is described in detail by the International Society of Rock

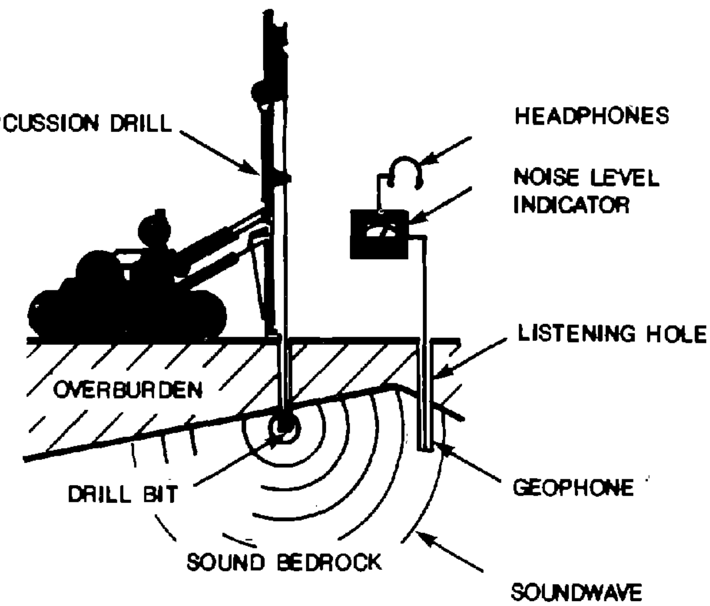


Figure 52. Acoustic sounding technique for determining top of bedrock.

Mechanics (1981). The important parameters to be determined are rock type, degree of weathering, and the nature, orientation, and spacing of discontinuities.

RQD Determination

RQD should be determined, along with recovery, for all rock core runs. The procedure for calculating RQD is described above in section 5.3, Rock Quality Designation.

Laboratory Tests

Laboratory testing of rock core is generally not required except for the case of heavily loaded piers or footings where very high bearing pressures would result in savings or where settlement estimates are desirable based on a modulus ratio of intact rock modulus to rock mass modulus.

Bearing Capacity Evaluation

It is recommended that allowable bearing pressures be determined based on a combination of presumptive bearing values based on rock type and the use of RQD to take into account discontinuities in the rock mass.

As noted previously, the selection of presumptive bearing pressures based on rock type, although very conservative, is generally satisfactory for most footings for bridge abutments because of the low imposed bearing stresses.

If higher bearing stresses would be desirable to save money by reducing footing size, RQD can be used to take into account the effects of discontinuities in the rock mass.

The resulting table 18 is a combination of the presumptive bearing values of BOCA (table 14) and the allowable contact pressures on jointed rock by Peck, Hanson and Thornburn (1974) based on RQD (table 15). It should be noted that allowable contact pressures noted are for unweathered rock. Use of the bearing values noted, in unweathered rock, should result in less than 0.5 in of settlement.

Most bridge footings, even when designed for the minimum possible footing width, impose a bearing pressure on rock less than 10 tsf, so it can be seen that an extensive core drilling program or laboratory testing program are usually not warranted. The most important design aspects in these cases will be to determine the depth to sound rock at foundation locations, and to ensure that there are no major defects such as solution cavities or fault or shear zones in the areas.

Settlement Evaluation

As noted previously, the use of allowable bearing pressures in table 18 should result in less than 1/2 in of settlement of the footing, provided the rock is relatively unweathered. In the case where small differential settlements could be a problem, such as in a rigid-arch bridge or long-span bridge on highly loaded piers, settlement analyses can be performed using modulus reduction factors which relate intact core modulus to rock mass modulus (see section 5.2, Settlement).

In addition, tolerable movements of most bridge abutments are relatively high compared to the 0.5 in criteria. A recent report for the National Cooperative Highway Research Program (1983) investigated tolerable settlements for bridge foundations. It was found that most transportation agencies consider 1 inch of settlement to be a tolerable criteria, and a few found 2 inches to be acceptable. The report concluded that these criteria were conservative and that 2 to 4 inches of differential vertical movements, depending on span length, were acceptable, provided that approach slabs or other provisions are made to minimize the effects of differential settlement between the approach embankment and the abutment. Thus, the proposed design bearing capacity procedures for footings on rock include limits for settlement which are much lower than the tolerable movement capacity of most bridges.

Table 18. Recommended allowable bearing pressures for footings on rock.

Material	Allowable Contact Pressure
Crystalline Bedrock, including granite, diorite, gneiss, traprock; and hard limestone, and dolomite, in sound condition:	
RQD = 75 to 100 percent	120 tsf
RQD = 50 to 75 percent	65 tsf
RQD = 25 to 50 percent	30 tsf
RQD = 0 to 25 percent	10 tsf
Foliated rocks, such as schist or slate; and bedded limestone, in sound condition:	
RQD $\geq$ 50 percent	40 tsf
RQD $\leq$ 50 percent	10 tsf
Sedimentary rocks, including hard shales and sandstones, in sound condition:	
RQD $\geq$ 50 percent	25 tsf
RQD $\leq$ 50 percent	10 tsf
Soft or broken bedrock (excluding shale), and soft limestone:	
RQD $\geq$ 50 percent	12 tsf
RQD $\leq$ 50 percent	8 tsf
Soft shale	4 tsf

5.6 CONSTRUCTION CONSIDERATIONS

Field Observations by Designer

An important step in the design/construction process for footings on rock is the observation of the exposed rock bearing surface by the designer or his representative, to ensure that rock conditions are as anticipated in the design. Footing bearing surfaces should have all loose, fragmented or weathered rock removed by jetting with high pressure air or water and should be clean and relatively dry. The designer or his representative should look for defects such as those described in section 5.4, and if necessary, modifications should be made to the foundation design or to footing bearing pressures.

If settlements were a concern during design, as for heavily loaded piers for long span or rigid arch bridges, field monitoring of performance may be required. The measured settlements should be checked against those predicted, and again design changes should be made if necessary.

Rock Excavation and Construction Methods

If the bedrock surface is relatively unweathered and not excessively fractured, and if the bridge design permits, the footing design should permit the elevation of footings to be adjusted in the field so that rock excavation is not necessary.

Where rock excavation by blasting is required to reach footing bearing elevation, the blasting procedures often result in overbreak below the design bearing level, and fracturing and opening of joints in the rock. The specifications generally provide for the contractor to remove all loose and fractured rock below bearing level and replace it with lean concrete. Often there is significant overbreak and significant quantities of fractured rock below bearing grade, resulting in a claim by the contractor for extra compensation because of "changed conditions."

Where these conditions are anticipated, alternative construction procedures may be specified for lightly loaded footings, such as for bridge abutments. One procedure involves backfilling the overbreak with gravel instead of lean concrete below the lightly loaded footings. Another alternative would utilize a heavy, smooth drum vibratory roller to compact any loose rock below subgrade level, followed by a choker course of crushed stone to prevent piping of soil into voids in the blasted rock.

For abutments which also serve as earth retaining structures, the use of crushed stone or gravel below footings on rock has the added advantage of allowing the walls to yield inward slightly by rotation, thus reducing lateral soil pressures on the wall to the active condition.

Dewatering

Design studies should have identified the natural groundwater levels within the soil and rock at the proposed bridge. If the excavation for footings is to be carried below the water table, provisions should be made in advance by the contractor for dealing with water inflow. Footing bearing surfaces should be kept clean and dry until the footings are concreted. This can usually be accomplished by pumping from sumps within the excavation. If large water inflows are expected or encountered in the field, deep wells into rock may be required to draw down the groundwater table during construction.

6. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

6.1 INTRODUCTION

This research study of bridge foundation performance was based on analysis of predicted and observed settlement of 10 case study bridges where foundations were supported on cohesionless sand. The results of field observations, including descriptions of monitoring equipment and procedures are addressed in section 2. Methods for prediction of settlement for foundations on sand were selected from existing engineering procedures; evaluations of the methods are contained in section 3.

An introduction to risk-based analyses for geotechnical design problems is presented in section 4 while recommendations for design of footings on rock are outlined in section 5.

6.2 FOUNDATION COST COMPARISON

Where technically feasible, the use of spread footing foundations for support of bridges represents an economical design choice. To confirm this accepted concept, a foundation cost comparison was completed for three bridge projects representing a range of highway bridge pier loads. The bridges were selected to be representative of the range in pier loads which could be encountered on a typical project.

To complete the comparison, a foundation design was prepared for each bridge utilizing both spread footings and end bearing piles. Each bridge was actually designed and built as a pile supported structure. Therefore basic pile cap size, number of piles and design pier loads were known. In the comparison, each site was assumed to be underlain by a medium dense sand with a standard penetration resistance, "N", of about 15 blows per ft. At this N value, extreme settlement due to liquefaction would not be a problem. However, the designer would have to choose between either a pile or a spread footing foundation based on comparative cost and settlement analyses.

For the pile alternative, pile cap size and number of piles were not changed. The length, however, of the piling was increased with increased depth of sand, but the pile length was limited to the depth where pile penetration would be stopped under normal pile driving criteria. The data for each site are summarized in table 19.

The shallow foundation alternate to deep piling was evaluated for each bridge for the same set of soil conditions including N-value and range of sand thickness. For each bridge the footing dimensions were based on limiting settlement to acceptable levels using predictive methods discussed in section 3. Basically the total predicted settlement was limited to 1.0 in and the resulting allowable bearing stress was used to size the footings.

Table 19. Summary of site data for three study bridges.

Project Site	Location	No. of Lanes	Pier Load (kips)	No. of Piles	Pile Type	Pile Unit Cost ($/LF)*
Mass. Rt. 31 over Rt. 2	Fitchburg MA	2	1,700	20	10 3/4 in X 0.438 in	$24
Rouses Point Bridge	Rouses Point NY/VT	4	4,606	10 vertical 18 battered	HP 12 X 74	$24
Charter Oak	Hartford CT	6	17,300	73	HP 14 X 89	$29

* Concrete costs for pile caps and footing were estimated at $165/cu yd.

Since footing settlement in sand is controlled primarily by the compressibility of the portion of the deposit (a depth of B to 4B below the footing), an increasingly thick sand deposit beyond a depth of 4B does not result in rapidly increasing footing costs.

The results are illustrated in figure 53 and clearly indicate increasing costs as depth to firm bearing is increased. However, pile cost increases more rapidly with increasing thickness of sand.

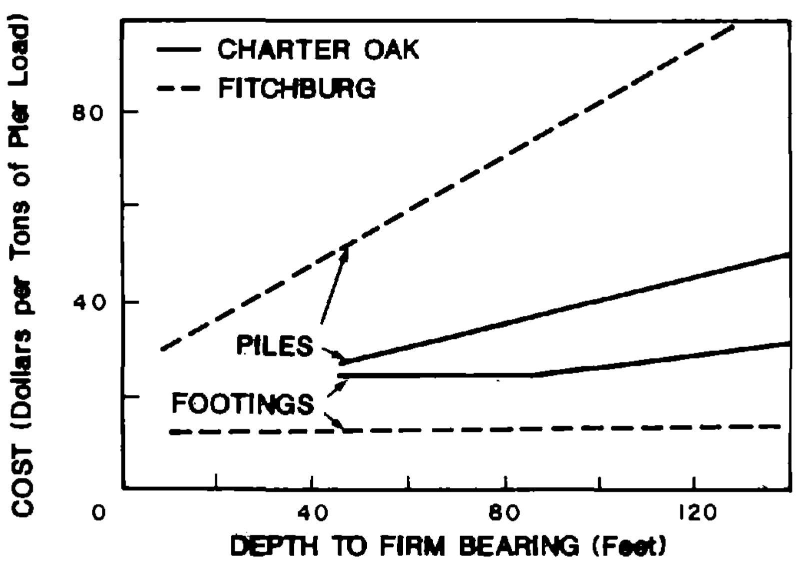


Figure 53. Foundation cost per ton versus depth to firm bearing.

The premium cost of deep foundations was then evaluated. Premium cost was defined as the difference in cost between a deep foundation system and a conventional spread footing foundation. The premium cost relationships shown in figure 54 were developed from the alternate foundation studies of the three bridge sites. From the figure it is concluded that:

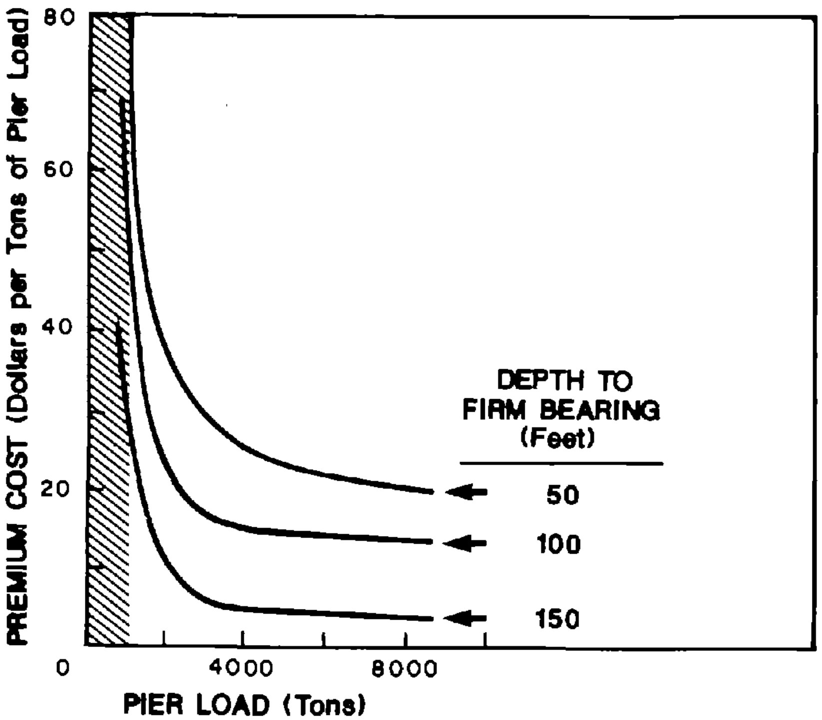


Figure 54. Relation of premium foundation cost and pier load.

- For bridge pier loads less than about 1,000 tons, the premium costs increase very rapidly.

- For larger structures with pier loads greater than about 2,000 tons the premium cost is relatively constant for a given depth to firm bearing.

These observations suggest that the premium foundation cost associated with larger structures is not as cost sensitive a choice when compared to the selection of foundations for "average" structures. For the typical highway overpass bridge, abutment and pier loads are generally less than 1,000 tons. In this load range a significant premium cost savings, as illustrated in figure 55, will be achieved if spread footings are selected over piles. Thus on the average project, every effort should be made to utilize spread footings.

In order to justify use of spread footings versus piles, the designer must predict footing settlement. The settlement prediction methods outlined in section 3 of this report, and particularly the D'Appolonia and Burland and Burbidge methods, should be utilized to estimate total footing settlement. However, it is most important that the predicted settlement be evaluated in terms of the time rate of settlement. The performance data (figure 12) shows that only about 25 percent of predicted settlement can be expected as post deck settlement. Thus by utilizing the available settlement prediction procedures and by comparison with observed bridge foundation settlement, the bridge designer can justify reasonable settlement performance, the use of spread footing foundations and therefore significantly reduce the project cost.

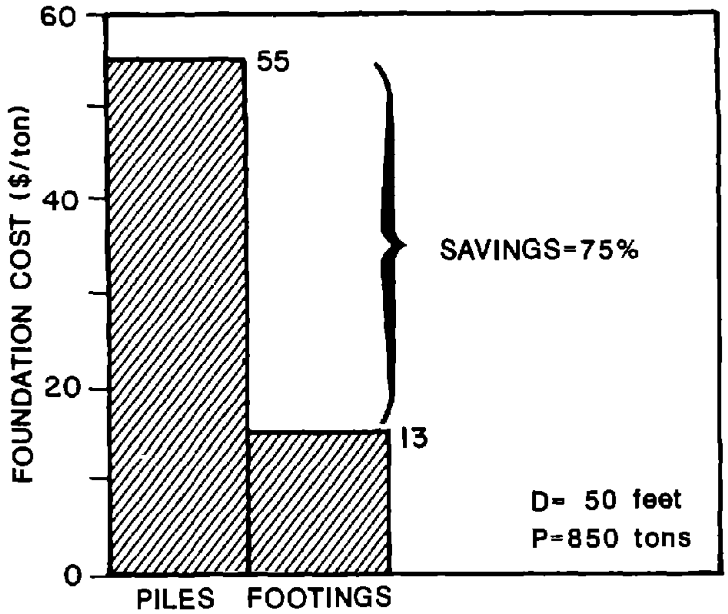


Figure 55. Cost savings for footings versus piles.

6.3 CONCLUSIONS

Each of the areas of the research study involved evaluation of different aspects of foundation design and practice for bridge foundations. Descriptions of activities and conclusions are contained in the appropriate sections of the report. Pertinent conclusions from each of the areas of interest are summarized below.

Monitoring Bridge Foundation Performance:

Actual footing settlement data has traditionally been used to develop empirical settlement prediction methods for foundations on sand. During the study, instrumentation plans were developed and a monitoring program was conducted to document bridge foundation settlement behavior for a period of several years. With regard to the instrumentation monitoring program it is concluded that:

- The measurement of basic settlement and tilt of the bridge foundation should be obtained following the "partial instrumentation" plans outlined in section 2.0.

- Careful coordination during design and construction must be established. In particular, personnel assigned to the monitoring of bridge performance must participate from beginning to end of the project. Communications from the project site must be established, for example, so that settlement points can be installed monitored, transferred etc. as construction proceeds. The points or other instruments will be lost, buried or ignored during the construction activities, unless the site staff from both engineering and contractor office actively communicates construction schedules to the monitoring staff.

- o About 75 percent of the observed bridge foundation settlement occurs within the construction period prior to placement of bridge deck.

- o For the typical bridges observed in this study, post deck settlement was less than 0.25 in.

Settlement Methods and Analyses for Bridge Footings on Sand:

Many methods for prediction of settlement of footings on sand have been proposed. Five methods were selected for evaluation in this study based on acceptance within general foundation engineering practice. Based on a review and application of the five selected methods it is concluded that:

- o No method as proposed by the original author deals with all aspects of foundation settlement predictions; aspects such as preloaded or normally consolidated sand, non-uniform contact bearing stress, overlapping stress from adjacent footings, groundwater and embedment effects are treated differently or ignored by each method.

- o Application of the selected methods by using reasonable interpretations of the procedures can predict footing settlement within about 0.4 in on average. Larger error can occur for individual calculations.

- o Some methods tend to overpredict while others underpredict the settlement. Based on the study data, the D'Appolonia method was most accurate followed by Burland and Burbidge.

Risk Based Methods in Geotechnical Design:

The propagation of errors or uncertainty through engineering calculations will lead to a range in the final predicted quantity. The procedures outlined in section 4 show that:

- o Methods of analyses are available to evaluate sources of uncertainty in terms of spatial variation, measurement noise, statistical error and bias.

Design of Spread Footings on Rock:

Traditionally, rock has been regarded as the best foundation bearing material. Design of foundations should consider the following:

- o Design of foundations on rock should be based on consideration of the discontinuities in the rock mass as well as the geologic classification (strength) of the intact rock mass.

- Construction methods such as drilling and blasting causing fractured rock placement of gravel or crushed stone fill over rock for grading or drainage purposes can limit bearing stresses to well below that of the rock mass.

6.4 RECOMMENDATIONS

Based on the experience gained through this study of bridge foundation settlement, the following recommendations have been prepared:

Bridge foundation performance monitoring programs should be implemented and the data stored to provide an enlarged statistical basis to judge the effectiveness of settlement predictions and the satisfactory performance of bridge foundations on sand.

Field explorations for bridges should include continuous sampling of the influence zone below the footing. Normally a depth of 2B below the footing should be more intensively tested, particularly when obtaining standard penetration test data. Reliable groundwater level data must also be obtained.

Footing design on sand should be evaluated in terms of settlement predicted by using two or more of the design procedures outlined in section 3. The range of settlement and level of confidence in the predictions must be evaluated by an engineer familiar with site conditions and limitations of the predictive methods. Acceptable settlement limits should be established based on considerations of the time-settlement behavior for sand. For example, relatively small "post-deck construction" settlement of about 25 percent of the total settlement should be used for long term settlement comparisons.

Design of spread footing bridge foundations on rock must consider the geologic classification of the rock mass effect of joints and discontinuities in the rock mass. Discontinuities may be natural or may include surface irregularities due to use of conventional rock blasting methods. Recommended design bearing capacity values are presented in section 5.

For larger complex projects where design issues such as bearing capacity, settlement or slope stability may be repeatedly analyzed for different soil parameters, or imposed loadings, the "propagation of error" through the calculations and the assessment of the risk of failure should be evaluated. The methodology outlined in section 4 and in FHWA RD86/010 is recommended.

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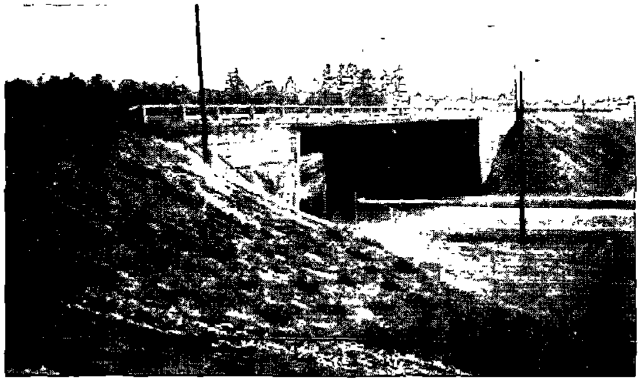
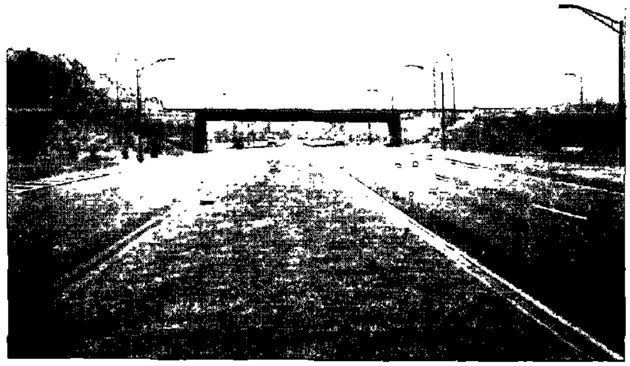
APPENDIX A: SUMMARY OF CANDIDATE BRIDGES

CANDIDATE BRIDGE NUMBER: M5000 **LOCATIONS:** NORTH AVENUE SIDELINE OVER VT 127
BURLINGTON, VERMONT

HISTORICAL BACKGROUND:

GEOLOGICAL SETTING: LACUSTRINE DEPOSITS FROM GLACIAL LAKE CHAMPLAIN, SOME VARVES INDICATED IN BORINGS, FINE SANDS INTERLAYERED WITH SILT





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0' - 12'	MEDIUM-FINE SAND	28 - 92	56	--
12' - 22'	SANDY SILT A-4	6 - 50	25	
22' - 37'	FINE SAND A-3	5 - 22	12	
37' - 57'	SANDY SILT A-4	5 - 8	6	
57' - 77'	FINE SAND A-3	8 - 30	22	
77' - 129'	SANDY SILT A-4	14 - 79	41	
129'	BEDROCK			

B. OTHER SUBSURFACE INFORMATION

NUMBER OF BORINGS: 8
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 10 TO 25 FEET
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 12 FEET
REMARKS: COMPRESSIBLE SILT STRATA BENEATH GRANULAR SOILS

2. **BRIDGE DESIGN DATA:**

NUMBER OF SPANS: ONE
SPAN LENGTH: 128 FT.
DESIGN BEARING PRESSURE: 4 KSF

SPECIAL FEATURES: SURCHARGE EMBANKMENT
REMARKS: 30 DAY SURCHARGE EMBANKMENT. 10-20 FEET HIGH OVER SILT. BOTTOM OF EMBANKMENT IS 12 FT. BELOW FOOTINGS.

3. **BRIDGE CONSTRUCTION DATA**

SURCHARGE 11/83 - 11/84
BEGIN CONSTRUCTION: 4/85
FOOTINGS COMPLETED: A1 - 5/85
A2 - 5/85
ABUTMENT WALL COMPLETION: 6/85
WING WALL COMPLETION: 6/85

BACKFILL COMPLETE: 6/85
STRUCTURE COMPLETE: 8/85
PAVEMENT COMPLETE: 9/85
OPEN TO TRAFFIC: 9/85
REMARKS:

4. **INSTRUMENTATION SYSTEM:** FULL SCALE

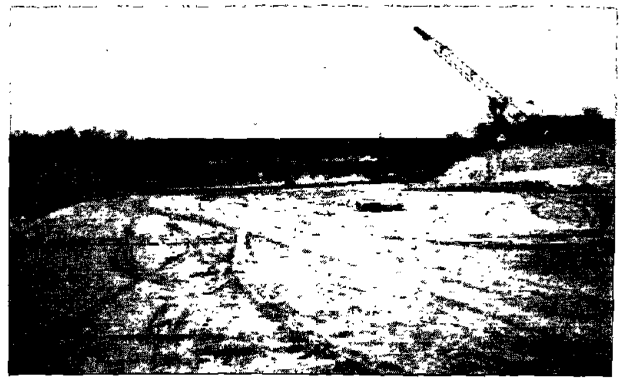
SETTLEMENT POINTS: 16
SETTLEMENT PLATFORMS: 14
TILT/OVERTURNING POINTS: 8

DEEP SETTLEMENT/STRAIN: 1
REMOTE SETTLEMENT/PROFILE: 3
CONTACT STRESS: 10
APPLIED LOADING: YES

CANDIDATE BRIDGE NUMBER: 131-132-11 LOCATIONS: I-691 UNDER RELOCATED DICKERMAN ROAD SOUTHINGTON/CHESHIRE, CONNECTICUT

HISTORICAL BACKGROUND: ______

GEOLOGICAL SETTING: GLACIAL OUTWASH SANDS OVER ARKOSIC SANDSTONE



1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0 - 35'	FINE SAND AND SILT	7 TO 24	16	
35' +	ARKOSIC SANDSTONE			

B. OTHER SUBSURFACE INFORMATION

NUMBER OF BORINGS: 9
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 35 FT.
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 0.5 TO 6.0 FT.
REMARKS: ______

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: TWO
SPAN LENGTHS: 112 FT.
DESIGN BEARING PRESSURE: 5.0 KSF

SPECIAL FEATURES: PRESTRESSED/PRETENSIONED GIRDERS
REMARKS: ______

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 10/84
FOOTINGS COMPLETED: A1 - 12/84
A2 - 11/84
ABUTMENT WALL COMPLETION: A1 - 3/85
A2 - 12/84
WING WALL COMPLETION: ______
PIER COMPLETION: 12/84

BACKFILL COMPLETE: A1 - 11/85, A2 - 3/85
STRUCTURE COMPLETE: 11/85
PAVEMENT COMPLETE: ______
OPEN TO TRAFFIC: ______
REMARKS: NOT COMPLETE AS OF 6/86

4. INSTRUMENTATION SYSTEM: FULL SCALE

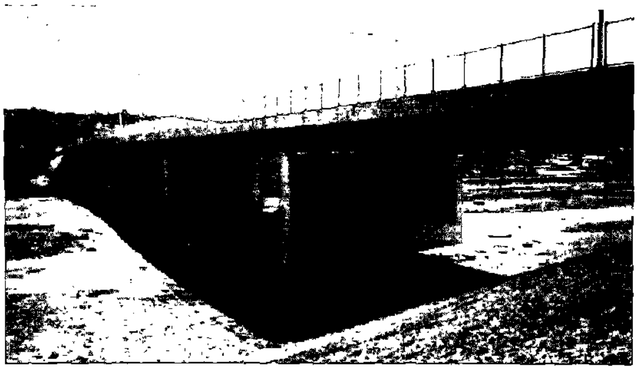
SETTLEMENT POINTS: 13
TILT/OVERTURNING POINTS: 10
REMOTE SETTLEMENT/PROFILE: 3
CONTACT STRESS: 9

APPLIED LOADING: YES
SETTLEMENT PLATFORMS: NOT APPLICABLE
DEEP SETTLEMENT/STRAIN: NOT APPLICABLE

CANDIDATE BRIDGE NUMBER: 931 LOCATIONS: BRANCH AVENUE, NORTHEAST CORRIDOR
PROVIDENCE, RHODE ISLAND

HISTORICAL BACKGROUND: REPLACEMENT OF 1910 VINTAGE STEEL BRIDGE ACROSS AMTRACK NORTHEAST CORRIDOR

GEOLOGICAL SETTING: GLACIAL OUTWASH PLAIN



1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0' - 15'	MEDIUM-FINE SAND	16 - 38	24	64
15' - 140'	FINE SAND (OUTWASH)	9 - 88	48	115
140' - 155'	GLACIAL TILL	100+		--
155	BEDROCK			--

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS: 13
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 20 TO 30 FT.
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 0 AS BUILT - 18'
REMARKS: PRECONSOLIDATED SAND

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: FOUR
SPAN LENGTHS: 120 FT.±
DESIGN BEARING PRESSURE: 4.0 KSF

SPECIAL FEATURES: 74 INCH SEWER UNDER CENTERLINE. BRIDGE FOOTING ON BOTH SIDES
REMARKS: BRIDGE ACROSS RAILROAD

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 10-83
FOOTINGS COMPLETED: A1 - 11/83
A2 - 5/84
ABUTMENT WALL COMPLETION: A1 - 11/83
A2 - 6/84
WING WALL COMPLETION: A1 - 12/83
A2 - 6/84

BACKFILL COMPLETE: A1 - 1/84, A2 - 7/84
STRUCTURE COMPLETE: 8/84
PAVEMENT COMPLETE: 11/84
OPEN TO TRAFFIC: 11/84
REMARKS: ____________

4. INSTRUMENTATION SYSTEM: FULL SCALE

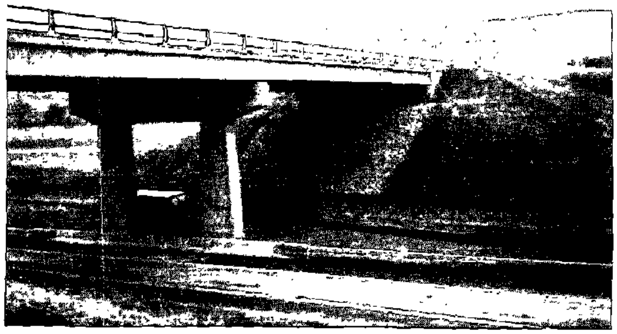
SETTLEMENT POINTS: 42
TILT/OVERTURNING POINTS: 12
CONTACT STRESS: 9
REMOTE SETTLEMENT/PROFILE: 3

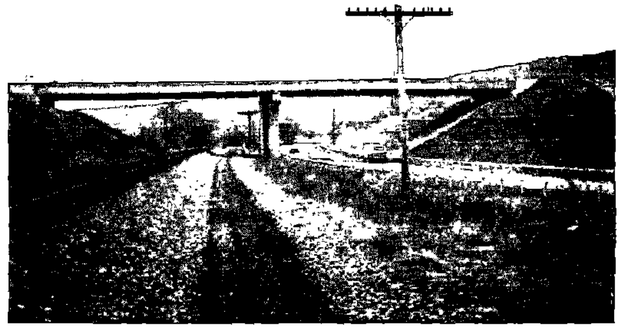
APPLIED LOADING: YES, 2 PLACES
REMARKS: ____________

CANDIDATE BRIDGE NUMBER: 5   LOCATIONS: RELOCATED GERSONI RD. - ROUTE 28 OVER D&H RR & ROUTE 7, COLLIERSVILLE, NY

HISTORICAL BACKGROUND: ______________________

GEOLOGICAL SETTING: QUARTENARY ALLUVIUM OVER SILT





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0 TO 20 FT.	COARSE-FINE SAND/ SILTY SAND	2 - 37	17	55
20 TO 45 FT.	SILTY FINE SAND	10 - 44	24	48
45 TO 165+	SILT	3 - 13	6	--

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS: 5
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 20 TO 45 FT.
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 25 FT.
REMARKS: COMPRESSIBLE SILT STRATA BENEATH GRANULAR SOILS

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: TWO
SPAN LENGTHS: 112 FT.
DESIGN BEARING PRESSURE: 5.0 KSF

SPECIAL FEATURES: SURCHARGE NORTH ABUTMENT
REMARKS: ______________

3. BRIDGE CONSTRUCTION DATA

SURCHARGE 10/83
BEGIN CONSTRUCTION: 3/84
FOOTINGS COMPLETED: N: 4/84
S: 4/84
ABUTMENT WALL COMPLETION: N: 5/84
S: 4/84
WING WALL COMPLETION: 5/84
PIER COMPLETION: 6/84

BACKFILL COMPLETE: 5/84
STRUCTURE COMPLETE: 9/84
PAVEMENT COMPLETE: 10/84
OPEN TO TRAFFIC: 11/84
REMARKS: ______________

4. INSTRUMENTATION SYSTEM: FULL SCALE

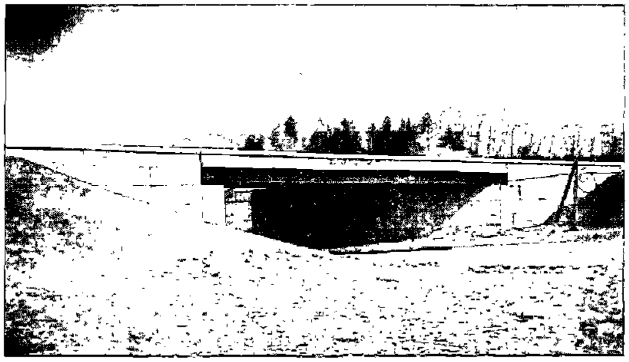
SETTLEMENT POINTS: 18
SETTLEMENT PLATFORMS: 3
DEEP SETTLEMENT/STRAIN: 1
TILT/OVERTURNING POINTS: 15

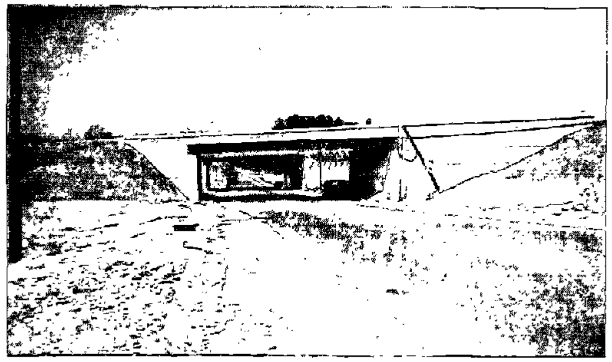
REMOTE SETTLEMENT/PROFILE: 2
CONTACT STRESS: 4
APPLIED LOADING: YES
REMARKS: ______________

CANDIDATE BRIDGE NUMBER: U-2-59 LOCATIONS: ROUTE 146 SOUTHBOUND OVER RELOCATED LACKEY DAM ROAD: UXBRIDGE, MA

HISTORICAL BACKGROUND: ROUTE 146 WIDENED TO 4 LANES - HISTORY OF MANY ACCIDENTS AT FORMER 2 LANE BRIDGE

GEOLOGICAL SETTING: GLACIAL OUTWASH PLAIN





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	AVERAGE	AVERAGE CONE RESISTANCE
0 TO 55 FT.	COARSE TO FINE SAND	10 - 47	27	--
55 +	GLACIAL TILL	100+	--	--

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS: 8
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 45 TO 55 FT.
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 4 TO 5.5 FT. PROPOSED, NONE AS BUILT
REMARKS: ______

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: ONE
SPAN LENGTHS: 112 FT.
DESIGN BEARING PRESSURE: 5.0 KSF MAX. TOE PRESSURE

SPECIAL FEATURES: ______
REMARKS: ______

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 11/84
FOOTINGS COMPLETED: N: 11/83
S: 12/83
ABUTMENT WALL COMPLETION: N: 1/84
S: 1/84
WING WALL COMPLETION: 1/84

BACKFILL COMPLETE: N: 5/84, S: 1/84
STRUCTURE COMPLETE: 6/84
PAVEMENT COMPLETE: 11/84
OPEN TO TRAFFIC: 11/84
REMARKS: ______

4. INSTRUMENTATION SYSTEM: FULL SCALE

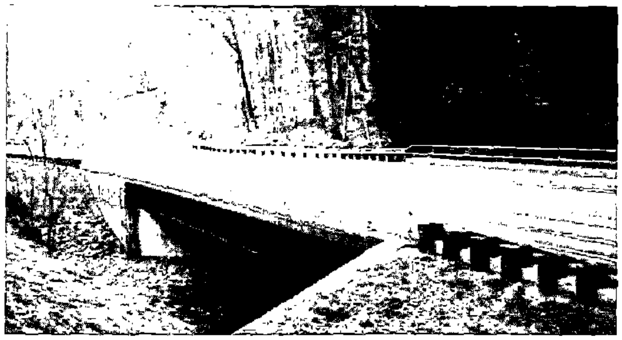
SETTLEMENT POINTS: 20
TILT/OVERTURNING POINTS: 12
REMOTE SETTLEMENT/PROFILE: 3

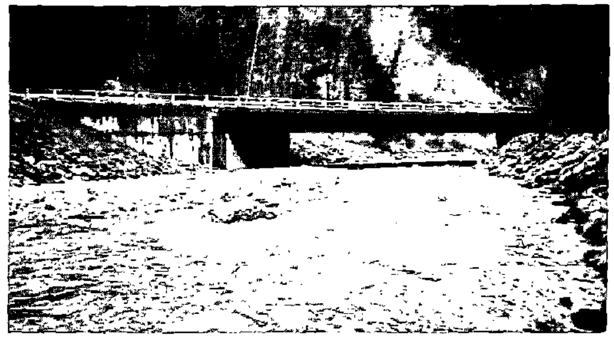
CONTACT STRESS: 9
APPLIED LOADING: YES
REMARKS: ______

CANDIDATE BRIDGE NUMBER: 45   LOCATIONS: VERMONT ROUTE 11 OVER THE MIDDLE BRANCH - WILLIAMS RIVER
CHESTER, VERMONT

HISTORICAL BACKGROUND: RIVER NOTED FOR RAPID RISE AND FALL OF WATER LEVEL.
FOURTH BRIDGE AT THIS LOCATION; SECOND BRIDGE DESTROYED IN 1927 FLOOD;
THIRD BRIDGE BUILT IN 1928 WAS OBSOLETE.

GEOLOGICAL SETTING: 100 FOOT HIGH EMBANKMENT OF LACUSTRINE SILT 100 YARDS DOWNSTREAM
20 FOOT HIGH OUTCROP OF MICA SCHIST 100 YARDS UPSTREAM.





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0' - 20'	SILTY SAND/SILT	18 - 171	95	--

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS 4
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 20 FEET ±
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 3 FEET
REMARKS: BOULDERS AND COBBLES MADE DRIVING OF SHEET PILING DIFFICULT.

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: ONE
SPAN LENGTH: 115 FEET
DESIGN BEARING PRESSURE: 1.5 KSF

SPECIAL FEATURES: ______
REMARKS: ______

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 6/83
FOOTINGS COMPLETED: A1 - 7/83
A2 - 8/83
ABUTMENT WALL COMPLETION: A1 - 8/83
A2 - 9/83
WING WALL COMPLETION: SAME AS ABUTMENTS

BACKFILL COMPLETE: 9/83
STRUCTURE COMPLETE: 9/83
PAVEMENT COMPLETE: 10/83
OPEN TO TRAFFIC: 11/83
REMARKS:: ______

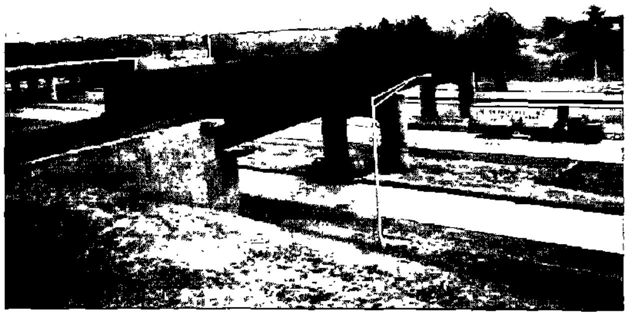
4. INSTRUMENTATION SYSTEM: PARTIAL

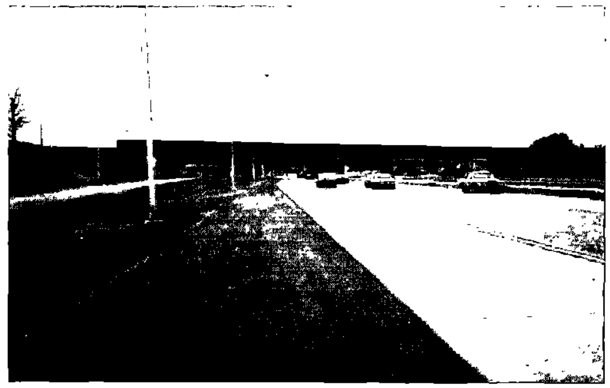
24 SETTLEMENT POINTS
6 TILT/OVERTURNING POINTS

CANDIDATE BRIDGE NUMBER: 76-88-7   LOCATIONS: I-86
MANCHESTER, CONNECTICUT

HISTORICAL BACKGROUND: ______________________

GEOLOGICAL SETTING: VARVED GLACIAL OUTWASH SANDS OVER ARKOSIC SANDSTONE





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0-90+	COARSE TO FINE SAND	8 - 59	29	67

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS: 9
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 90 FEET
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: A-2 - 2 FEET
REMARKS: ______________________

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: FOUR
SPAN LENGTHS: 114 FT., 132 FT., 162 FT., & 174 FT.
DESIGN BEARING PRESSURE: 6.0 KSF

SPECIAL FEATURES: ______________
REMARKS: RAILROAD BRIDGE CROSSING I-86

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 10/83
FOOTINGS COMPLETED: A1 - 10/83
A2 - 11/83
ABUTMENT WALL COMPLETION: A1 - 11/83
A2 - 12/83
WING WALL COMPLETION: 12/83
PIER COMPLETION: 3/84

BACKFILL COMPLETE: 12/83
STRUCTURE COMPLETE: 9/84
PAVEMENT COMPLETE: 9/84
OPEN TO TRAFFIC: 10/84
REMARKS: ______________

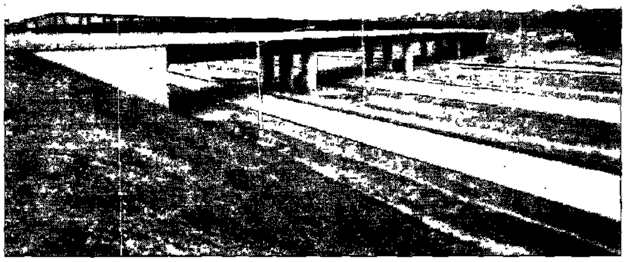
4. INSTRUMENTATION SYSTEM: PARTIAL

SETTLEMENT POINTS: 8
TILT/OVERTURNING POINTS: 3

CANDIDATE BRIDGE NUMBER: 76-88-8 LOCATIONS: I-86 AND CD ROADWAY UNDER TOLLAND TURNPIKE
MANCHESTER, CONNECTICUT

HISTORICAL BACKGROUND: ______________________________

GEOLOGICAL SETTING: VARVED GLACIAL OUTWASH SANDS



1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0 - 5 FT.	COARSE TO FINE SAND	19 - 43	31	55
5 - 45 FT.	MEDIUM SAND	9 - 62	38	142

B. OTHER SUBSURFACE INFORMATION

NUMBER OF BORINGS: 8
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 45 +
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 5
REMARKS: ______________________________

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: FOUR
SPAN LENGTHS: 145, 220, 220, 175 FT.
DESIGN BEARING PRESSURE: 6.14 KSF

SPECIAL FEATURES: ______________
REMARKS: SUPERSTRUCTURE DESIGNED FOR POST-CONSTRUCTION DIFFERENTIAL SETTLEMENT BETWEEN ADJACENT SUPERSTRUCTURE UNITS

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 6/83
FOOTINGS COMPLETED: A1 - 1/84
A2 - 7/83
ABUTMENT WALL COMPLETION: A1 - 2/84
A2 - 7/83
WING WALL COMPLETION: SAME AS ABUTMENTS
PIER COMPLETION: 2/84

BACKFILL COMPLETE: 7/84
STRUCTURE COMPLETE: 9/84
PAVEMENT COMPLETE: 11/84
OPEN TO TRAFFIC: 11/84
REMARKS: ______________

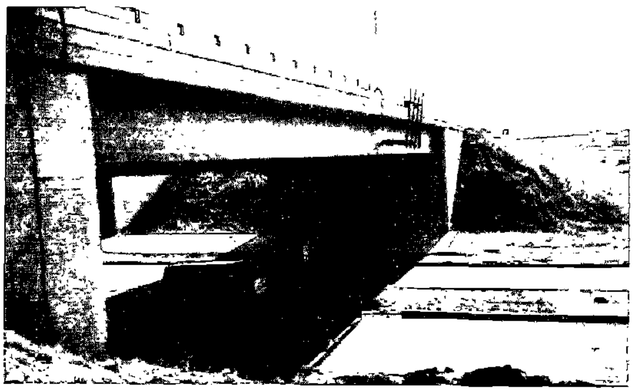
4. INSTRUMENTATION SYSTEM: PARTIAL

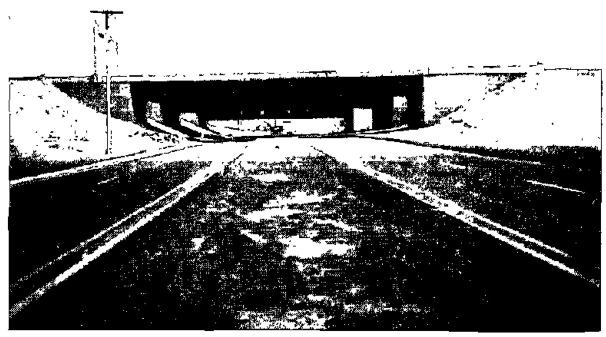
SETTLEMENT POINTS: 25
TILT/OVERTURNING POINTS: 8

CANDIDATE BRIDGE NUMBER: 76-88-9 LOCATIONS: CD-WB ROADWAY & RAMP 1 OVER BUCKLAND ST. (I-86) MANCHESTER, CONNECTICUT

HISTORICAL BACKGROUND: ______

GEOLOGICAL SETTING: ______





1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	STANDARD PENETRATION RESISTANCE AVERAGE	AVERAGE CONE RESISTANCE
0 - 35	COARSE TO FINE SAND SOME SILT	20 - 81	44	--
35 +	DECOMPOSED ROCK	100+		--

B. OTHER SUBSURFACE INFORMATION

NUMBER OF BORINGS: 5
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 30 FEET
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 5 TO 10 FEET
REMARKS: ______

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: ONE
SPAN LENGTH: 146
DESIGN BEARING PRESSURE: 7.0 KSF

SPECIAL FEATURES: ______
REMARKS: ______

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: 8/83
FOOTINGS COMPLETED: 8/83
ABUTMENT WALL COMPLETION: 9/83
WING WALL COMPLETION: 10/83

BACKFILL COMPLETE: 11/83
STRUCTURE COMPLETE: 6/84
PAVEMENT COMPLETE: Prior to 11/85
OPEN TO TRAFFIC: Prior to 11/85
REMARKS: ______

4. INSTRUMENTATION SYSTEM: PARTIAL

SETTLEMENT POINTS: 20
TILT/OVERTURNING POINTS: 10

CANDIDATE BRIDGE NUMBER: 78-88-3 LOCATIONS: I-86 EB & WB & RAMPS A, J, & P UNDER MIDDLE TURNPIKE WEST, MANCHESTER, CONNECTICUT

HISTORICAL BACKGROUND: ______

GEOLOGICAL SETTING: ______

1. SUBSURFACE INFORMATION

A. TYPICAL SOIL PROFILE

DEPTH BELOW FOOTING	GENERALIZED SOIL DESCRIPTION	STANDARD PENETRATION RESISTANCE RANGE	AVERAGE	AVERAGE CONE RESISTANCE
0 - 10'	GRAVELY SAND	8 - 55	24	--
10 - 30'	FINE SAND & SILT	3 - 44	30	--
30' +	ARKOSIC SANDSTONE			

B. OTHER SUBSURFACE INFORMATION

NUMBER OF TEST BORINGS: 12
THICKNESS OF NATURAL GRANULAR SOIL BENEATH FOOTING: 28 FEET
THICKNESS OF STRUCTURAL FILL BENEATH FOOTING: 2 FEET TYPICAL
REMARKS: ______

2. BRIDGE DESIGN DATA:

NUMBER OF SPANS: FIVE
SPAN LENGTHS: 155, 160, 171, 180 & 195 FT.
DESIGN BEARING PRESSURE: 6.0 TO 8.0 KSF
SPECIFICATION FOR STRUCTURAL FILL ______

SPECIAL FEATURES: ______
REMARKS: SUPERSTRUCTURE DESIGNED FOR POST-CONSTRUCTION DIFFERENTIAL SETTLEMENT BETWEEN ABUTMENT NO. 2 AND PIER NO. 4.

3. BRIDGE CONSTRUCTION DATA

BEGIN CONSTRUCTION: Not Determined
FOOTINGS COMPLETED: Not Determined

ABUTMENT WALL COMPLETION: Not Determined

WING WALL COMPLETION: Not Determined

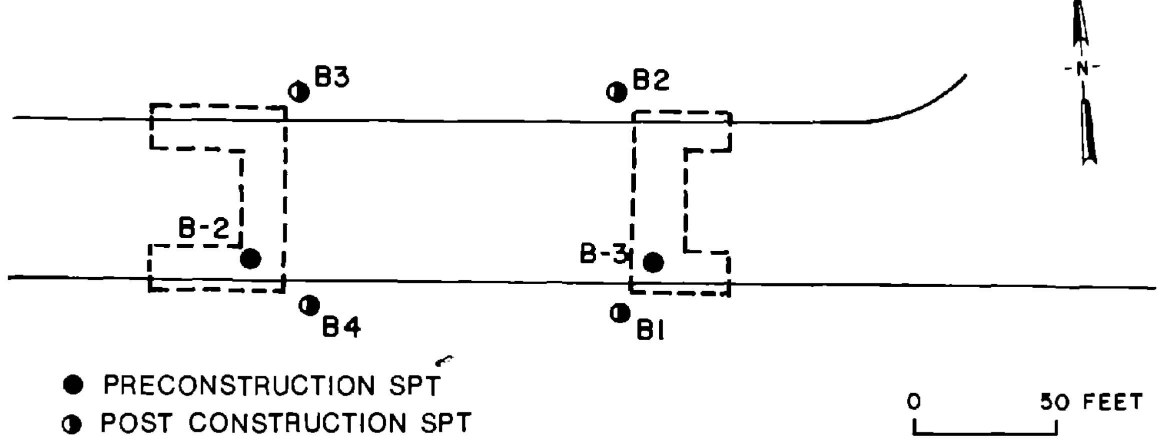
BACKFILL COMPLETE: Not Determined
STRUCTURE COMPLETE: Not Determined
PAVEMENT COMPLETE: Not Determined
OPEN TO TRAFFIC: 11/85
REMARKS: ______

4. INSTRUMENTATION SYSTEM: PARTIAL

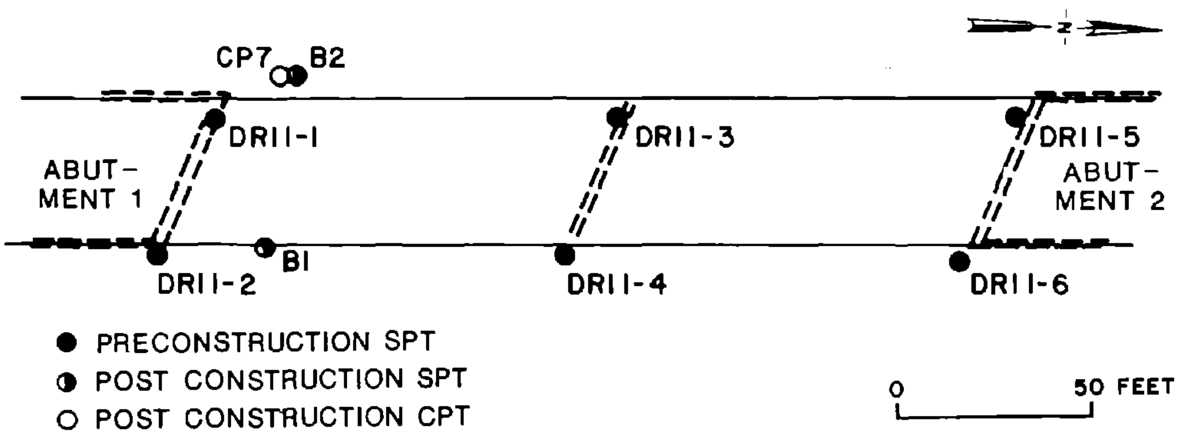
SETTLEMENT POINTS: 20

REMARKS: POST CONSTRUCTION SETTLEMENT ONLY

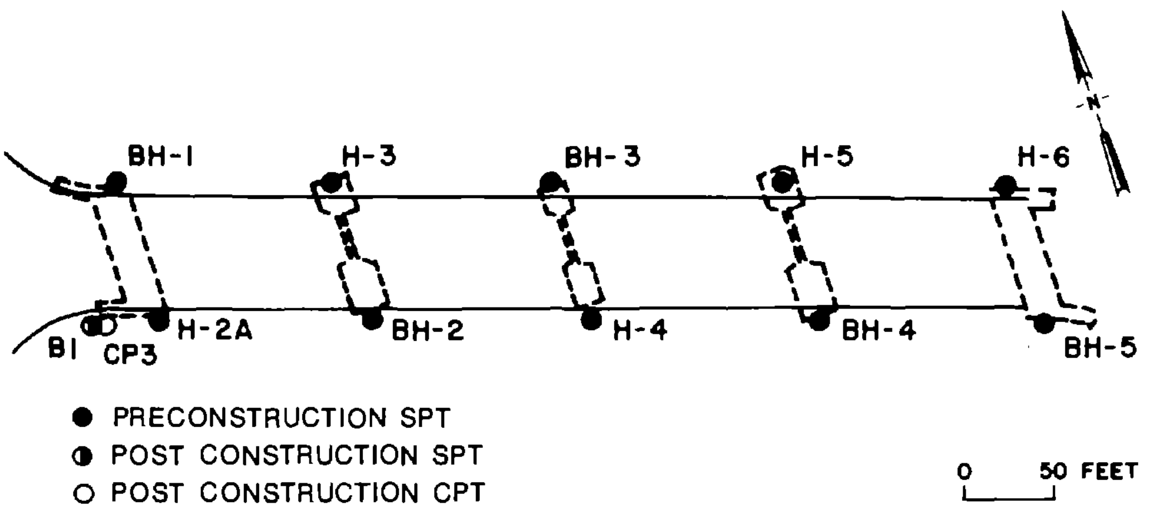
APPENDIX B: SUBSURFACE EXPLORATION LOCATION PLANS



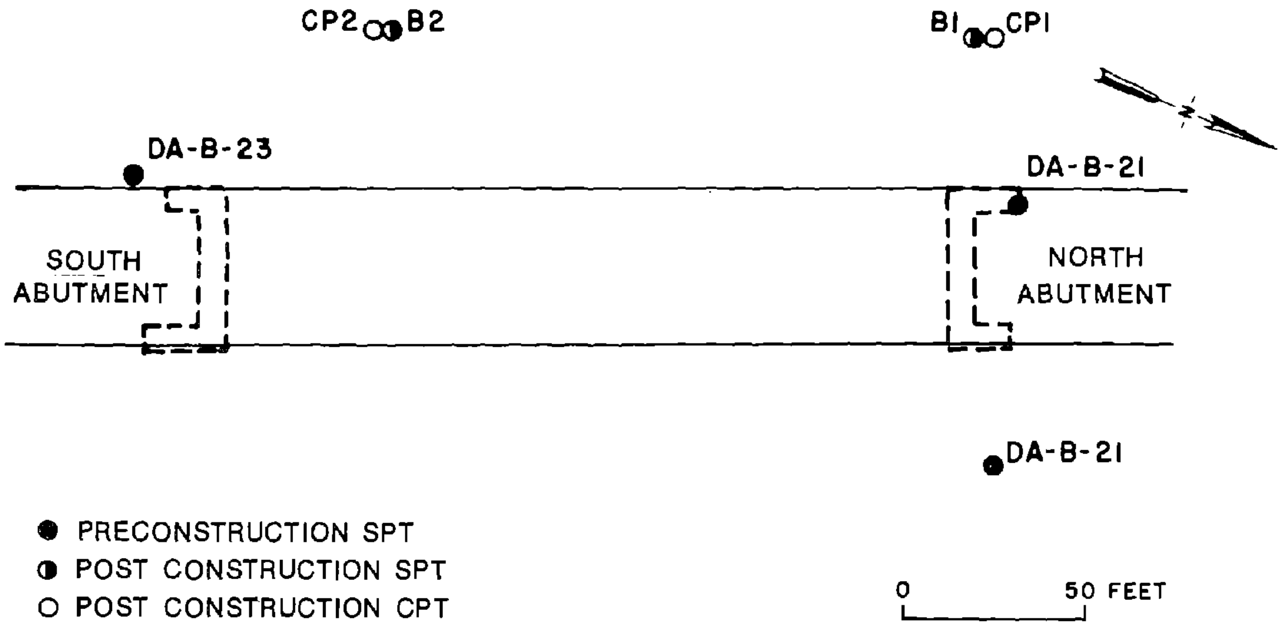
BRIDGE #1 (BURLINGTON, VT)



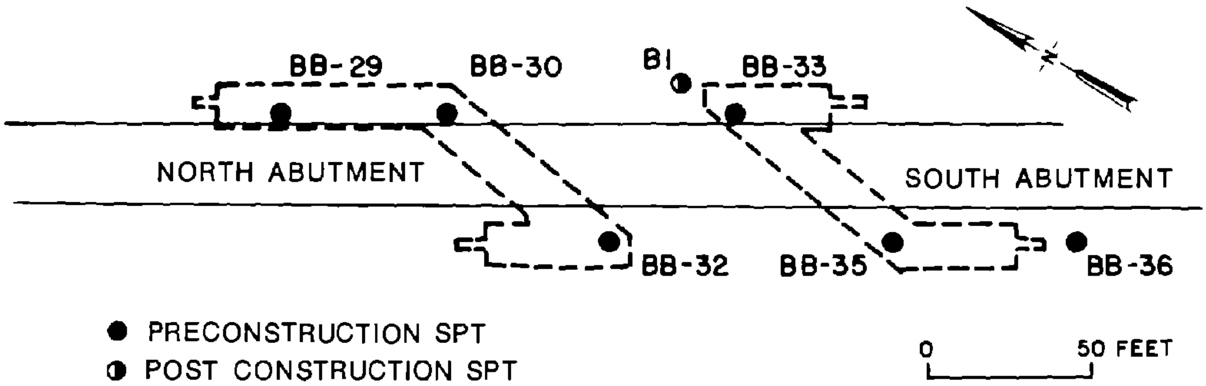
BRIDGE #2 (SOUTHINGTON, CT)



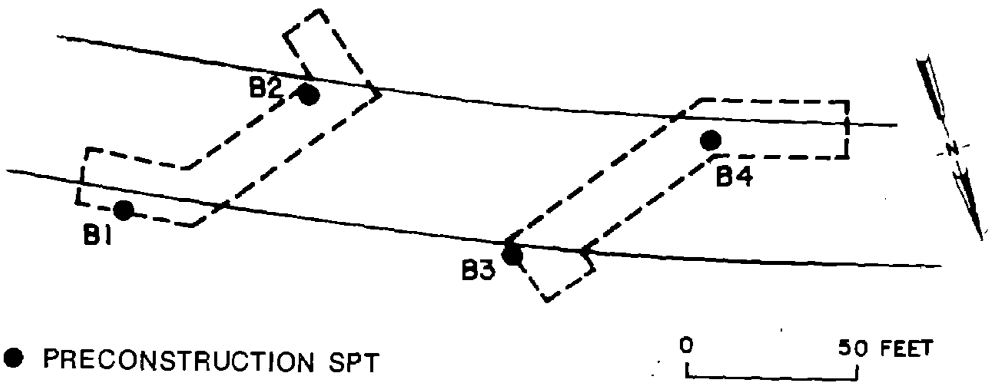
BRIDGE #3 (PROVIDENCE, RI)



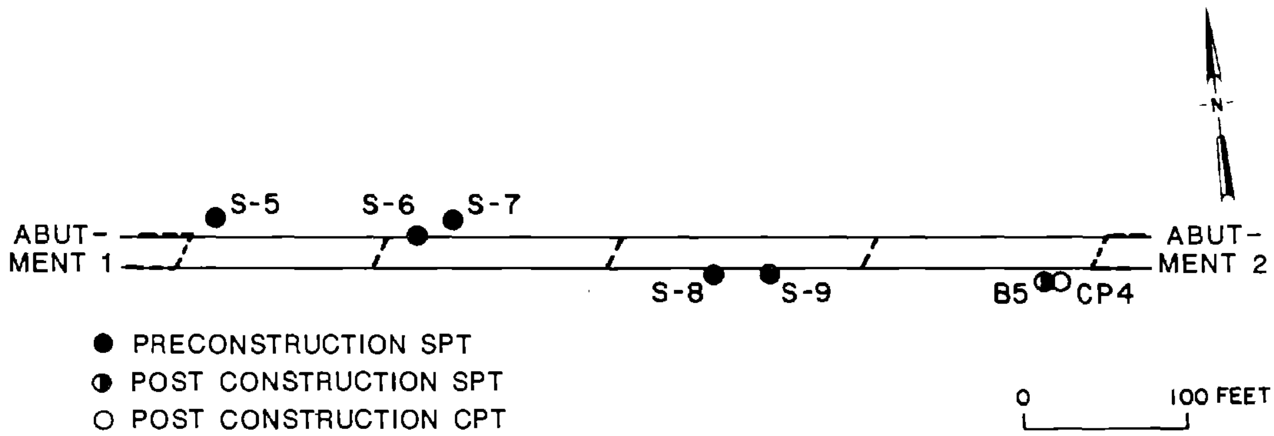
BRIDGE #4 (COLLIERSVILLE, NY)



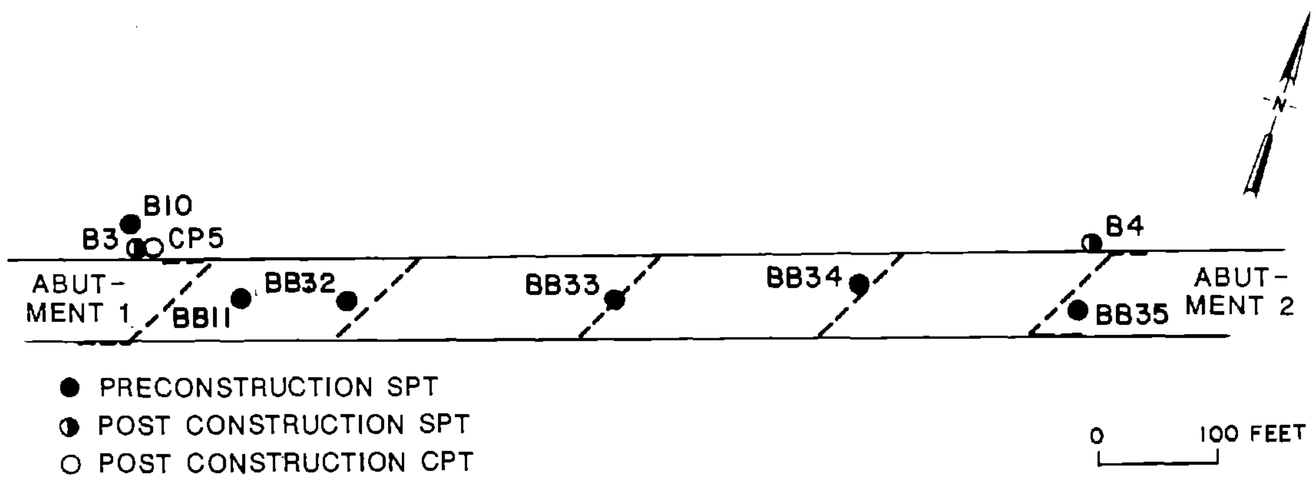
BRIDGE #5 (UXBRIDGE, MA)



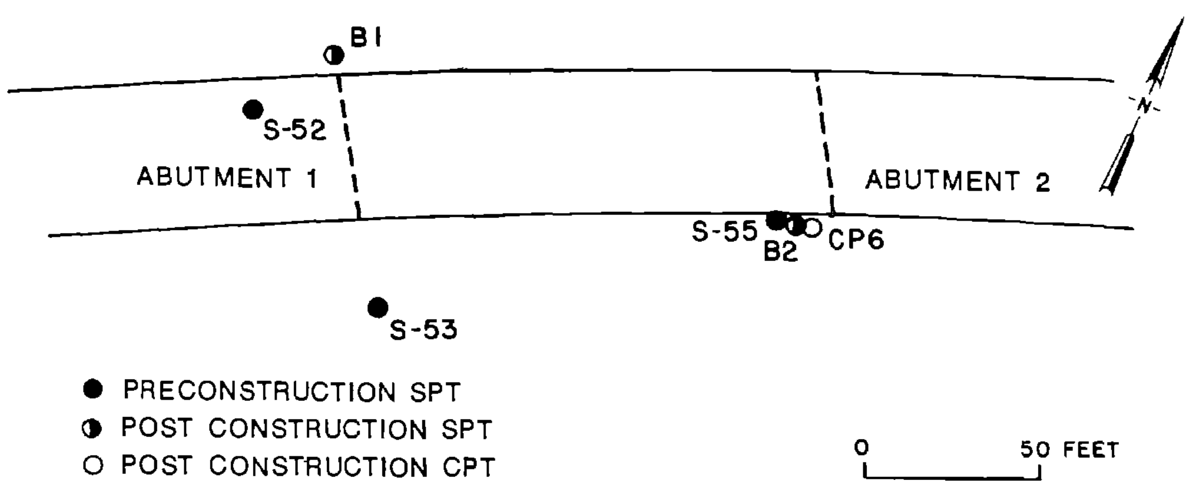
BRIDGE #6 (CHESTER, VT)



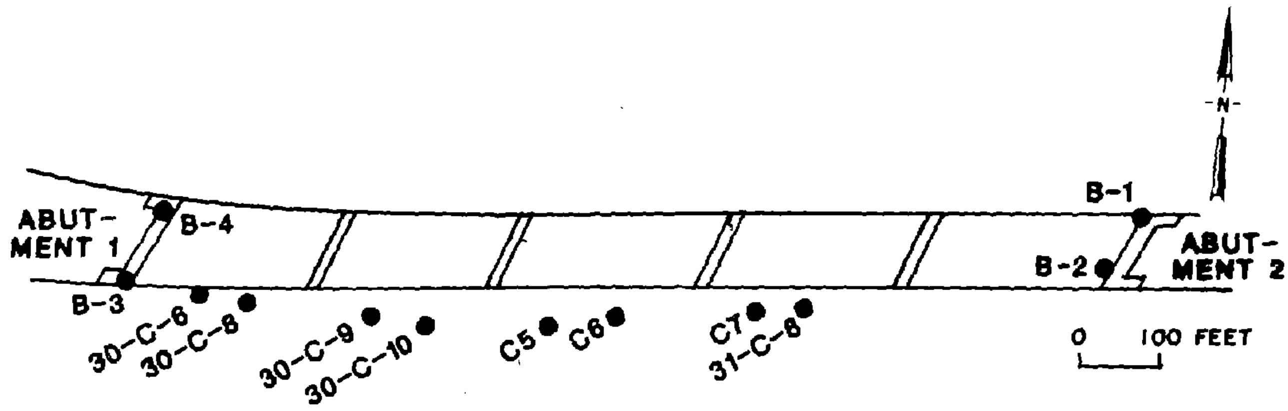
BRIDGE #7 (MANCHESTER, CT 76-88-7)



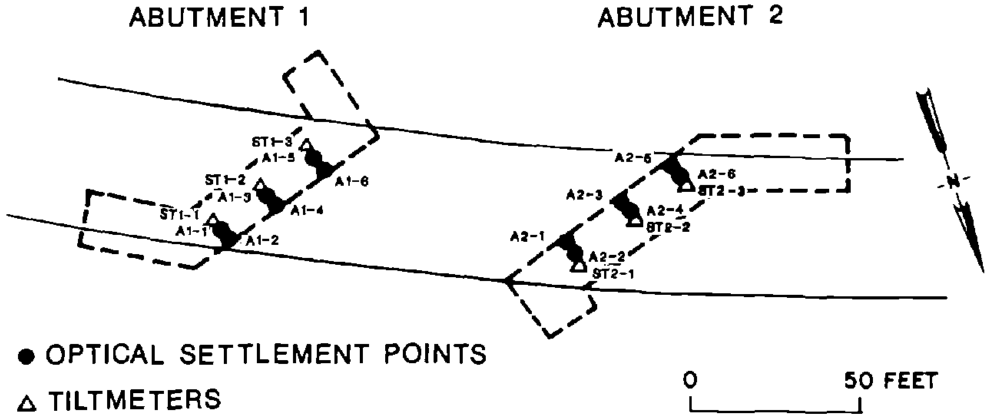
BRIDGE #8 (MANCHESTER, CT 76-88-7)



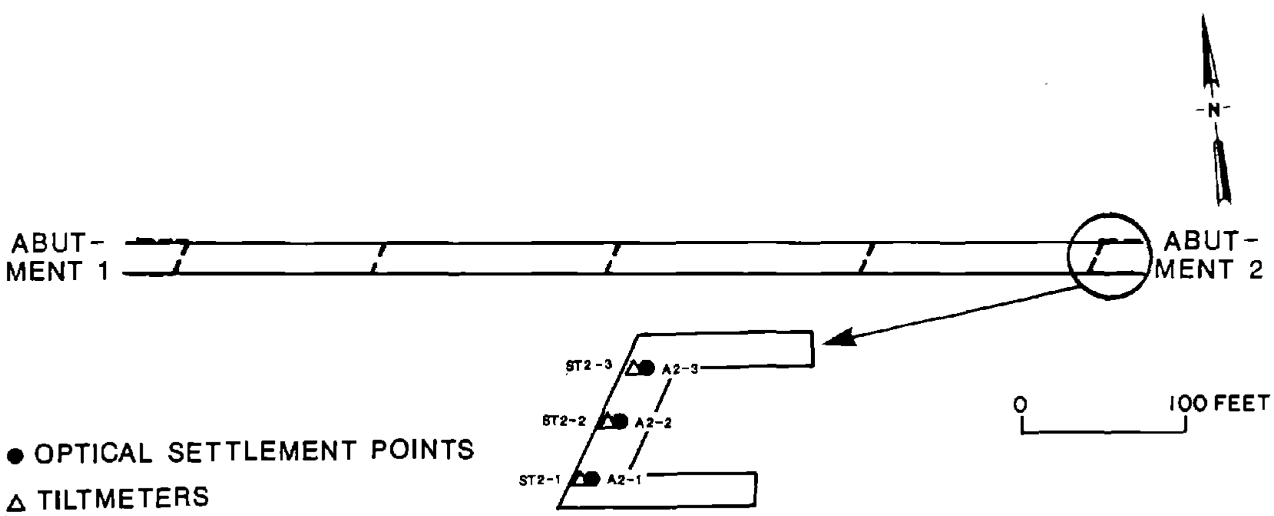
BRIDGE #9 (MANCHESTER, CT 76-88-9)



BRIDGE #10 (MANCHESTER, CT)



BRIDGE #6 (CHESTER, VT)



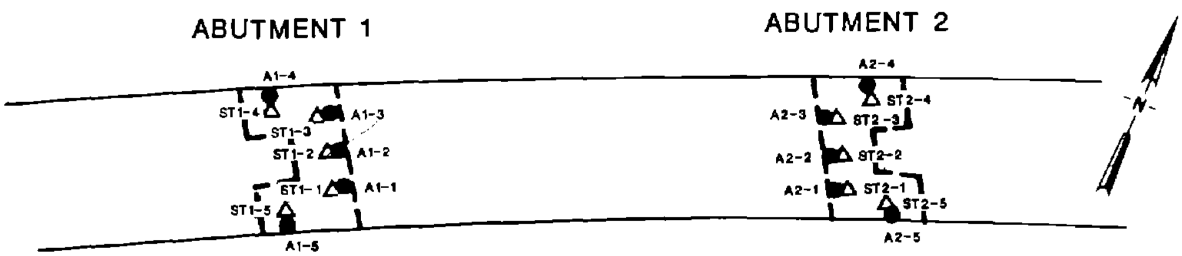
BRIDGE #7 (MANCHESTER, CT 76-88-7)

● OPTICAL SETTLEMENT POINTS

△ TILTMETERS

0 100 FEET

BRIDGE #8 (MANCHESTER, CT 76-88-7)



● OPTICAL SETTLEMENT POINTS

△ TILTMETERS

0 50 FEET

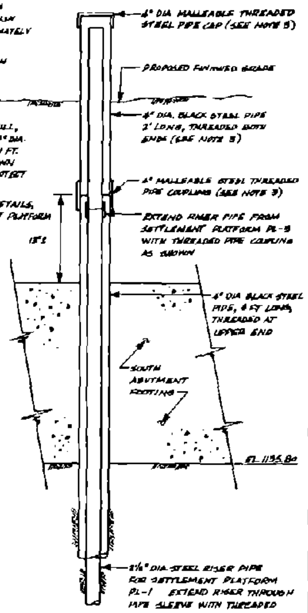
BRIDGE #9 (MANCHESTER, CT 76-88-9)

I. SETTLEMENT PLATFORM PROTECTIVE PIPE SLEEVE DETAIL

SCALE: 1½" = 1'-0"

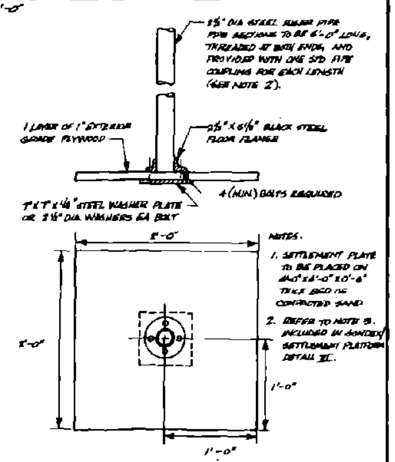
NOTES:

1. EXTEND 2½" RISER PIPE FROM SETTLEMENT PLATFORM THROUGH 4" DIA PIPE SLEEVE APPROXIMATELY 1 FT. ABOVE TOP OF FOOTING
2. INSTALL THREADED PIPE CAP ON TOP OF 4" PIPE SLEEVE CAST THROUGH FOOTING TO PROTECT SETTLEMENT PLATFORM RISER PIPE.
3. PRIOR TO PLACEMENT OF BACKFILL, EXTEND 2½" RISER PIPE AND 4" DIA. PIPE SLEEVE TO APPROXIMATELY 1 FT. ABOVE FINISHED GRADE AS SHOWN. INSTALL THREADED CAP TO PROTECT RISER PIPE.
4. FOR SETTLEMENT PLATFORM DETAILS, REFER TO SONDEX/SETTLEMENT PLATFORM DETAIL III, NOTE 3



II. SETTLEMENT PLATFORM DETAIL

SCALE: 1½" = 1'-0"

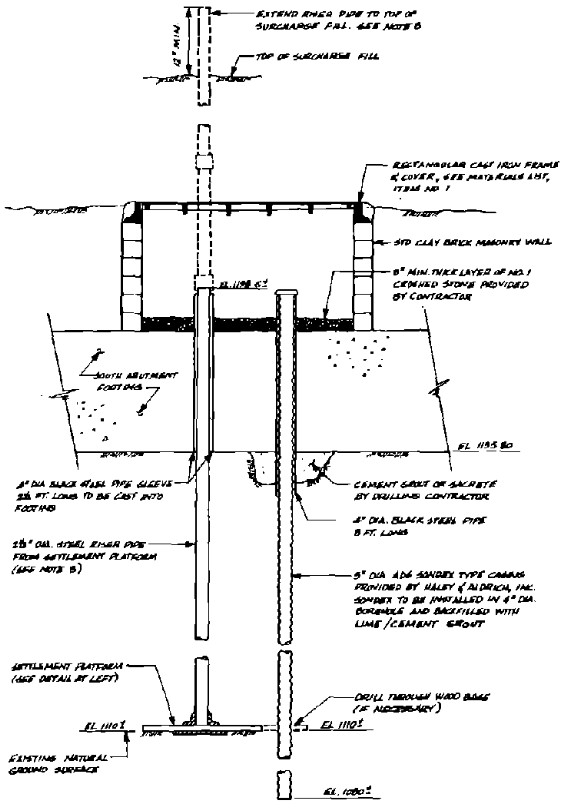


NOTES.

1. SETTLEMENT PLATE TO BE PLACED ON 4'-0" X 4'-0" X 0'-6" THICK BED OF COMPACTED SAND
2. REFER TO NOTE 3. INCLUDED IN SONDEX/SETTLEMENT PLATFORM DETAIL III.

III. SONDEX/SETTLEMENT PLATFORM DETAIL AT MANHOLE MH-2

SCALE: 1" = 1'-0"



NOTES

1. SONDEX CASING WILL BE INSTALLED IN MH-2. ACCESS FOR A DRILL RIG SHALL BE PROVIDED PRIOR TO CONCRETING THE FOOTING. ESTIMATED TIME FOR INSTALLATION IS 3 DAYS
2. BOREHOLE REQUIRED FOR SONDEX SHALL BE 4" DIA AND SHALL BE ADVANCED UTILIZING STANDARD SPT SAMPLING AT 5 FT. INTERVALS. TEST BORING RIG TO BE FURNISHED BY N.Y. STATE DEPARTMENT OF TRANSPORTATION.
3. SETTLEMENT PLATFORM TO BE INSTALLED ON EXISTING NATURAL GROUND SURFACE AT APPROXIMATELY EL 1110±. THE RISER PIPE FOR EACH SETTLEMENT PLATFORM SHALL BE EXTENDED TO TOP OF SURCHARGE ELEVATION WITH THREADED PIPE LENGTHS AND COUPLINGS. FOLLOWING THE SURCHARGE PERIOD, THE RISER PIPE FOR EACH PLATFORM SHALL BE SHORTENED TO APPROXIMATELY EL. 1138.6 AS SHOWN PRIOR TO CONSTRUCTION OF THE SOUTH ABUTMENT FOOTING.

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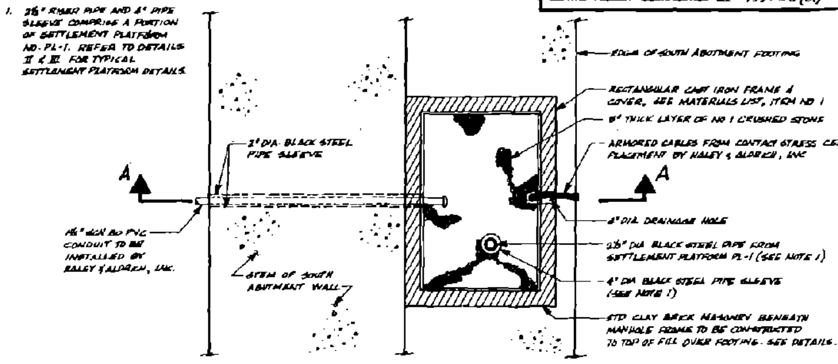
IV. PLAN VIEW OF INSTRUMENTATION MANHOLE MH-1

SCALE: 1" = 1'-0"

NOTES:

1. 2½" RISER PIPE AND 4" PIPE SLEEVE COMPRISE A PORTION OF SETTLEMENT PLATFORM NO. PL-1. REFER TO DETAILS II & III FOR TYPICAL SETTLEMENT PLATFORM DETAILS.

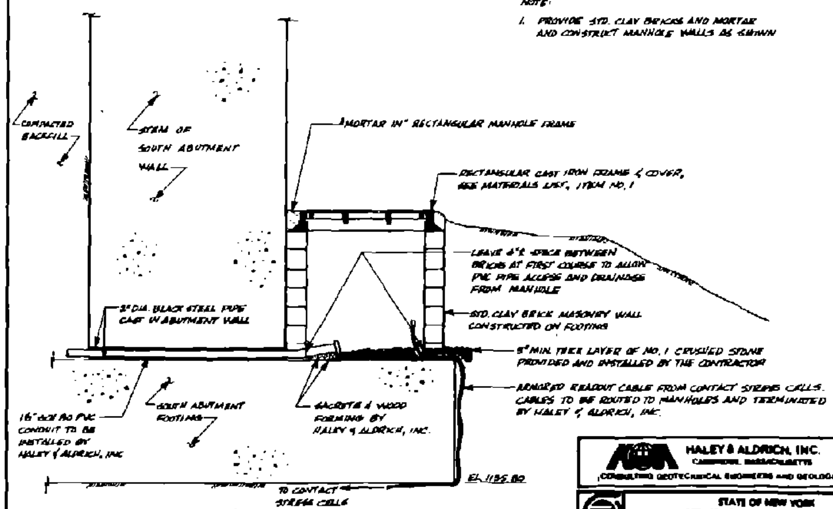


V. INSTRUMENTATION MANHOLE DETAIL - SECTION A-A

SCALE: 1" = 1'-0"

NOTE:

1. PROVIDE STD. CLAY BRICKS AND MORTAR AND CONSTRUCT MANHOLE WALLS AS SHOWN



FED. RD. REG. NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.	TOTAL SHEETS
	NEW YORK			

COLLIERSVILLE CONNECTOR SH 77-20
OTSEGO COUNTY

CAPITAL PROJECT IDENTIFICATION NO. 9357.08(01)

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN AND CONSTRUCTION DIVISION

BRIDGE NO. 5 RELOCATED RT. 28
OVER D&H R.R. & EXIST. RT. 7
(ITEM 09203.92)
GEOTECHNICAL INSTRUMENTATION
DETAILS

PROJ. ENG.
SCHED
DATE MADE 6 APRIL 1983
DRAWING NO. 10 OF 34

151 c

DESIGNED BY
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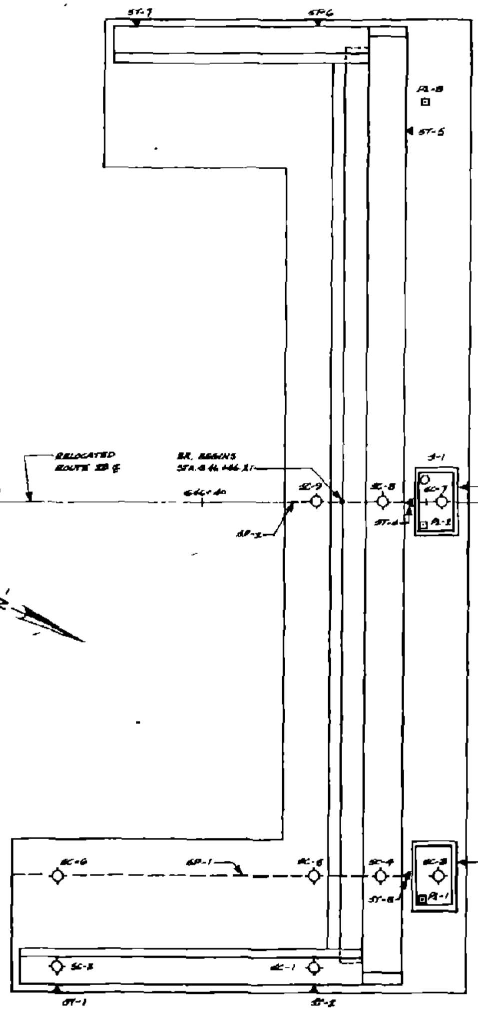
DETAIL CHECKED BY

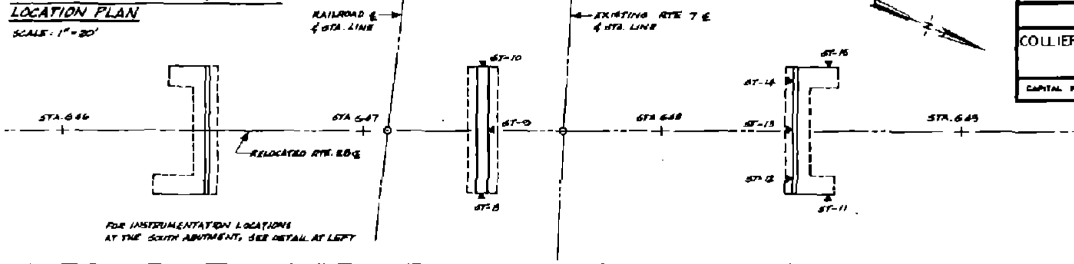
GEOTECHNICAL INSTRUMENTATION DETAIL - SOUTH ABUTMENT

SCALE: 3/8" = 1'-0"



GEOTECHNICAL INSTRUMENTATION LOCATION PLAN

SCALE: 1" = 30'



FOR INSTRUMENTATION LOCATIONS AT THE SOUTH ABUTMENT, SEE DETAIL AT LEFT

I. GEOTECHNICAL INSTRUMENTATION LOCATION SCHEDULE

SYMBOL	INSTRUMENT TYPE	QUANTITY	INSTRUMENT NUMBER	LOCATION	STATION	OFFSET	READOUT TERMINATION	REMARKS
SC-9	CONTACT STRESS CELLS	9	SC-1 SC-2 SC-3 SC-4 SC-5 SC-6 SC-7 SC-8 SC-9	South Abutment " " " " " " " "	[illegible]	[illegible]	MH-1 MH-1 MH-1 MH-1 MH-1 MH-1 MH-2 MH-2 MH-2	CABLE LENGTH: [illegible]
SP-1	REMOTE SETTLEMENT/ PROFILE CONDUIT	2	SP-1 SP-2	South Abutment "	[illegible]	[illegible]	MH-1 MH-2	
S-1	SONDEX	1	S-1	South Abutment	[illegible]	[illegible]	MH-2	
ST-6	ABUTMENT SETTLEMENT/ TILT REFERENCE STUDS	15	ST-1 ST-2 ST-3 ST-4 ST-5 ST-6 ST-7 ST-8 ST-9 ST-10 ST-11 ST-12 ST-13 ST-14 ST-15	South Abutment " " " " " " Pier " " North Abutment " " " "	[illegible]	[illegible]		REFERENCE EL. [illegible]
PL-3	SETTLEMENT PLATFORM	3	PL-1 PL-2 PL-3	South Abutment " "	[illegible]	[illegible]	MH-1 MH-2 "	
MH-1	INSTRUMENTATION MANHOLE	2	MH-1 MH-2	South Abutment "	[illegible]	[illegible]		

II. LIST OF MATERIALS

NO.	ITEM DESCRIPTION	QUANTITY	REFERENCE	REMARKS
1	RECTANGULAR CAST IRON FRAME [illegible] IN. BY [illegible] IN. OUTER DIMENSIONS, [illegible] IN. BY [illegible] IN. REMOVABLE COVER	2 EA.	[illegible] LEBANON FOUNDRY CO., NO. [illegible] OR EQUIVALENT	
2	STANDARD CLAY BRICKS, REFER TO FIGURE [illegible], DETAIL IV AND V	AS REQUIRED TO CONSTRUCT WALL, FIG. [illegible], DETAIL IV AND V.		
3	[illegible] IN. SCH. [illegible] PVC CONDUIT WITH MISC. SOCKET COUPLINGS AND END CAPS	[illegible] L.F. [illegible] COUPLINGS [illegible] SLIP CAPS	ASTM D[illegible]	
4	VIBRATING WIRE TYPE CONTACT STRESS CELL - [illegible] MODEL [illegible], ETC. BY GEOKON, INC., W. LEBANON, N.H.	9 EA.		PRESSURE RANGE [illegible] PSI, ARMORED READOUT CABLE LENGTH FOR EACH CELL TABULATED IN GEOTECHNICAL INSTRUMENTATION LOCATION SCHEDULE
5	[illegible] IN. STD. BLACK STEEL PIPE	[illegible] L.F.	ASTM [illegible] ASTM [illegible]	PIPE TO BE SUPPLIED IN THE FOLLOWING LENGTH [illegible] EA. AT [illegible] L.F.
6	[illegible] IN. STD. BLACK STEEL PIPE THREADED ON BOTH ENDS	[illegible] L.F.	ASTM [illegible] ASTM [illegible]	PIPE TO BE SUPPLIED IN THE FOLLOWING LENGTHS. [illegible] EA. AT [illegible] L.F. [illegible] EA. AT [illegible] L.F. [illegible] EA. AT [illegible] L.F. [illegible] EA. AT [illegible] L.F.
7	[illegible] IN. STD. MALLEABLE THREADED PIPE COUPLING	[illegible] EA.	ASTM [illegible] ASTM [illegible]	
8	[illegible] IN. STD. THREADED STEEL PIPE CAPS	[illegible] EA.	ASTM [illegible] ASTM [illegible]	
9	SETTLEMENT PLATFORMS (SEE FIGURE [illegible], DETAIL [illegible])	[illegible] EA.		

III. GEOTECHNICAL INSTRUMENTATION WORK ITEM RESPONSIBILITIES

INSTRUMENT TYPE	QUANTITY	WORK ITEM RESPONSIBILITIES: H&A INC. RESPONSIBILITIES — MATERIALS TO BE SUPPLIED	H&A INC. RESPONSIBILITIES — WORK ITEMS	CONTRACTOR'S RESPONSIBILITIES — MATERIALS TO BE SUPPLIED	CONTRACTOR'S RESPONSIBILITIES — WORK ITEMS	REMARKS
SETTLEMENT PLATFORM (STANDPIPE WITH WOODEN BASE)	3	• NONE	• MONITOR AS REQUIRED	• REFER TO FIG. [illegible], DETAIL [illegible], FOR SETTLEMENT PLATFORM MATERIALS • REFER TO LIST OF MATERIALS, ITEMS [illegible] FOR PROTECTIVE PIPE SLEEVE MATERIALS	• REFER TO FIG. [illegible], DETAIL [illegible] FOR INSTALLATION DETAILS • EXTEND RISER PIPE THROUGH FOOTING AND UP TO FINISHED GRADE • PROTECT RISER PIPE FROM DAMAGE DURING EXCAVATION AND BACKFILLING • INSTALL [illegible] IN. PROTECTIVE PIPE SLEEVES AS INDICATED	1. REFER TO GEOTECHNICAL INSTRUMENTATION DETAIL, SOUTH ABUTMENT, AND SCHEDULE I; SEE DETAILS [illegible] AND [illegible], FIGURE [illegible]. 2. TEST BORING RIG TO BE FURNISHED BY THE NEW YORK STATE DEPARTMENT OF TRANSPORTATION. SONDEX TO BE INSTALLED FOLLOWING EXCAVATION TO BEARING ELEVATION AT SOUTH ABUTMENT FOOTING. ESTIMATED [illegible] DAYS REQUIRED TO COMPLETE SONDEX INSTALLATION.
CONTACT STRESS CELLS	9	• ALL INSTALLATION HARDWARE AND READOUT EQUIPMENT	• INSTALL EACH STRESS CELL • ROUTE CABLES TO MANHOLES • MONITOR EQUIPMENT AS REQUIRED	• REFER TO LIST OF MATERIALS, ITEM 4	• PROVIDE ACCESS FOR INSTALLATION PRIOR TO PLACEMENT OF REBAR • PROTECT INSTRUMENTS FROM DAMAGE PRIOR TO CONCRETING FOOTING	
INSTRUMENTATION MANHOLE	2	• ALL READOUT EQUIPMENT TO BE INSTALLED IN MANHOLES	• ROUTE PVC CONDUIT AND READOUT CABLES TO MANHOLES • INSTALL INSTRUMENTATION READOUT EQUIPMENT	• REFER TO LIST OF MATERIALS, ITEMS [illegible] • MORTAR FOR BRICK WALL CONSTRUCTION • NO. 1 CRUSHED STONE	• CONSTRUCT MANHOLES AT LOCATIONS SPECIFIED. SEE REMARK 1. • PLACE NO. 1 CRUSHED STONE DRAINAGE MATERIAL FOLLOWING MANHOLE CONSTRUCTION	
ABUTMENT SETTLEMENT/ TILT REFERENCE STUDS	15	• ALL HARDWARE AND READOUT EQUIPMENT	• INSTALL STUDS AT LOCATIONS INDICATED • MONITOR EQUIPMENT AS REQUIRED	• NONE	• PROVIDE ACCESS FOR INSTALLATION AND MONITORING	
LOAD MEASUREMENT RING	[illegible]	• ALL HARDWARE AND READOUT EQUIPMENT	• SELECT LOCATIONS FOR EACH RING ON STEEL GIRDERS ABOVE ABUTMENT WALLS • INSTALL EACH RING IN EARS MOUNTED BY H&A, INC. • MONITOR AS REQUIRED	• NONE	• PROVIDE ACCESS FOR RING INSTALLATION • PROVIDE [illegible] FULLY LOADED [illegible] TRUCKS FOR [illegible] HOURS EACH TO USE AS LOAD ON BRIDGE • PARK TRUCKS ON BRIDGE AS EQUIPMENT IS MONITORED	
REMOTE SETTLEMENT/ PROFILE CONDUIT	2	• ALL READOUT EQUIPMENT	• INSTALL PVC CONDUIT THROUGH PIPE CAST IN CONCRETE ABUTMENT WALLS • FASTEN PVC CONDUIT TO TOP OF FOOTING • ROUTE CONDUIT TO MANHOLES AS SHOWN	• REFER TO LIST OF MATERIALS, ITEMS [illegible]	• CUT [illegible] IN. PIPE TO SIZE AND CAST IN ABUTMENT WALL. SEE REMARK 1. • ASSIST H&A INC. WITH ROUTING OF PVC CONDUIT TO MANHOLES • PROTECT CONDUIT FROM DAMAGE PRIOR TO PLACEMENT OF BACKFILL	
SONDEX - DEEP SETTLEMENT MONITORING DEVICE	1	• ALL READOUT EQUIPMENT • SONDEX CASING AND MISCELLANEOUS FITTINGS	• INSTALL SONDEX CASING FOLLOWING EXCAVATION TO FOOTING BEARING ELEVATION • INSTALL [illegible] FT. LENGTH OF [illegible] IN. PIPE FOR TEMPORARY PROTECTION PRIOR TO CONCRETING FOOTING	• REFER TO LIST OF MATERIALS, ITEM [illegible]	• PROVIDE ACCESS FOR TEST BORING RIG PRIOR TO FOOTING CONSTRUCTION. SEE REMARK 2. • PROTECT INSTRUMENT PRIOR TO CONCRETING FOOTING	

GENERAL NOTE:

THE CONTRACTOR MAY CONTACT MESSRS. DOUGLAS GIFFORD OR JAMES WHEELER OF HALEY & ALDRICH, INC., 238 MAIN STREET, CAMBRIDGE, MA. TELEPHONE (617) 492-6460 FOR ANSWERS TO TECHNICAL QUESTIONS ON GEOTECHNICAL INSTRUMENTATION.

FED. RD. REG. NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.	TOTAL SHEETS
	NEW YORK			

COLLIERSVILLE CONNECTOR SH 77-20

OTSEGO COUNTY

CAPITAL PROJECT IDENTIFICATION NO. 9357.08 (01)

HALEY & ALDRICH, INC.
Cambridge, Massachusetts
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN AND CONSTRUCTION DIVISION

BRIDGE NO. 5 RELOCATED RT. 28
OVER D&H R.R. & EXIST. RT. 7
(ITEM 09203.92)
GEOTECHNICAL INSTRUMENTATION
LOCATION PLANS & SCHEDULES

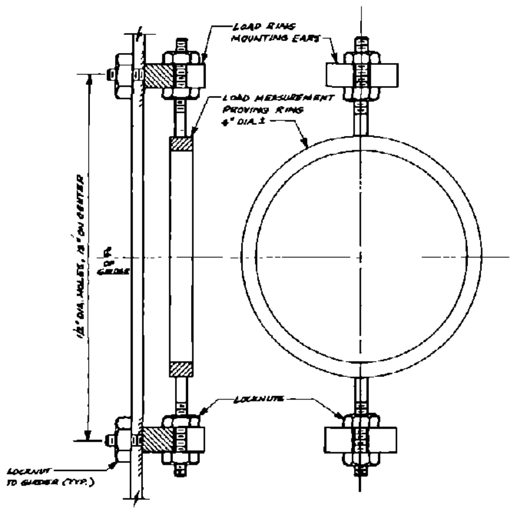
DATE MADE 8 APRIL 1983

DRAWING NO. 8 OF 34

DESIGNED BY [illegible] DESIGN CHECKED BY DETAILED BY [illegible] DETAIL CHECKED BY

I. LOAD MEASUREMENT RING DETAIL

SCALE: APPROXIMATELY FULL SIZE



NOTES:

1. ALL LOAD MEASUREMENT RING EQUIPMENT TO BE PROVIDED AND INSTALLED BY HALEY & ALDRICH, INC. THIS DETAIL HAS BEEN INCLUDED FOR THE CONTRACTOR'S INFORMATION ONLY. THE CONTRACTOR SHALL PROVIDE ACCESS NECESSARY FOR INSTALLATION.
2. A TOTAL OF SIX RINGS WILL BE INSTALLED BY HALEY & ALDRICH, INC. ABOVE SOUTH ABUTMENT WALL ON GIRDER FACES AS SHOWN AT LEFT.
3. LOCATIONS AND DIMENSIONS OF HOLES REQUIRED FOR MOUNTING WILL BE PROVIDED AFTER GIRDER ERECTION.
4. THE CONTRACTOR IS TO PROVIDE THREE 3-AXLE STRAIGHT TRUCKS EACH LOADED TO THE MAXIMUM GROSS LOAD OF 65,000 LBS. THESE TRUCKS WILL BE USED TO PLACE LOADS ON THE BRIDGE AT VARIOUS LOCATIONS WHILE INSTRUMENTS ARE MONITORED BY HALEY & ALDRICH, INC. NO LOADS SHALL BE PLACED SOONER THAN 28 DAYS AFTER THE STRUCTURAL SLAB HAS BEEN POURED. HALEY & ALDRICH, INC. WILL PROVIDE AT LEAST 1 WEEK NOTICE AS TO WHEN TRUCKS WILL BE REQUIRED.

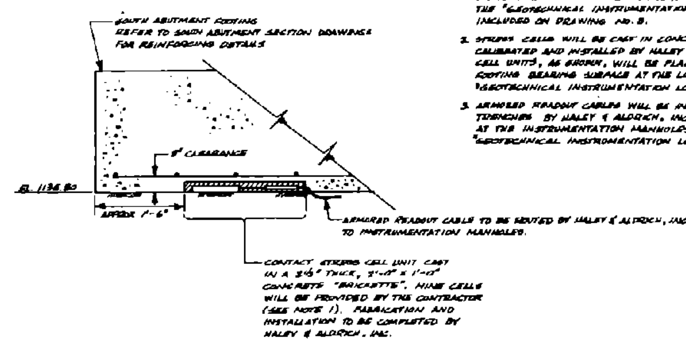
III. CONTACT STRESS CELL INSTALLATION DETAIL

SCALE: 1" = 1'-0"

FED. RD. REG. NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.	TOTAL SHEETS
	NEW YORK			

COLLIERSVILLE CONNECTOR SH 77-20
OTSEGO COUNTY

CAPITAL PROJECT IDENTIFICATION NO. 9357.08 (01)

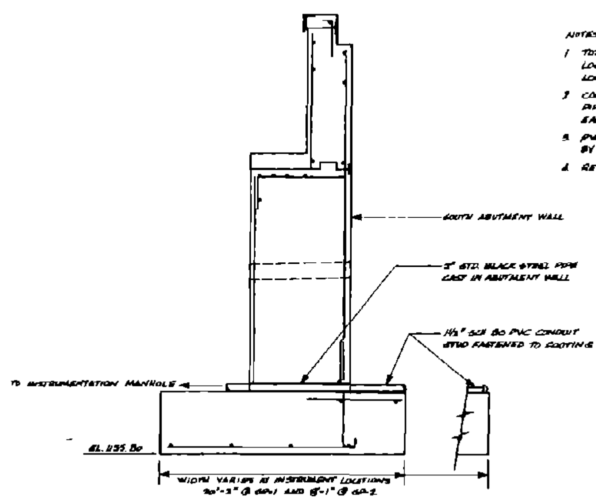


NOTES:

1. A TOTAL OF 9 CONTACT STRESS CELLS ARE TO BE DELIVERED TO HALEY & ALDRICH, INC. 238 MAIN ST., CAMBRIDGE, MA AT LEAST 3 WEEKS PRIOR TO PLACEMENT OF CONCRETE AT SOUTH ABUTMENT FOOTING. READOUT CABLE LENGTHS WHICH ARE FACTORY INSTALLED ON EACH INSTRUMENT ARE TABULATED IN THE "GEOTECHNICAL INSTRUMENTATION LOCATION SCHEDULE" INCLUDED ON DRAWING NO. 8.
2. STRESS CELLS WILL BE CAST IN CONCRETE "BRICKETTES", CALIBRATED AND INSTALLED BY HALEY & ALDRICH, INC. STRESS CELL UNITS, AS SHOWN, WILL BE PLACED ON THE PREPARED FOOTING BEARING SURFACE AT THE LOCATIONS INDICATED ON THE "GEOTECHNICAL INSTRUMENTATION LOCATION PLAN."
3. ARMORED READOUT CABLES WILL BE INSTALLED IN SHALLOW TRENCHES BY HALEY & ALDRICH, INC. AND WILL TERMINATE AT THE INSTRUMENTATION MANHOLES. REFER TO "GEOTECHNICAL INSTRUMENTATION LOCATION SCHEDULE."

II. REMOTE SETTLEMENT/PROFILE MONITORING CONDUIT INSTALLATION DETAIL

SCALE: ½" = 1'-0"



NOTES:

1. TOTAL 2 CONDUIT INSTALLATIONS ON SOUTH ABUTMENT FOOTING AT THE LOCATIONS INDICATED ON THE "GEOTECHNICAL INSTRUMENTATION LOCATION PLAN"
2. CONTRACTOR TO PROVIDE AND INSTALL BLACK IRON PIPE AS SHOWN. PIPE SHALL BE CUT TO LENGTH, BLOCKED OFF WITH STYROFOAM AT EACH END, AND CAST INTO THE SOUTH ABUTMENT WALL.
3. PVC CONDUIT TO BE PROVIDED BY THE CONTRACTOR AND INSTALLED BY HALEY & ALDRICH, INC. PRIOR TO BACKFILL PLACEMENT.
4. REFER TO REINFORCING PLANS FOR LOCATIONS OF REBAR.

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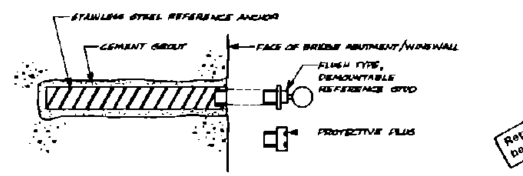
IV. ABUTMENT SETTLEMENT/TILT REFERENCE STUD DETAIL



NOTES:

1. APPROXIMATELY 30 STUDS TO BE SUPPLIED AND INSTALLED IN PAIRS BY HALEY & ALDRICH, INC. AT THE LOCATIONS SHOWN ON THE "GEOTECHNICAL INSTRUMENTATION LOCATION PLAN.
2. THIS DETAIL IS INCLUDED FOR INFORMATION ONLY. CONTRACTOR TO PROVIDE ACCESS NECESSARY FOR INSTALLATION. NO OTHER WORK IS REQUIRED.

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HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN AND CONSTRUCTION DIVISION

BRIDGE NO. 5 RELOCATED RT. 28
OVER O & H R.R. & EXIST. RT. 7
(ITEM 0920392)
GEOTECHNICAL INSTRUMENTATION
DETAILS

DATE MADE 8 APRIL 1983
DRAWING NO. 9 OF 34

DESIGNED BY / DETAILED BY / DESIGN CHECKED BY / DETAIL CHECKED BY

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APPENDIX D: SETTLEMENT CALCULATION METHODS

METHOD: Alpan

REFERENCES: Alpan, I., "Estimating the Settlements of Foundations on Sands," Civil Engineering and Public Works Review, November 1964.

BASIC EQUATION: $\rho \text{ (inches)} = \alpha_0 P \left(\frac{2B}{B+1}\right)^2$

PARAMETERS:

α_0 = reciprocal of modulus of subgrade reaction of test plate (in.-ft.^{2}/ton)

P = footing bearing pressure in tons per sq. ft.

B = footing width in feet

COMMENTS:

1. α_0 is determined from chart, Alpan (1964). α_0 is a function of the SPT blow count at foundation level, corrected for overburden pressure by Gibbs and Holtz method. A chart for correcting SPT results for overburden pressure is also given by Alpan (1964).

2. Correction factor for footing shape (rectangle or circle) is also given in chart form by Alpan (1964).

3. Method is based on Terzaghi and Peck approach. See comments given for Terzaghi and Peck method.

METHOD: Buisman-DeBeer

REFERENCES: DeBeer, E., "Bearing Capacity and Settlement of Shallow Foundations on Sand," Proceedings, Symposium on Bearing Capacity and Settlement of Foundations, Duke University, 1965.

BASIC EQUATION: $$\rho = \sum_{0}^{z} \left(\frac{2.3}{C}\right) \Delta z \log \left(\frac{\bar{\sigma}_{vo} + \Delta\bar{\sigma}_{v}}{\bar{\sigma}_{vo}}\right)$$

PARAMETERS:

$\bar{\sigma}_{vo}$ = initial effective overburden pressure at center of layer of constant C

C = sand compressibility = $\frac{3}{2} \left(\frac{q_c}{\bar{\sigma}_{vo}}\right)$

q_c = cone point penetration resistance

Δ_z = thickness of layer of constant C

$\Delta\bar{\sigma}_v$ = change in vertical effective stress at center of layer

COMMENTS:

1. Method follows the same approach as used for calculating consolidation settlement of clays. The soil is divided into layers of constant C, and the change in vertical stress as a result of the applied loading is calculated at the center of each layer. If q_c is constant with depth use Boussinesq equation for $\Delta\sigma_v$:

$$\Delta\sigma_v = \frac{3P}{2\pi z^2} \cos^5 \theta$$

If q_c increases with depth use Buisman equation:

$$\Delta\sigma_v = \frac{2P}{\pi z^2} \cos^6 \theta$$

2. Equation above applies only to normally consolidated sands. For overconsolidated sands, DeBeer suggests performing an oedometer test on the sand in the laboratory and multiply C by the slope of the laboratory e vs. log σ_v line in the rebound compression range divided by the slope of the virgin compression line. The quantity calculated is the value of C to be used in the above equation for over-consolidated sands.

METHOD: Buisman-DeBeer (continued)

3. The mean ratio of predicted to measured settlement using this method was 2, based on 50 highway bridges studied in Belgium by DeBeer.

4. Meyerhof has proposed the use of
$C = 1.9\ (q_c/\bar{\sigma}_{vo})$

METHOD: Burland

REFERENCES: Burland, J.B. and Burbridge, M.C., "Settlement of Foundations on Sand and Gravel," Institution of Civil Engineers - Glasgow and West of Scotland Association, 1984.

BASIC EQUATION: $\rho\ (\text{mm}) = f_s\ f_l\ f_t\ [(q' - \frac{2}{3}\ \sigma'_{vo}) \cdot B^{0.7} \cdot I_c]$

PARAMETERS:

q' = average gross effective applied pressure (kN/m^2)

σ_{vo}' = maximum previous effective overburden pressure (kN/m^2)

B = width of footing (m)

I_c = compressibility index = $\dfrac{1.71}{\bar{N}^{1.4}}$

N = mean SPT N over depth of influence (Z_I)

Z_I = function of B, presented graphically by Burland (1984) (see below).

f_s = shape correction factor

f_l = correction factor for thickness of sand or gravel layer

f_t = time factor, used if $t \geq 3$ yrs.

COMMENTS:

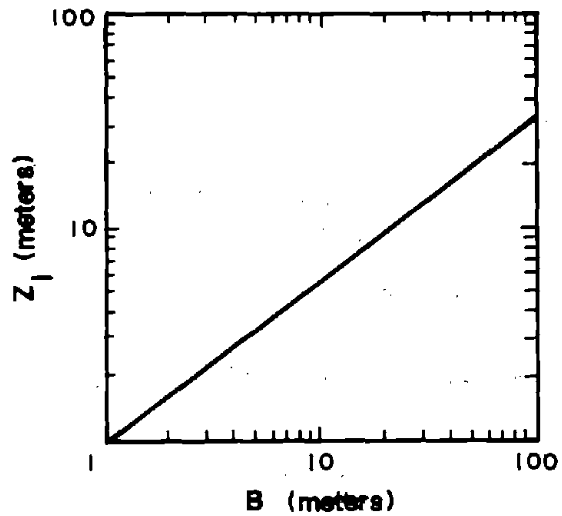
1. This method establishes an empirical relationship between average SPT blow count, foundation width, and foundation subgrade compressibility. It is based on regression analysis of case studies.

2. Blow counts are not corrected for overburden pressure, but are corrected if subgrade consists of fine or silty sands below the water table. Correction is made according to that proposed by Terzaghi and Peck (1948), $N_c = 15 + 0.5\ (N-15)$ for $N>15$. If subgrade is gravel or sandy gravel, $N_c = 1.25N$.

3. Use of the three correction factors, (f_s, f_l, f_t) is outlined in detail by Burland (1984).

METHOD: Burland (continued)

4. If the sand is overconsolidated and the change is less than the effective preconsolidation pressure, the valve of I_c should be reduced by a factor of 3.



Depth of influence Z_I vs. footing width B.

METHOD: D'Appolonia

REFERENCES: D'Appolonia, D.J., D'Appolonia, E., Brissette, R.F., (May 1968), "Settlement of Spread Footings on Sand," (closure) Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96 (SM2), pp. 754-761.

BASIC EQUATION: $\rho = \frac{qB}{M} \mu_o \mu_1$

PARAMETERS:

q = footing bearing pressure

B = footing width

M = modulus of compressibility of sand

μ_o = embedment correction factor (see below)

μ_1 = correction factor for thickness of sand layer (see below)

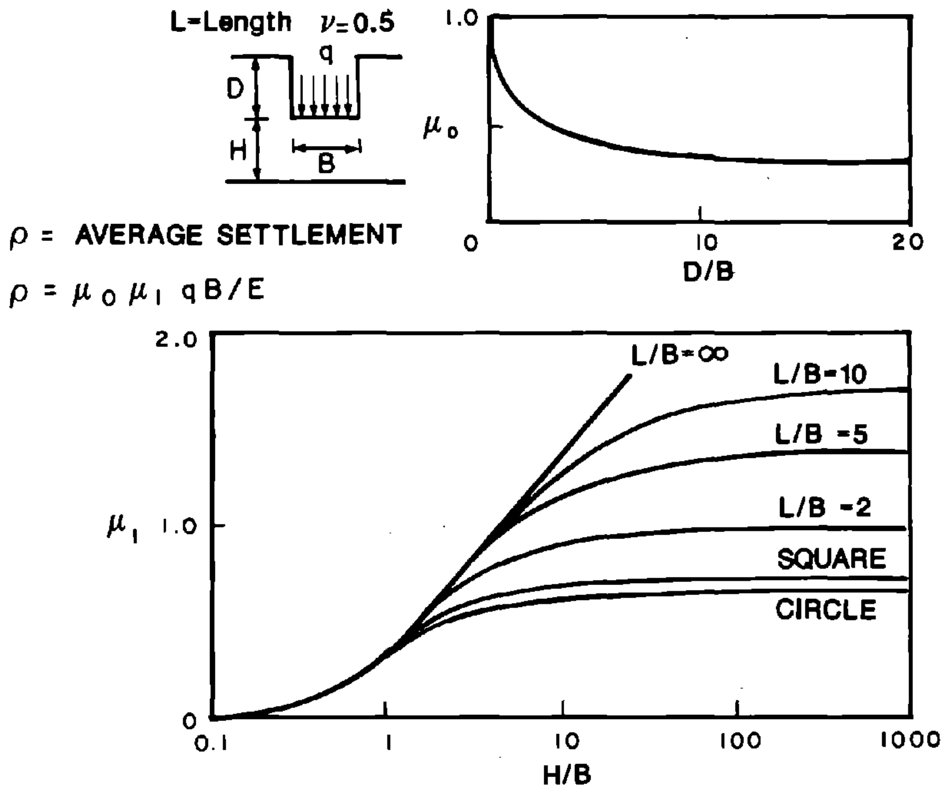
COMMENTS:

1. Corrections for embedment and layer thickness are given in D'Appolonia (1968).

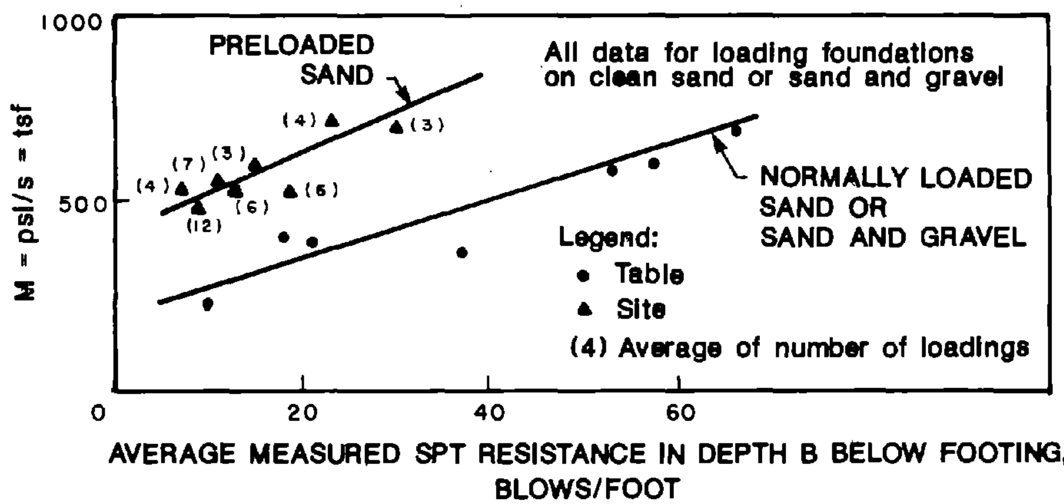
2. Empirical chart for determining M from Standard Penetration Test (SPT) results is given by D'Appolonia (1968) (see below) for both normally consolidated and overconsolidated sands. The overconsolidation of the sand deposit must be determined by geological or other methods. SPT resistance used in the chart is the average blow count in the depth B below the footing bearing level.

3. Method is based on elastic theory. Soil modulus versus SPT relationships determined by backcalculating M from case studies of actual footings. Relationships have been established from a limited database, particularly for the overconsolidated soils.

METHOD: D'Appolonia (continued)



Correction factors (after Christian and Carrier 1978).



Modulus of compressibility vs. blow count.

METHOD: Hough

REFERENCES: Hough, "Compressibility as the Basis for Soil Bearing Value," ASCE Proceedings, August 1959.

Hough, Basic Soils Engineering, 1967, Ronald Press, New York, NY.

BASIC EQUATION: $\rho = \sum_{0}^{z} \left(\frac{1}{C}\right) \Delta z \log \left(\frac{\bar{\sigma}_{vo} + \Delta\bar{\sigma}_v}{\bar{\sigma}_{vo}}\right)$

PARAMETERS: C = bearing capacity index = $\frac{1+e_o}{C_c}$

e_o = initial void ratio

C_c = virgin compression index

Δz = layer thickness

$\bar{\sigma}_{vo}$ = initial effective overburden pressure at mid-height of layer

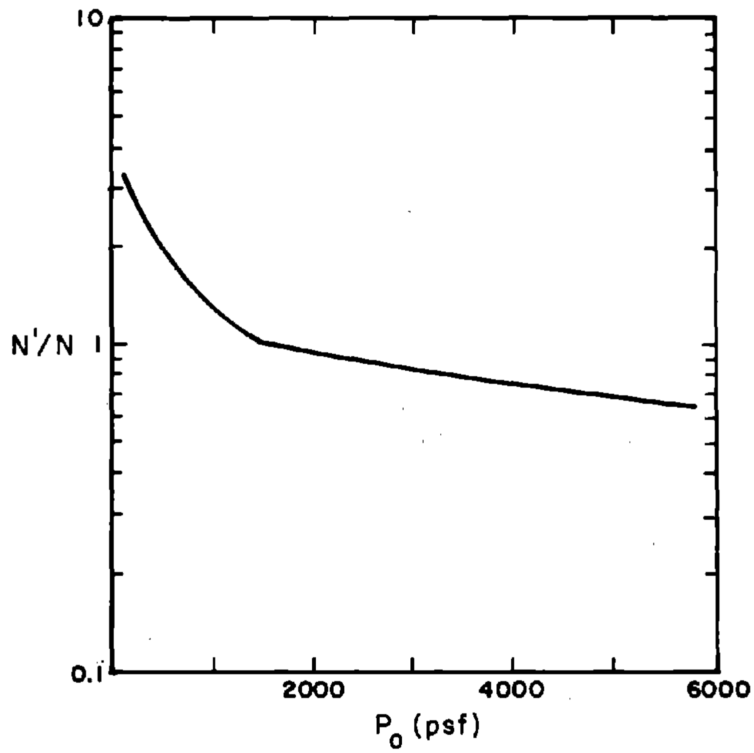
$\Delta\bar{\sigma}_v$ = change in effective vertical stress at layer mid-height

COMMENTS:

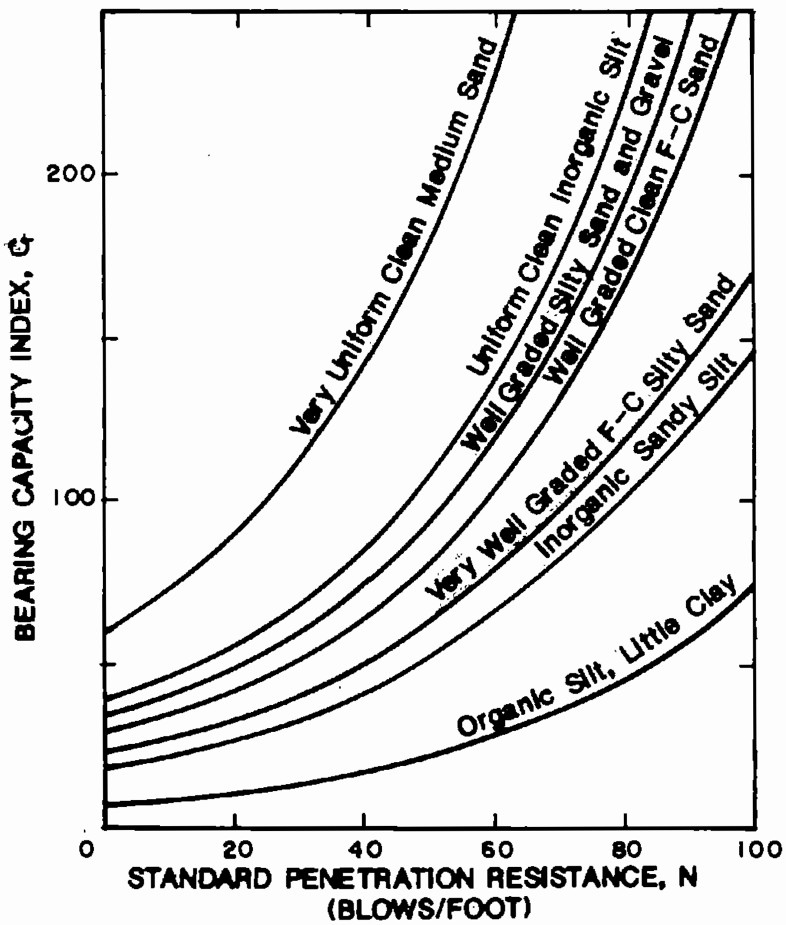
1. Method follows same approach as that for calculating consolidation settlement of clays. The soil is divided into layers, and the change in effective vertical stress at the mid-height of the layer as a result of the applied loading is estimated using an elastic theory relationship such as Fadum's chart.

2. Method applies only to normally consolidated sands.

3. An empirical chart relating SPT resistance corrected for overburden (see below), to the bearing capacity index C for various soil types is given by Hough (1959) (see below).



Corrected blow count (N') after Bazaraa (1967).



Bearing capacity index C vs. blow count N.

METHOD: Menard

REFERENCES: Menard, L., "Rules for the Calculation of Bearing Capacity and Foundation Settlement based on Pressuremeter Tests," 1972 (as reported in the Canadian Manual on Foundation Engineering, 1975).

BASIC EQUATION: $\rho \text{ (ft.)} = f \dfrac{q_{net}}{E_p}$

PARAMETERS:

q_{net} = net bearing pressure at footing level

E_p = pressuremeter modulus within 2B below footing bearing level

f = settlement coefficient in feet

COMMENTS:

1. Empirical method based on experience with pressuremeter measurements in Europe.
2. See Menard (1972) for chart of settlement coefficient f versus soil type, footing width and footing shape.

METHOD: Meyerhof

REFERENCES: Meyerhof, George G., "Shallow Foundations," Proceedings Journal of the Soil Mechanics and Foundations Division, ASCE, March 1965.

BASIC EQUATION: $\rho \text{ (inches)} = C_D \left(\frac{2P}{N}\right) \left(\frac{2B}{B+1}\right)^2$

PARAMETERS: C_D = embedment correction factor = $1 - \frac{D}{4B}$

D = depth of footing embedment in feet

B = footing width in feet

P = footing bearing pressure in tons per sq. ft.

N = SPT blow count

COMMENTS:

1. Method is empirical and is a modified version of the Terzaghi and Peck approach. See comments for Terzaghi and Peck method.

2. Meyerhof believes that presence of water table is reflected in SPT blow count, so no water table correction factor is necessary.

3. Because of the over-conservativism of predicted settlements using the Terzaghi and Peck method, Meyerhof reduced the predicted settlements by one-third to arrive at the constant of 2 in his equation.

4. D'Appolonia, D'Appolonia and Brissette corrected N for the change in overburden pressure caused by site grading between the time of the soil boring and footing construction and obtained good settlement predictions. The Gibbs and Holtz relationships were used to correct the blow counts.

METHOD: NAVFAC DM-7

REFERENCES: "Soil Mechanics," Design Manual 7.1, Department of the Navy U.S. Government Printing Office, Washington D.C., 1982.

BASIC EQUATION: $\rho\ (\text{ft.}) = \dfrac{4qB^2}{K_{v1}(B+1)^2}$

PARAMETERS:

q = footing bearing pressure in tons per sq. ft.

B = footing width in feet

K_{v1} = modulus of vertical subgrade reaction for 1 ft. square bearing plate at ground surface.

COMMENTS:

1. Above equation is for footing width B less than or equal to 20 ft. For B greater than or equal to 40 ft., divide settlements obtained from above equation by two. Interpolate settlement results for B between 20 and 40 ft.

2. Method applies to shallow footings where depth of embedment is less than B.

3. If plate load test not performed, chart is provided in DM7.1 (1982) to obtain K_{v1} from relative density. Relative density usually obtained from correlation with SPT or CPT results.

4. Chart provides K_{v1} values for case of groundwater level at least 1.5B below base of footing. If groundwater level at base of footing, divide K_{v1} values from chart by two.

METHOD: Oweis

REFERENCES: Oweis, Issa S., "Equivalent Linear Model for Predicting Settlements of Sand Bases," Journal of Geotechnical Division, ASCE, December 1979.

BASIC EQUATION: $\rho = \sum_{i=1}^{n} \frac{qB}{E_i} (F_i - F_{i-1})$

PARAMETERS:

n = number of layers of soil

q = net bearing pressure at footing level

B = footing width

E_i = equivalent linear soil modulus for layer i

F_{i-1} = settlement factor at top of layer i

F_i = settlement factor at bottom of layer i

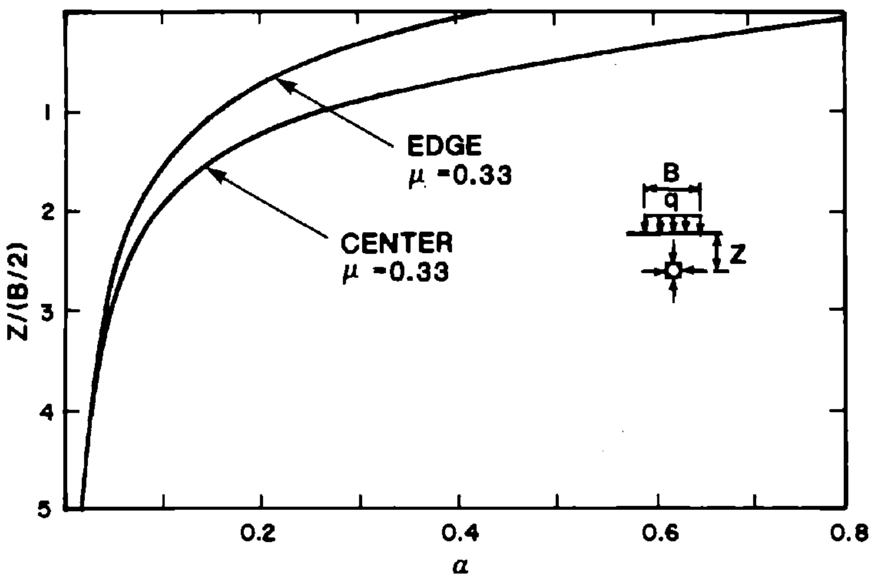
COMMENTS:

1. Basis of method is elastic theory, but non-linear soil stress-strain behavior is accounted for by use of an iterative procedure. The soil is divided into layers to a depth of at least 2B below the footing. An initial soil modulus is calculated for each layer based on SPT blow count using correlations provided by Oweis (1979). The modulus is then multiplied by a reduction factor based on an initial estimate of vertical strain below the footing. This reduced or equivalent linear soil modulus is used in the elastic equation above to calculate settlements. A step-by-step procedure including the charts and equations required, is given by Oweis (1979). Charts are provided below.

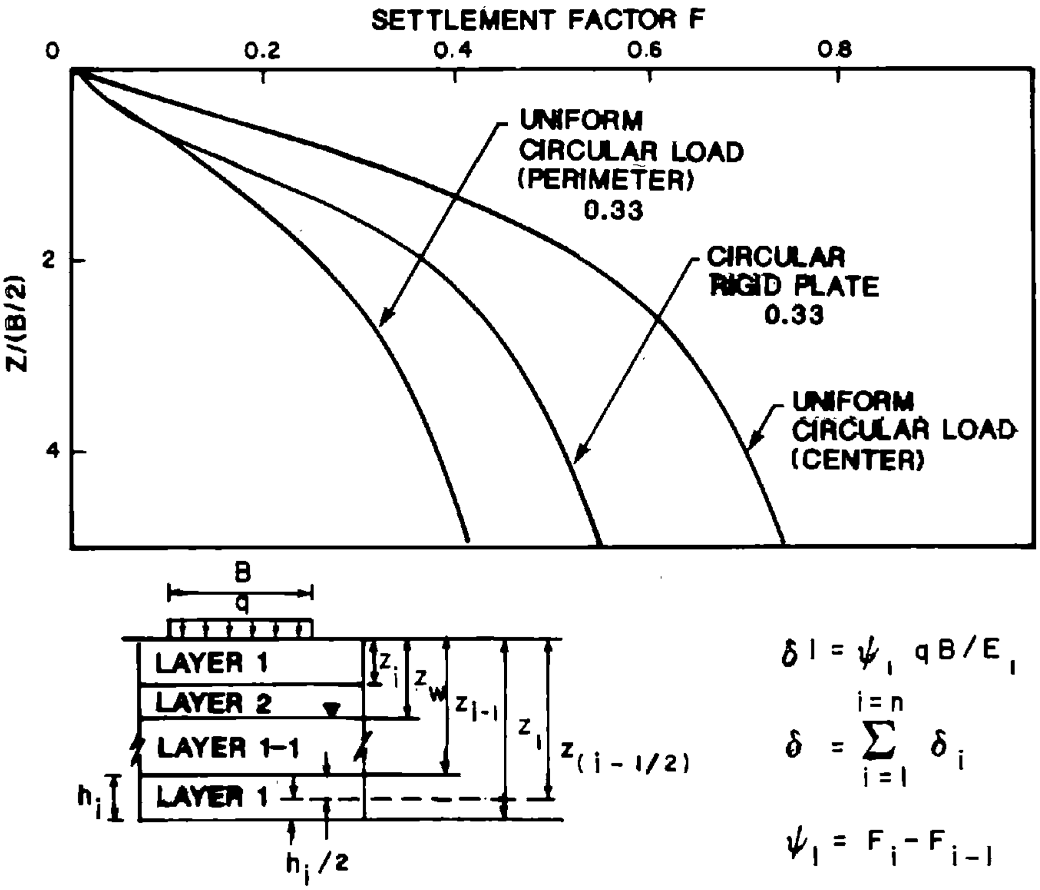
2. Method has strong theoretical basis, but requires significantly more time for the calculation than other methods, especially for parametric studies.

3. No means of distinguishing between normally consolidated and overconsolidated sands in the method is currently available.

METHOD: Oweis (continued)



Ratio $\alpha = \Delta\sigma_m/q$ beneath circular flexible foundation.



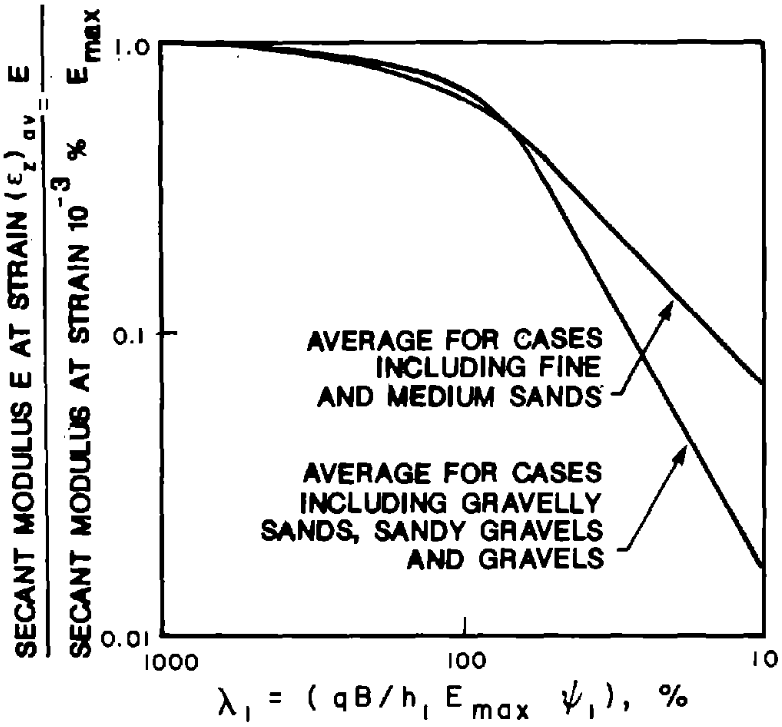
$$\delta_i = \psi_i \, qB/E_i$$

$$\delta = \sum_{i=1}^{i=n} \delta_i$$

$$\psi_i = F_i - F_{i-1}$$

Settlement factors for layered elastic solid.

METHOD: Oweis (continued)



Secant modulus reduction versus strain parameter λ_i.

METHOD: Parry

REFERENCES: Parry, R.H.G., "A Direct Method of Estimating Settlements in Sands from SPT Values," Proceedings, Symposium on Interaction of Structure and Foundation, Midland Soil Mechanics and Foundation Engineering Society, Birmingham, 1971.

BASIC EQUATION: $\rho\ (\text{mm}) = \frac{200qB}{N} C_D C_W C_T$

PARAMETERS:

q = footing bearing pressure in MN/m^2

B = footing width in meters

N = averaged SPT blow count (see Appendix A, Parry (1971))

C_D = correction factor for excavation depth (Only used if excavation is not backfilled)

$$CD = \frac{1.3\ (0.75 + D/B)}{(1 + 0.25D/B)}$$

C_W = correction factor for water table influence (only used if excavation not backfilled)

$$C_W = 1 + \frac{D_W}{D + 3/4\ B}, \quad 0 < D_W < D$$

$$C_W = 1 + \frac{D_W\ (2B + D - D_W)}{2B\ (D + 0.75B)}, \quad 0 < (D_W\text{-}D) < 2B$$

C_T = correction factor for thickness of sand layer (see figure 3, Parry (1971))

D = depth of footing embedment in meters

D_W = depth to water table in meters

COMMENTS:

1. For design, Parry recommends multiplying settlement from above equation by factor of 1.5.

2. Equation is based on elastic theory. Plate load test results used to backfigure the constant 200. Constants C_W and C_D based on study of effective stresses below footing.

3. See Parry (1971) for chart for correction factor C_T and for method of determining average SPT blow count.

METHOD: Peck and Bazaraa

REFERENCES: Bazaraa, A.R.S., "Use of the Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand," Ph.D. thesis presented to University of Illinois, at Urbana, IL (1967).

Peck, R.B. and Bazaraa, A.R.S., Discussion of "Settlement of Spread Footings on Sand" (by D'Appolonia, D'Appolonia and Brissette), Journal of Soil Mechanics and Foundation Division, ASCE, May 1969.

BASIC EQUATION: $\rho \text{ (inches)} = C_w C_D \left(\frac{2P}{N_B}\right) \left(\frac{2B}{B+1}\right)^2$

PARAMETERS:

C_w = water table correction factor = $\sigma_v/\bar{\sigma}_v$ at depth of B/2 below footing bearing level

C_D = embedment correction factor = $1-0.4 \left(\frac{\gamma D}{P}\right)^{1/2}$

σ_v = total vertical pressure

$\bar{\sigma}_v$ = effective vertical pressure

D = depth of footing embedment in feet

γ = unit weight of soil in pounds per cubic ft.

P = footing bearing pressure in tons per sq. ft. (TSF)

B = footing width in feet

N_B = SPT blow count corrected for overburden pressure. See chart included in Hough method description.

COMMENTS:

1. Method based on original Terzaghi and Peck empirical equation, but constant reduced from 3 to 2, thus reducing the predicted settlement by one-third. Also, SPT blow count is corrected for overburden pressure to obtain a soil parameter (N_B) reflecting the relative density of the soil.

2. Use of water table correction factor is controversial, but Bazaraa and Peck recommend its use in their approach.

3. See comments for Terzaghi and Peck method.

METHOD: Peck, Hanson and Thornburn

REFERENCES: Peck, R.B., Hanson, W.E. and Thornburn, T.H., "Foundation Engineering," John Wiley and Sons, Inc., New York, NY, 1973.

BASIC EQUATION: For $\rho < 1$ inch, $q_a = 0.11 C_w N_1$

PARAMETERS:

q_a = allowable soil bearing pressure in tons per sq. ft. (TSF)

N_1 = average Standard Penetration Test blow count corrected to 1 ton per sq. ft. effective overburden pressure
$N_1 = 0.77 \log (20/\bar{\sigma}_{vo})N$, $\bar{\sigma}_{vo} < 0.25$ TSF
$N_1 = 2.0N$ for $\bar{\sigma}_{vo} = 0$ TSF
$N_1 = 0.4N$ for $\bar{\sigma}_{vo} \geq 5$ TSF

C_w = water table correction factor
$C_w = 0.5 + 0.5\ (D_w/D_f + B)$

$\bar{\sigma}_{vo}$ = effective overburden pressure in TSF

D_w = depth to water table

D_f = depth of footing embedment

B = footing width

N = Standard Penetration Test blow count

COMMENTS:

1. Method is empirical, based on observations of settlement of actual footings.

2. Above equation only valid when bearing capacity of soil is adequate, usually when footing width greater than 3 to 4 ft.

3. N_1 value used in equation should be average N_1 between depths of 0 to B below footing bearing level.

4. To obtain settlement at bearing pressure q other than q_a, settlement is often calculated as q/q_a, although this approach is not discussed by Peck et al. (1973).

METHOD: Schmertmann

REFERENCES: Schmertmann, J.H., (May 1970), "Static Cone to Compute Static Settlement Over Sand," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96 (SM3), pp. 1011-1041.

Schmertmann, J.H., (July 1978), "Guidelines for Cone Penetration Test, Performance and Design," Federal Highway Administration, Report FHWA-TS-78-209, Washington, D.C., (July 1978).

Schmertmann, J.H., Hartman, J.P., Brown, P.B., (Aug. 1978), "Improved Strain Influence Factor Diagrams," Journal Geotechnical Engineering, ASCE, Vol. 104 (GT8), pp. 1131-1135.

BASIC EQUATION: $\rho = C_1 C_2 \, \Delta P \sum_{0}^{2B} \frac{I_z \Delta_z}{E_s}$

PARAMETERS:

B = footing width

I_z = strain influence factor

Δ_z = thickness of layer of constant E_s

ΔP = net bearing pressure = $P - P_0$

E_s = soil modulus

P = footing bearing pressure

P_0 = initial effective overburden pressure at footing bearing level

C_1 = embedment correction = $1 - 0.5\,(P_0/\Delta P)$, $C_1 \geq 0.5$

C_2 = creep correction factor = $1 + 0.2 \log (10t)$

t = time in years after load applied to footing

COMMENTS:

1. Strain influence factors and relationships between E_s and cone penetration resistance are given in Schmertmann 1970 and presented below.

2. Method based on observation of distribution of vertical strain vs. depth in model and finite element method studies. The method is empirical, but has a theoretical basis.

METHOD: Schmertmann (continued)

3. If only Standard Penetration Test results are available, these must be converted to cone penetration resistance by empirical relationships (see below). The unknown reliability of such conversions results in additional uncertainty in settlement predictions.

4. The method is applicable only to normally consolidated sands.

5. The creep correction is sometimes regarded as being too conservative and is ignored.

6. Harr (1966) has proposed alternative strain influence factors based on his probabilistic soil theory. His strain influence factors are strongly dependent on the coefficient of at-rest lateral earth pressure (K_0) for the sand.

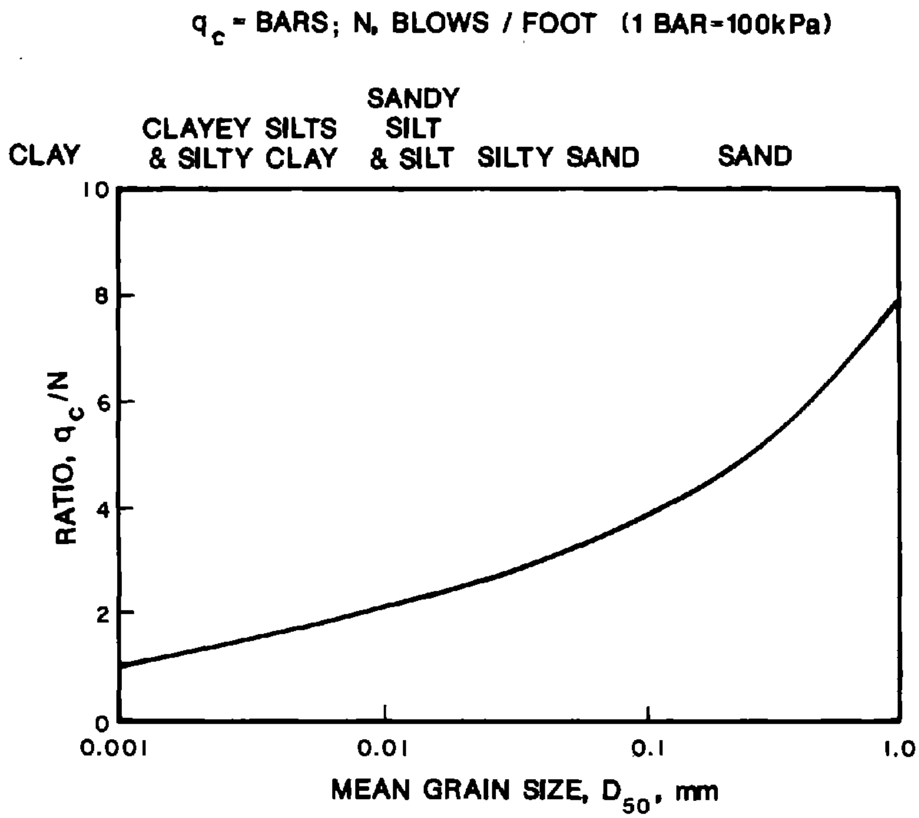
Relationship between equivalent Young's Modulus (Es) and static Dutch cone bearing capacity (q_c) (kg/cm^2). Schmertmann 1970.

For footing length to width ration (L/B):	1	$Es = 2.5q_c$
	10	$Es = 3.5q_c$
	$1 < \frac{L}{B} < 10$	interpolate between $2.5q_c$ and $3.5q_c$

q_c/N Ratio: Schmertmann 1970.

Soil Type	q_c/N
Silts, sandy silts, slightly cohesive silt-sand mixtures	2.0
Clean, fine to medium sands and slightly silty sands	3.5
Coarse sands and sands with little gravel	5
Sandy gravels and gravel	6

METHOD: Schmertmann (continued)



q_c/N Ratio: Robertson a Campanella 1983.

METHOD: Schultz and Sherif

REFERENCES: Schultz, E. and Sherif, G., "Prediction of Settlements from Evaluated Settlement Observations for Sand," Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, 1973.

BASIC EQUATION:

$$\rho\ (\text{cm}) = \frac{p\ f\ B}{1.71\ N^{0.87}\ /\sqrt{(B/B_1)}\ (1 + 0.4\ t/B)}$$

PARAMETERS:

P = footing bearing pressure in kg/cm^2

f = correction factor for footing shape and thickness of sand stratum

B = footing width in cm

B_1 = 1 cm

N = SPT blow count

t = depth of footing embedment in cm

COMMENTS:

1. Started with elastic theory equation then performed statistical study of 48 measurements of footing and plate settlement to obtain the soil modulus as a function of N.

2. Influence factor f depends on the ratio of thickness of compressible layer to foundation width (ds/B). It can be found in tables for elastic isotropic half-space. (Steinbrenner 1934, Kany 1959, etc.) (see Schultz and Sherif (1973)).

METHOD: Terzaghi and Peck

REFERENCES: Terzaghi, Karl and Peck, R.B., "Soil Mechanics in Engineering Practice," John Wiley and Sons, Inc., New York, NY, 1948.

BASIC EQUATION: $\rho \text{ (inches)} = C_W C_D \left(\frac{3P}{N}\right) \left(\frac{2B}{B+1}\right)^2$

PARAMETERS:

C_W = water table correction factor
- C_W = 1.0 if water table at depth greater than 2B below footing
- C_W = 2.0 if water table at ground surface

C_D = embedment correction factor = $1 - \frac{D}{4B}$

B = footing width in feet

D = depth of footing embedment in feet

P = footing bearing pressure in tons per sq. ft.

N = SPT blow count

COMMENTS:

1. Empirical method based on observed settlement of footings on sand. Method was intended to provide an upper bound, or highest value of settlement to be expected. Predicted settlements using this method are, therefore, likely to be very conservative compared to typical measured settlements.

2. Water table correction is controversial, often considered too conservative.

3. The relationship between settlement and footing width is highly variable and is dependent on soil type and relative density. The $(2B/B+1)^2$ factor in the above equation has been shown to be unconservative under some soil conditions for large values of B, and should be used with caution.

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Calculated Applied Load - West Abutment

$\gamma_{conc} = 150\ pcf$

$\gamma_{backfill} = 120\ pcf$

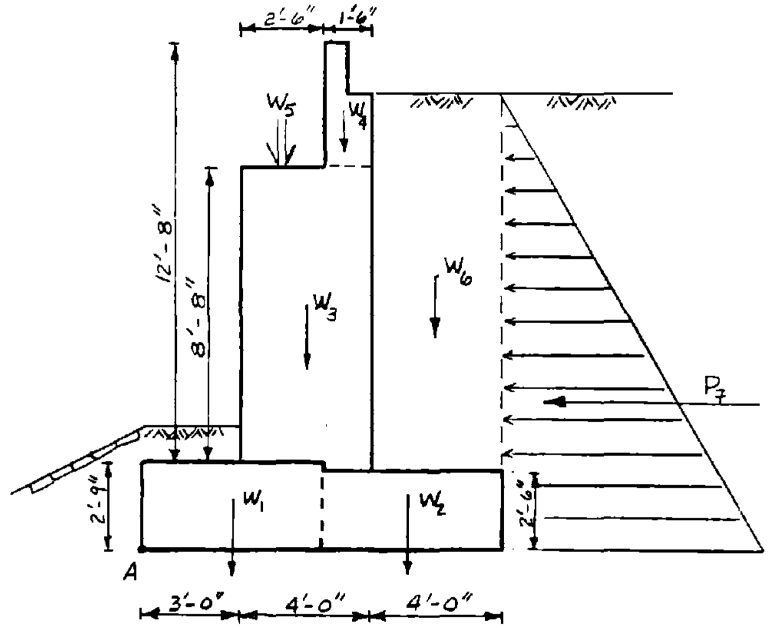
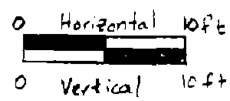


Figure 1



Structural Loads (W_5)

Deck Load 101.75 C.Y.
Steel 58.4 TONS

Footing Dimensions:

Length (L) = 74.6 ft
Width (B) = 11.0 ft

Vertical Load

$W_1 = (2.75')(5.5')(.150\ kcf) = 2.27\ k/ft\text{-}wall$

$W_2 = (2.50')(5.5')(.150\ kcf) = 2.06\ k/ft\text{-}wall$

$W_3 = (4.0')(8.6')(.150\ kcf) = 5.16\ k/ft\text{-}wall$

$W_4 = (1.5')(4.0')(.150\ kcf) = 0.90\ k/ft\text{-}wall$

$$W_{5_{DECK}} = \frac{(101.75\ c.y.)\left(\frac{27\ ft^3}{c.y.}\right)(0.150\ kcf)}{74.6\ ft} = 5.52\ k/ft\text{-}wall$$

$$W_{5_{STEEL}} = \frac{(58.4\ TON)(2.0\ ^{KIP}/_{TON})}{74.6\ ft} = 1.56\ k/ft\text{-}wall$$

$W_6 = (4.0')(11.2')(.120\ kcf) = 5.38\ k/ft\text{-}wall$

$$\sum_{i=1}^{6} W_i = 22.9\ k/ft\text{-}wall$$

Lateral Earth Pressure

$P_7 = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2}(0.33)(.120\ kcf)(13.7\ ft)^2 = 3.72\ ^{k}/_{ft\text{-}wall}$

MOMENTS ABOUT THE TOE (PT. A)

	FORCE	* ARM	= MOMENT
W_1	2.27 k	$(5.5'/2)$	= 6.2 k-ft
W_2	2.06 k	$(5.5' + \frac{5.5}{2})$	= 17.0 k-ft
W_3	5.16 k	$(3.0' + \frac{4.0'}{2})$	= 25.8 k-ft
W_4	0.90 k	$(3.0' + 2.5' + 1.5/2)$	= 5.6 k-ft
W_5	5.52 k + 1.56 k	$(3.0' + 2.5/2)$	= 30.1 k-ft
W_6	5.38 k	$(3.0' + 4.0' + 4.0/2)$	= 48.4 k-ft

$$\sum_{i=1}^{6} M = 133. \text{ K-ft}$$

Overturning Moment

$$O = P_7 * H/3 = (3.72 \text{ K})(13.7'/3) = 17.0 \text{ k-ft}$$

Location of Resultant

$$\bar{x} = \frac{\sum_{i=1}^{6} M_i - O}{\sum_{i=1}^{6} W_i} = \frac{133.1 \text{ k-ft} - 17.0 \text{ k-ft}}{22.9 \text{ ft}}$$

$\bar{x} = 5.08$ ft O.K. middle third of footing

Pressure @ Toe

$$q = \frac{P}{A} + \frac{M}{S}$$

Rectangle
$$S = \tfrac{1}{6} B^2$$
$$S = \tfrac{1}{6} (11.0 \text{ ft})^2$$
$$S = 20.2 \text{ ft}^2$$

Moment
$$M = \left(\sum_{i=1}^{6} W_i\right)\left(\frac{B}{2} - \bar{x}\right)$$
$$M = (22.9 \text{ k})(11.0 \text{ ft}/2 - 5.06 \text{ ft})$$
$$M = 10.1 \text{ k-ft}$$

Pressure
$$q = \frac{22.9 \text{ K/ft-wall}}{11.0 \text{ ft}} + \frac{10.1 \text{ K-ft/ft-wall}}{20.2 \text{ ft}^2}$$

$q = 2.1\ ksf + 0.5\ ksf$

$q_{TOE} = 2.6\ ksf$

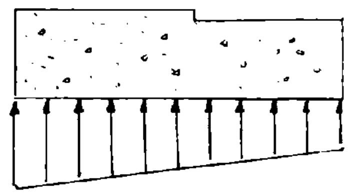


Figure 2
(NTS)

Pressure @ Heel

$q_{Heel} = 2.1\ ksf - 0.5\ ksf$

$q_{Heel} = 1.6\ ksf$

Average Applied Pressure $q_{ave} = 2.1\ ksf$

NOTE: Measured settlement values have been recorded by optical settlement points installed on the stem wall of the abutment. These points were installed after footing construction therefore the measured values reflect settlement after footing construction

$\therefore\ q_{ave}$ - minus the weight of the footing

$$2.1\ ksf - ((\overset{w_1}{2.27\ k/ft\ wall} \overset{}{+} \overset{w_2}{2.06\ k/ft\ wall}) / 11.0\ ft)$$

q_{ave} - minus the weight of the footing = 1.70 ksf (0.85 kgf/cm²)

This value will be used in settlement predictions and compared with measured data.

PreConstruction Design Recommendations

Cantilever wall design:

allowable bearing pressure = 5.00 ksf
maximum applied pressure = 2.60 ksf

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I. BURLAND METHOD

Settlement = f (subgrade compressibility, mean SPT value)

Governing Eqn.

$$\rho = f_s \cdot f_l \cdot f_t \left[\left(q' - \tfrac{2}{3}\sigma'_{vo} \right) \times B^{0.7} \times I_c \right] \qquad (1)$$

ρ = settlement (mm)
q' = average gross effective applied pressure (kN/m²)
σ'_{vo} = maximum previous effective overburden pressure (kN/cm²
B = footing width (m)
I_c = compressibility index
f_s = shape correction factor
f_l = correction factor for thickness of the sand layer
f_t = time factor

NOTE a.) For normally consolidated sand the immediate average settlement may be calculated as

$$\rho = f_s \cdot f_l \cdot f_t \left[q' \times B^{0.7} \times I_c \right] \quad (mm) \qquad (2)$$

b.) For overconsolidated sand, or for loading at the base of an excavation for which the maximum previous effective overburden pressure is σ'_{vo} use equation (1).

c.) For overconsolidated sand when q' is less than σ'_{vo} equation (1) becomes

$$\rho = f_s \cdot f_l \cdot f_t \left[q' \times B^{0.7} \times \frac{I_c}{3} \right] \quad (mm) \qquad (3)$$

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Correction Factors

1) shape (f_s)

$$f_s = \left[\frac{1.25\ L/B}{L/B + 0.25} \right]^2 \qquad (4)$$

L = length (m)
B = width (m)

2) sand layer thickness (f_l)

$$f_l = \frac{H_s}{z_I} \left[2 - \frac{H_s}{z_I} \right] \qquad (5)$$

H_s = thickness of sand and/or gravel layer (m)
z_I = depth of influence (m)

3) time (f_t)

$$f_t = \left[1 + R_3 + R \log \frac{t}{3} \right] \qquad (6)$$

$t \geq 3$ yrs.

for static loads $R_3 = 0.2$, $R = 0.3$

for fluctuating loads $R_3 = 0.7$, $R = 0.8$

PROCEDURE

1.) Determine the depth of influence below the base of the footing.

depth of influence (z_I) = f (SPT)

NOTE: a.) when N-values increase or are constant with depth z_I is given by the full line in Figure 3 Burland (1,

b.) where N-values show a consistent decrease with depth z_I is taken as 2B or the bottom of the soft layer whichever is the lesser.

Consider Boring B1 (May 1986)

The N-values have been plotted to a depth corresponding to 2B below the base of the footing.

The trend in the blow counts is relatively consistent or increasing with depth.

∴ z_I obtained from Fig. 3 Burland (1)

$B = 11.0ft\ (3.4m)$
$z_I = 2.7m\ (9ft)$

N_{ave} within depth of influence

$\bar{N} = N_{ave} = 18$

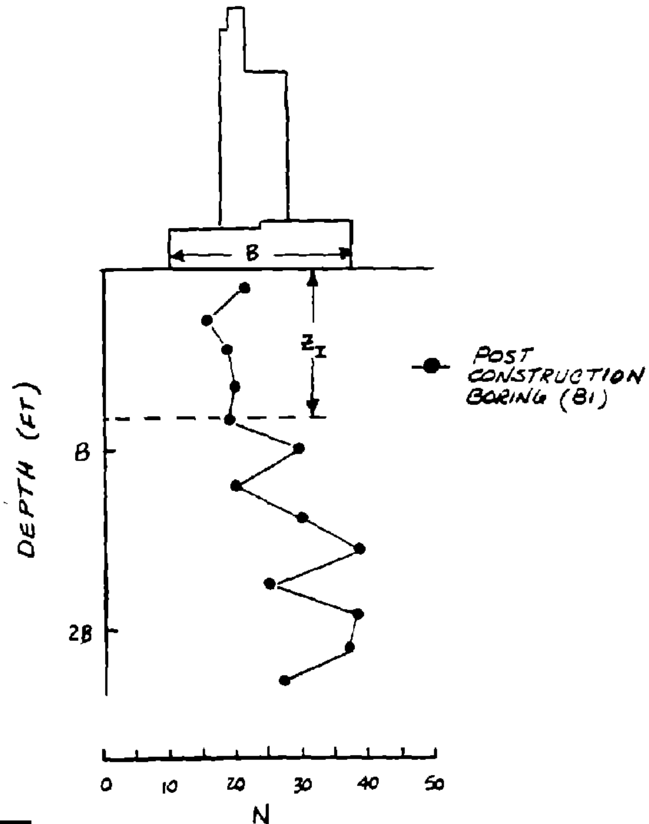

Figure 3

2.) Determine compressibility factor (I_c)

From Burland (1)

$$I_c = \frac{1.71}{\bar{N}^{1.4}}$$

$$I_c = \frac{1.71}{(18)^{1.4}} = \boxed{3.0 \times 10^{-2}}$$

3.) Determine shape correction factor (f_s)

from statistical analysis the author concludes that there is a significant correlation between settlement and L/B (length to width ratio)

$$f_s = \left[\frac{1.25\ L/B}{L/B + 0.25}\right]^2 = \left[\frac{(1.25)\frac{22.7m}{3.4m}}{\frac{22.7m}{3.4m} + 0.25}\right]^2 = \boxed{1.45}$$

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4) Determine sand layer thickness correction factor (f_l)

the author recommends a correction when H_s (the thickness of sand or gravel layer beneath the footing) is less than z_I (depth of influence)

∴ @ Branch Avenue $H_s >> z_I$
∴ $f_l = \boxed{1.0}$

5) Determine time factor (f_t)

based on case history studies the author suggests a time factor (f_t) for foundations subject to fluctuating loads (i.e. bridges, for time greather than 3 years

∴ @ Branch Avenue elapsed time $t \approx 2.5$ yrs.
∴ $f_t = \boxed{1.0}$

6) Calculate settlement - normally consolidated sand use equation (2)

$B = 11.0$ ft (3.4m)
$q' = 1.70$ ksf (85 kN/m²)

$$\rho = f_s \cdot f_l \cdot f_t \cdot [\, q' \times B^{0.7} \times I_c \,]$$

$$\rho = (1.45)(1.0)(1.0)\ [(850\,\mathrm{kN/m^2}) \times (3.4\,m)^{.7} \times (3.0 \times 10^{-2})]$$

$\rho = 8.7$ mm
$\boxed{\rho = 0.34 \text{ inch}}$

For the west abutment, the average total measured settlement is $\boxed{0.42 \text{ inch}}$ to date.

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II. D'APPOLONIA METHOD

D'Appolonia's (et. al.) approach to estimating settlement of footing on sand by applying SPT data is based on the theory of elasticity.

Governing eqn.

$$S = \mu_0 \mu_1 \frac{qB}{M} \qquad (1)$$

S = footing settlement (ft)
q = average applied bearing pressure (tsf)
B = footing width (ft)
M = Modulus of compressibility (tsf)
μ_0 = embedment influence factor
μ_1 = compressible strata influence factor

PROCEDURE

1.) Determine embedment influence factor (μ_0)

$\mu_0 = f$ (depth of embedment with respect to original ground surface (D), footing width (B))

Compute D/B and obtain μ_0 from Figure 6 of Christian & Carrier (3)

@ West Abutment - Fill condition $D = 0$
$\therefore$ from Fig. 6 with $D=0$; $\boxed{\mu_0 = .97}$

2.) Determine compressible strata influence factor (μ_1)

$\mu_1 = f$ (depth to incompressible layer below base of footing (H), footing width (B), footing length (L))

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compute H/B and L/B and obtain μ_1 from Fig. 6 of Christian & Carrier (3)

@ West Abutment - $H = 155$ ft. refer to boring log H-2A
$B = 11.0$ ft.
$L = 74.6$ ft.

$$\frac{L}{B} = \frac{74.6}{11.0} = 6.8 \ ; \ \frac{H}{B} = \frac{155}{11} = 14.0 \ ; \ \boxed{\mu_1 = 1.25}$$

3) Determine Modulus of compressibility (tsf)

The authors state that compressibility and SPT are related because both variables are associated to the same variables. Specifically, they are related to relative density, effective overburden stress, and sand grain size, shape, and distribution.

To correlate compressibility and SPT use the average SPT in the zone of influence (taken as depth B) below the base of the footing

@ West Abutment $N_{ave} = 20$

from Fig. 44 - D'Appolonia (2) for normally loaded sand

$\boxed{M = 270 \text{ tsf}}$

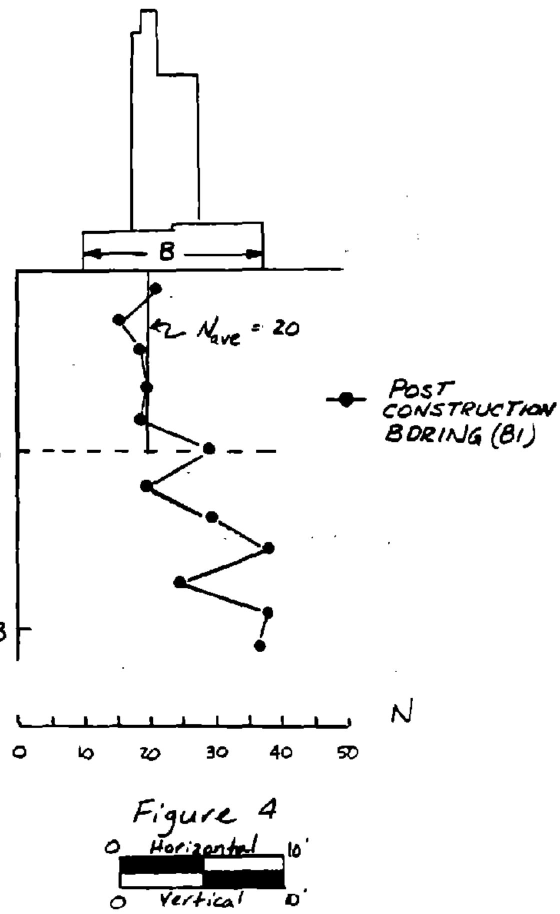

Figure 4

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4) Calculate Settlement.

$$S = \mu_0 \mu_1 \frac{q B}{M}$$

$$S = \frac{(0.97)(1.25)(0.85\ tsf)(11.0 ft)}{270\ tsf}$$

$$S = 0.042\ ft$$
$$\boxed{S = 0.50\ inch.}$$

For the west abutment, the average total measured settlement is $\boxed{0.42\ inch}$ to date

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III. HOUGH METHOD

The author states that settlement of spread footings on granular soils is usually elastic and consolidation occurs immediately on the application of the load.

Governing Eqn.

$$\Delta H = H \frac{1}{C'} \log \frac{P_o + \Delta P}{P_o} \qquad (1)$$

ΔH = settlement (ft)
H = thickness of the soil layer considered (ft)
P_o = existing effective overburden pressure @ center of the considered layer (ksf)
C' = Bearing Capacity Index

ΔP = Distributed footing pressure @ center of considered layer (ksf)

Distributed Loads:

$$\Delta P = \frac{P_{applied}}{(B+h)^2} \qquad \text{square footing (2a.)}$$

$$\Delta P = \frac{P_{applied}}{(B+h)(L+h)} \qquad \text{rectangular footing (2b.)}$$

h = mid-depth of the layer considered (ft)

NOTE Depth of significant stress is the depth above the level where

$$\frac{\Delta P}{P_o} = \frac{1}{10} \qquad (3)$$

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PROCEDURE

1.) the author suggests constructing table 2 to organize calculations for settlement.

The information listed below will aid in the development of Table 2.

1a.) Plot N-values vs. depth below the base of the footing

Assume : $\gamma_{FILL} = 0.120$ Kcf

$\gamma_{OUTWASH} = 0.118$ Kcf

1b.) $P_{applied} = (q')(L)(B)$
$P_{applied} = (1.70 \text{ Ksf})(74.6 \text{ ft})(11.0 \text{ ft})$

$P_{applied} = 1400$ Kips

1c.) Use Figure 12 - Hough (12) to obtain the ratio N'/N

1d.) Use Figure 5 at the right to approximate N for the depth consider. NOTE: the limit of excavation and its effect on the approximation of pre-const. borings.

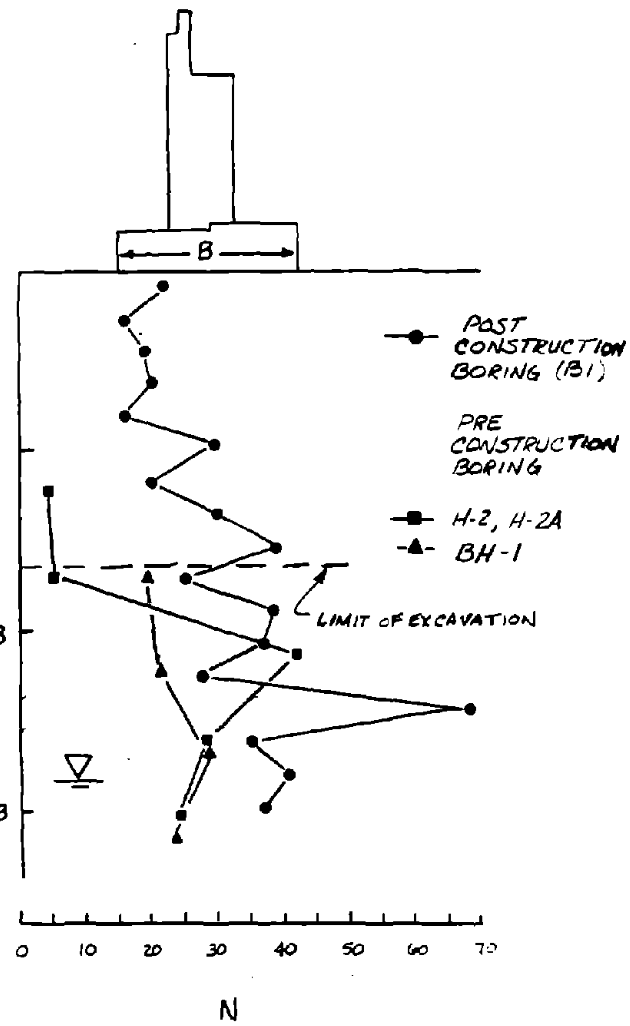
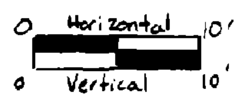


Figure 5



1e.) Use Figure 13 - Hough (12) to obtain C'
assume "well graded fine to coarse sand" for the fill material

assume "clean uniform fine sand" for the natural outwash material although this classification is not specifically plotted in Figure 13 - assume it between "clean uniform med. sand"

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TABLE 1

(1) MID DEPTH (ft)	(2) P_o (Ksf)	(3) ΔP (Ksf)	(4) $\frac{\Delta P}{P_o}$	(5) $\log\left(\frac{\Delta P}{P_o}+1\right)$	(6) N'/N	(7) N	(8) N'	(9) C'	(10) H (ft)	(11) ΔH * (in)
4.5	1.12	1.14	1.02	0.30	1.23	18	22	85	9	0.40
13.5	1.62	0.64	0.40	0.15	0.99	29	29	108	9	0.15
22	2.63	0.44	0.17	0.07	0.87	28	24	88	8	0.08 (0.04
30	3.86	0.33	0.08	0.03	0.76	31	24	88	8	0.03 (0.02

* Natural material considered preloaded by the old abutment and fill.

Σ = 0.61 inch

2.) Calculate Settlement

Column 11 in Table 2 above is the computation of settlement based on equation 1 for this method.

$\boxed{\Delta H = 0.61 \text{ inch}}$

For the west abutment, the average total settlement is $\boxed{0.42 \text{ inch}}$ to date.

NOTE: The depth of significant stress extended through fill material and natural outwash sands. The natural outwash sand has been preloaded and the H&A procedure is to reduce the settlement predictions in these soil in half as shown in rows 3 and 4 of column 11 above.

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IV PECK & BAZARAA METHOD

Modification of the original Terzaghi & Peck (1948) and Gibbs & Holtz procedures for predicting the settlement of spread footings on sand.

Governing Eqn.

$$S = K C_d \frac{2q}{N_c} \left(\frac{2B}{B+1} \right)^2 \qquad (1)$$

S = settlement (inches)
q = applied bearing pressure (tsf)
B = footing width (ft)
N_c = corrected N-value
K = groundwater correction factor
C_d = embedment correction factor

Correction Factors

a.) groundwater

$$K = \frac{P_o \text{ @ depth } 0.5B \text{ below the footing assuming no water}}{P_o \text{ @ depth } 0.5B \text{ below footing for existing water level}}$$

P_o = overburden pressure

b.) embedment

$$C_d = 1.0 - 0.4 \left(\frac{\gamma D}{q} \right)^{1/2} \qquad (3)$$

γ = assumed unit weight of soil above the footing (Ksf)

D = depth of embedment (ft) with respect to the orignal ground surface c.) N-value

$$N_c = \frac{4N}{1+2p_o'} \quad \text{for } p_o' \leq 1.5 \text{ ksf} \quad (4a.)$$

$$N_c = \frac{4N}{3.25+0.5p_o'} \quad \text{for } p_o' \geq 1.5 \text{ ksf} \quad (4b.)$$

where N = average measured N-value for the depth B below the base of footing

p_o' = effective overburden pressure at mid-depth
p_o' = 1500 psf corresponds to the pressure @ the normal depth for most shallow footings.

PROCEDURE

Refer to Figure 4 in this handout for applicable blow-counts below the base of the footing.

1) Calculate the groundwater correction factor (if applicable)

@ West Abutment - measured groundwater table at approximately El. 22.5 - in other words, 31^{+} ft below the base of the footing.

$\therefore \boxed{K = 1.0}$

2.) Calculate the embedment correction factor (if applicable)

@ West Abutment - fill condition such that the finish grade is approximately 12 ft above the previous existing grade. Since the embedment correction factor is really a reduction factor - it is not applicable in fill situations similar to the conditions at this abutment.

$\therefore \boxed{C_d = 1.0}$

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3.) Calculate the corrected N-value.

@ West Abutment - fill material down to a depth B below the base of the footing.

$\bar{N} = 20$

the existing effective overburden pressure at mid-depth 0.5 B below base of footing

$$p_o = (5.5\,ft)(0.120\,kcf) = 0.66\,ksf \leq 1.5\,Ksf$$

∴ from (4a.)

$$N_c = \frac{4(20)}{1 + 2(0.66\,ksf)}$$

$$\boxed{N_c = 34}$$

4) Calculate settlement using eqn (1)

$$s = K C_d \frac{2q}{N_c'} \left(\frac{2B}{B+1}\right)^2$$

$$s = (1.0)(1.0)\ \frac{2\,(0.85\,tsf)}{34} \left[\frac{2\,(11.0')}{11.0' + 1}\right]^2$$

$$\boxed{s = 0.17\ inch}$$

For the west abutment, the average total settlement is $\boxed{0.42\ inch}$ to date.

I. SCHMERTMANN METHOD

Settlement = f (static cone bearing capacity q_c)
from Dutch static cone field measurements

Governing Equation :

$$S = C_1 C_2 \Delta p \sum \frac{I_z}{E_s} \Delta z \qquad (1)$$

where :

S = settlement (cm)
Δp = net foundation pressure increase (Kg/cm²)
I_z = strain influence factor

E_s = Equivalent Young's Modulus (Kg/cm²)

Δz = change in depth (cm)

C_1 = linear correction factor
to conform to arching-compression relief concept

$$C_1 = 1 - 0.5 \frac{P_o}{\Delta p} \qquad (2)$$

C_2 = creep correction factor

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right) \qquad (3)$$

t = elapsed time (yr)

PROCEDURE

1) Construct strain influence diagram

Refer to Schmertmann et al. (4) Figure 4 for recommended modified values for Strain Influence Factor Diagram and matching Sand Moduli

Requirements : a) strain influence factor
b) peak influence factor and corresponding depth
c) relative depth

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For a square, axisymmetric footing ($L/B = 1.0$) the authors recommends the following:

- strain influence factor, $I_{z@0} = 0.1$
- relative depth below footing = 2B
- equivalent Young's Modulus = $2.5 q_c$

For a rectangular, plane strain footing ($L/B \geq 10$) the authors recommends the following:

- strain influence factor, $I_{z@0} = 0.2$
- relative depth below footing = 4B
- equivalent Young's Modulus = $3.5 q_c$

1a.) Strain influence factor - west abutment, Branch Avenue

$$\frac{L}{B} = \frac{74.6 ft}{11.0 ft} = 6.8$$

Interpolate between $I_z = 0.1$ ($L/B = 1.0$) and $I_z = 0.2$ ($L/B = 10.0$)

Consider point A

$$A = 0.1 + \frac{\frac{L/B - 1}{9}}{10}$$

$$A = 0.1 + \frac{\frac{6.8-1}{9}}{10}$$

$A = 0.1 + 0.06$

$\boxed{A = 0.16}$

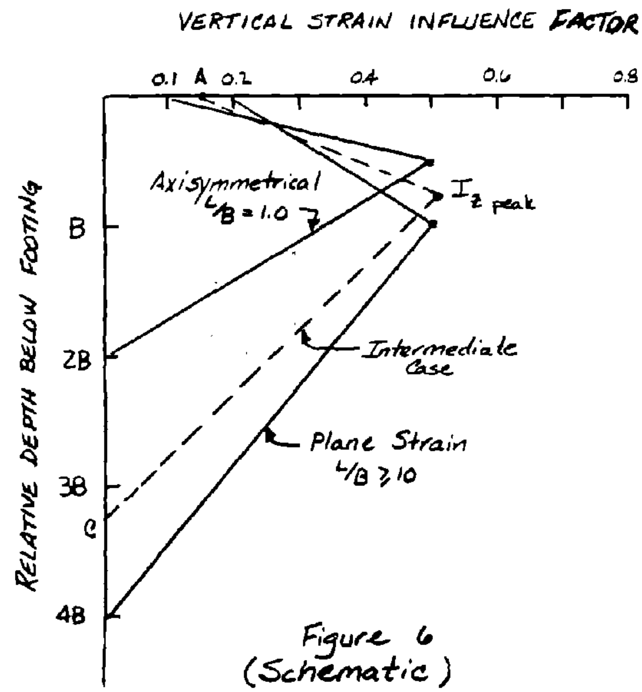

Figure 6
(Schematic)

Schmertmann (18)

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1b) Peak influence factor and corresponding depth - Branch Avenue

Refer to Schmertmann et. al. (4)
the authors suggest the following

$$I_{zp} = 0.5 + 0.1\left(\frac{\Delta P}{\bar{\sigma}_{vp}}\right)^{1/2} \qquad (4)$$

$\Delta P = p - p_o$ where p = applied pressure
p_o = effective stress at depth of footing
$\bar{\sigma}_{vp}$ = effective vertical pressure at depth to I_{zp}

Depth to I_{zp}; interpolate between B/2 and B.

$$Depth = \left(0.5 + \frac{\frac{L/B - 1}{9}}{2}\right) B$$

$$Depth = \left(0.5 + \frac{\frac{6.8-1}{9}}{2}\right) B$$

$$Depth = 0.82\,B$$

$$Depth = 0.82\,(11.0\,ft)$$

$$\underline{\underline{Depth = 9.02\,ft.}}$$ below base of footing -

∴ At the toe of the abutment ... 3.9 ft of fill placed above the base of the footing. Therefore, 12.9 ft of fill above depth of I_{zp}

$$\bar{\sigma}'_{vp} = (12.9\,ft) \times (.120\,kcf)$$

$$\underline{\underline{\bar{\sigma}'_{vp} = 1.55\,ksf \quad (0.78\,Kg/cm^2)}}$$

Magnitude of I_{zp}; $p = 1.69\,ksf$
$p_o = 0$ - fill condition

$$\therefore \underline{\underline{\Delta p = 1.69\,ksf \quad (0.84\,Kg/cm^2)}}$$

$$I_{zp} = 0.5 + 0.1\left(\frac{0.84\,Kg/cm^2}{0.78\,Kg/cm^2}\right)^{\frac{1}{2}} = \boxed{0.60}$$

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1c.) Relative depth below base of footing (Point C - Figure 6)

Interpolate between 2B (axisymmetrical) and 4B (plane strain)

$$\therefore \quad C = 2B + 2\left[\frac{L/B - 1}{9}\right] B$$

$$C = 2B + 2(0.64)B$$
$$C = 3.28B$$
$$C = 3.28(11 \text{ ft})$$
$$\boxed{C = 36.1 \text{ ft.}}$$

2.) Determine Equivalent Young's Modulus (E_s)

The authors suggest:

$$E_s = 2.5\,q_c \quad \text{for} \quad L/B = 1.0$$
$$E_s = 3.5\,q_c \quad \text{for} \quad L/B \geq 10.0$$

Interpolate:

$$E_s = 2.5\,q_c + \left[\frac{L/B - 1}{9}\right] q_c$$

$$E_s = 2.5\,q_c + (0.64)\,q_c$$
$$\boxed{E_s = 3.14\,q_c}$$

3) Calculate the Correction factors C_1, C_2

$$C_1 = 1.0 - 0.5\left(\frac{P_o}{\Delta P}\right)$$

$\boxed{C_1 = 1.0}$ $\quad P_o = 0$ fill condition

$$C_2 = 1.0 + 0.2 \log\left(\frac{t}{0.1}\right)$$

start of construction November, 1983
elapsed time; May 1, 1986 897 days (2.5 yrs.)

$$C_2 = 1.0 + 0.2 \log\left(\frac{2.5}{0.1}\right) = \boxed{1.28}$$

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4) Plot measured static cone resistance q_c (Kg/cm²)
Plot strain influence triangle

Divide the q_c profile into a convenient number of layers, each with constant $\bar{q}_c$, over the depth interval 0 to point C

- Refer to cone test CP-1 May, 1986 for actual test readings.

- Constant $\bar{q}_c$ value below 2B level has been approximated from cone tests run at corresponding depths at Pier 1.

DEPTH	Measured $\bar{q}_c$ (Kg/cm²)	Approximated* q_c (Kg/cm²)
0-5	28	35
5-10	61	35
10-19	90	35/40
19-36	125	112/126

* Approximated q_c values were obtained from SPT-CPT correlations discussed in Robertson et. al. (1983) using D_{50} from the grain size curve for the outwash sand samples tested by H&A

$$\frac{q_c}{N} = 4.5$$

based on D_{50} grain size

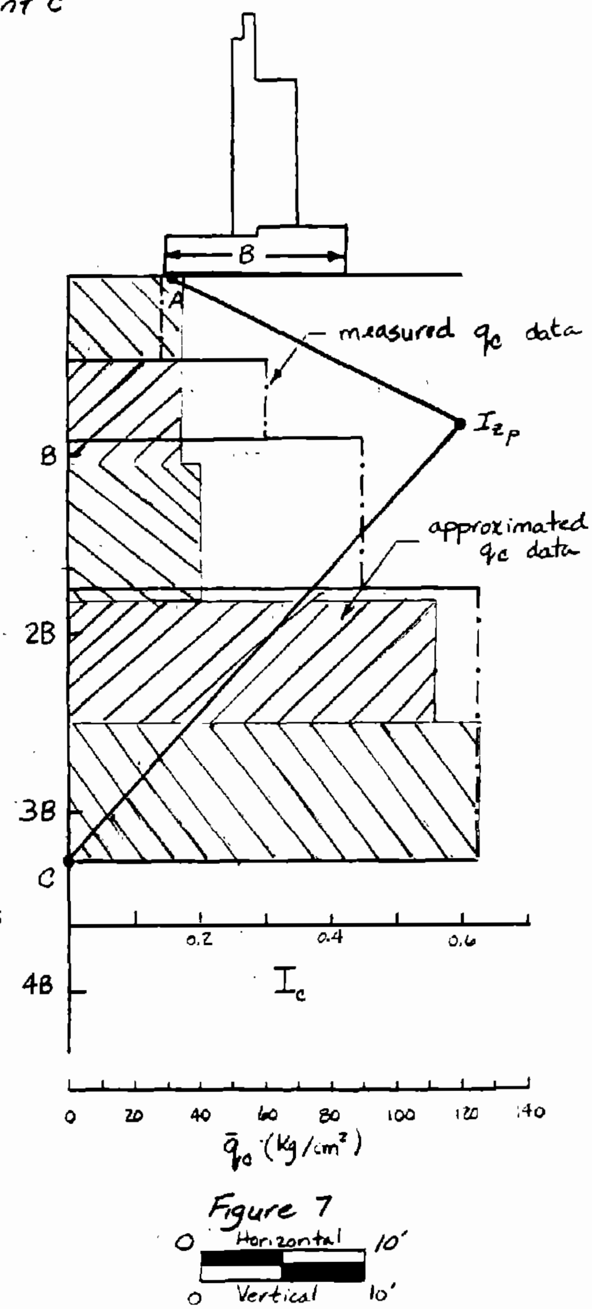

Figure 7
0 Horizontal 10'
0 Vertical 10'

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5) Construct table 3 for settlement calculation.

TABLE 2

(1) Layer	(2) Δz ft	(2) Δz cm	(3) $\bar{q}_c$ (a) Kg/cm²	(4) E_s (b) Kg/cm²	(5) $z_{\mathbb{C}}$ ft	(6) I_z (c)	(7) $\left(\frac{I_z}{E_s}\right)\Delta z$ cm/Kg/cm²
1	5.3	162	28	88	2.7	0.27	0.50
2a	3.7	113	61	192	7.2	0.52	0.31
2b	1.6	49	61	192	9.8	0.58	0.15
3	8.7	265	90	283	15.0	0.47	0.44
4	16.8	512	125	393	27.7	0.17	0.22
Total							1.62

a. $\bar{q}_c$; see figure 7
b. $E_s = 3.14\,\bar{q}_c$
c. I_z ; see figure 7

6) Calculate settlement using equation (1)

$$s = (1.0)(1.28)(0.84\ \text{Kg/cm}^2)(1.62\ \text{cm/Kg/cm}^2)$$

$$s = 1.7\ \text{cm}$$

$$\boxed{s = 0.70\ \text{inches}}$$

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The author suggests that the settlement calculation previously described should be used only with first-loading cases with adequate bearing capacity. The author states "that if the sand has been pre-strained by previous footings or other loads ... then real settlements will likely be significantly less this predicted method."

The author suggests that settlement predictions be reduced by one-half to account for preloading.

West Abutment

Settlement predictions at the west abutment are relative to a depth 3.28 B (36.1 ft) below the base of the footing. This is well below the "new" embankment placed for the construction of the "new" abutment. In other words approximately half of the relative depth is made up of preloaded natural outwash sands. Therefore, we will reduce the predicted settlement method by 25% to coincide with the discussion above and the amount of preloaded soil considered

$$s = (0.75)(.70\,\text{inch})$$

$$\boxed{s = 0.52\ \text{inch}}$$

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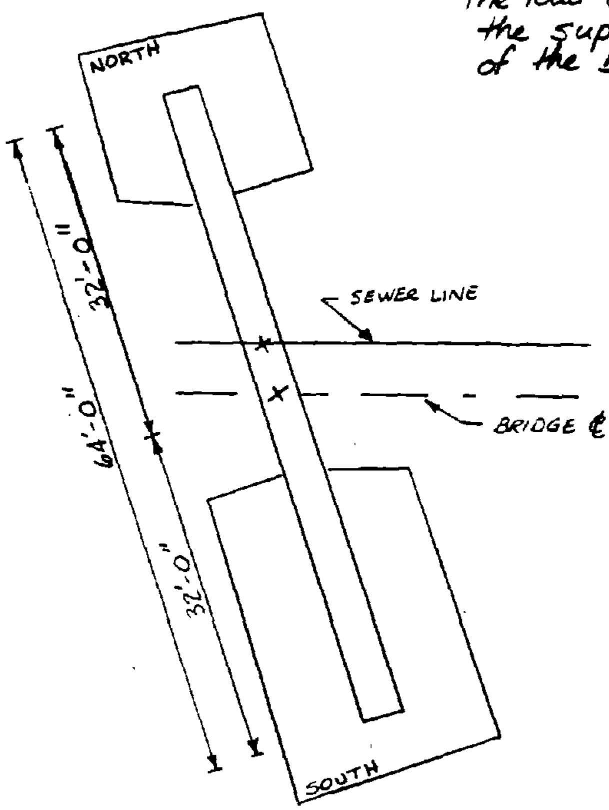
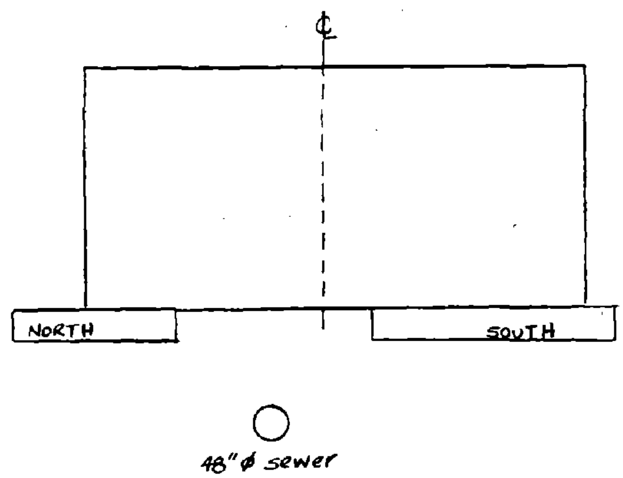
Pier 1 Calculations

Due to the presence of the 48 inch diameter sewer running adjacent to the bridge alignment the pier foundations implemented a double footing arrangement (north and south) to support the loads of the interior spans. (see figure 8 below)

The load distributed to each footing of Pier 1 from the superstructure is equal due to the location of the bridge center line.



(PLAN)



(CROSS-SECTION)

Figure 8
(NTS)

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Calculated Applied Loads - Pier 1

Assume

$\gamma_{conc} = 150 pcf$

$\gamma_{backfill} = 120 pcf$, $\gamma_{OUTWASH} = 118 pcf$

Concrete : Stem Wall - 176 C.Y.
Deck - 231. C.Y. (Span 1 + Span 2)

Steel : Span 1 58.4 TONS
Span 2 58 TONS

$$W_1 = (231 \text{ c.y.})\left(27 \frac{ft^3}{c.y.}\right)(.150 \text{ Kcf}) + (58.4\,T + 58\,T)\left(\frac{2 \text{ Kip}}{1\,T}\right)$$

$W_1 = 1168$ Kips

$$W_2 = (176 \text{ c.y.})\left(27 \frac{ft^3}{c.y.}\right)(.150 \text{ Kcf})$$

$W_2 = 713$ Kips

$W_T = 1880$ Kips

South Footing : $\frac{1880 \text{ Kips}}{2} = 940$ Kips

North Footing : $\frac{1880 \text{ Kips}}{2} = 940$ Kips

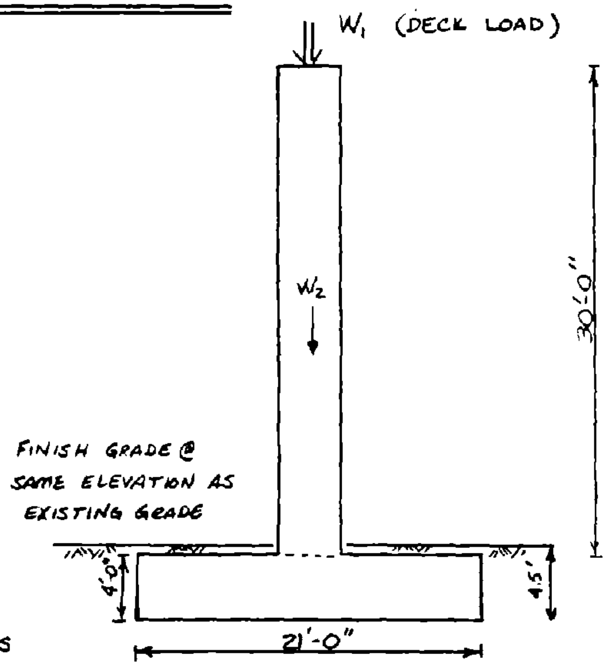

Figure 9
Scale : 1" = 10'

Footing Dimensions

South : Length (L) = 30.3 ft
Width (B) = 21.0 ft

North : Length (L) = 21.0 ft
Width (B) = 21.0 ft

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South Footing : $\dfrac{940 \text{ Kips}}{(30.3\text{ ft})(21.0\text{ ft})} = 1.5 \text{ Ksf}$

North Footing : $\dfrac{940 \text{ Kips}}{(21.0\text{ ft})(21.0\text{ ft})} = 2.1 \text{ Ksf}$

NOTE : The weight of the footing has been neglected in order to compare predicted settlements with measured stem wall settlements

STRESS INTERACTION :

The effects of stress interaction on settlement calculations will be addressed in Hough and Schmortmann

Assume stress overlap begins at the depth coinciding with the depth of the invert of the 48" ϕ sewer.

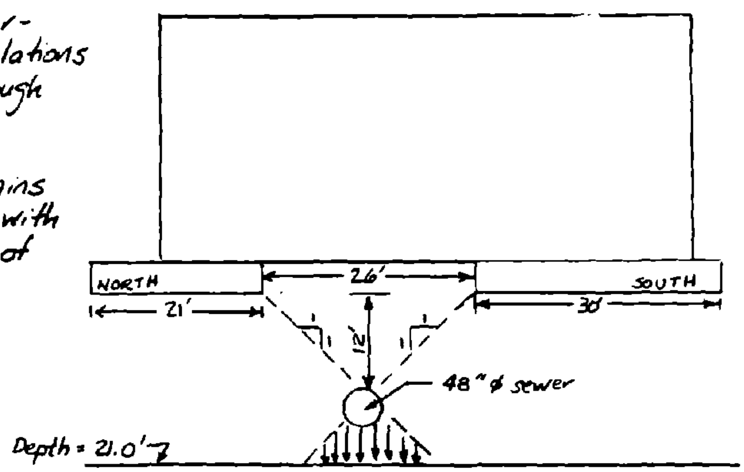

Figure 10

Scale 1" = 20'

PRELOADED OUTWASH DEPOSITS

The natural outwash sands at this location have been preloaded by the previous bridge as well 25^{+} ft of material that was cut to form the railroad yard years ago.

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I BURLAND METHOD

Refer to sheet C4 of the west abutment calculations for the discussion of Burland's equation.

Procedure

1) Determine the depth of influence below the base of the footing

- Plot N-values vs. depth to a depth of 4B below the base of the footing

- Considering boring H-3 and BH-2 the general trend is decreasing with depth.

∴ The zone of influence

$z_I = 2B$

$z_I = 42 ft.$

From 0 - B; $\bar{N} = 19$

B-2B; $\bar{N} = 20$

USE $\bar{N} = 19$

2) Determine compressibility factor (I_c)

$$I_c = \frac{1.71}{\bar{N}^{1.4}}$$

$$I_c = \frac{1.71}{(19)^{1.4}}$$

$$\boxed{I_c = 2.77 \times 10^{-2}}$$

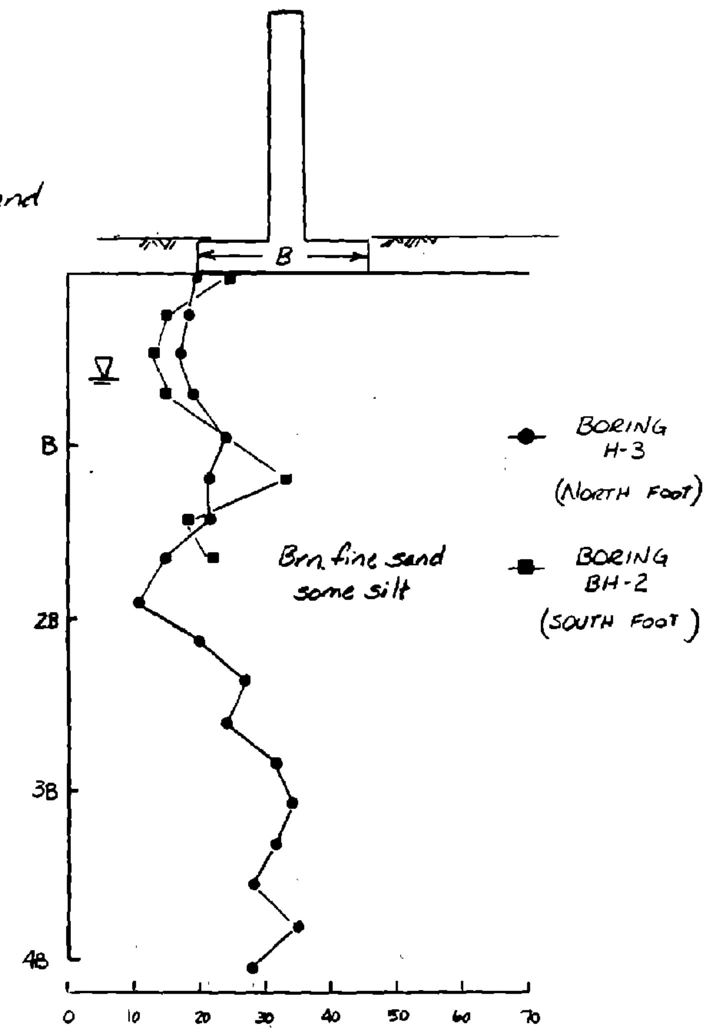

Figure 11
Scale 1" = 20'

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3) Determine shape correction factor (f_s)

$$f_s = \left[\frac{1.25\ L/B}{L/B + 0.25} \right]^2$$

NORTH FOOTING

$$f_s = \left[\frac{1.25 \left(\frac{21.0'}{21.0'} \right)}{21/21' + 0.25} \right]^2$$

$$\boxed{f_s = 1.0}$$

SOUTH FOOTING

$$f_s = \left[\frac{1.25 \left(\frac{30.3'}{21.0'} \right)}{30.3/21' + 0.25} \right]^2$$

$$\boxed{f_s = 1.1}$$

4) Determine sand layer thickness correction factor (f_ℓ)

$H_s >> z_I$, as was the case @ the West Abutment

$\therefore \boxed{f_\ell = 1.0}$

5) Determine time factor (f_t)

elapsed time $t \approx 2.5\ yr < 3.0\ yr$

$\therefore \boxed{f_t = 1.0}$

6) Calculate Settlement - over consolidated sand

$q' < \sigma_{v0}'$

$$\rho = f_s \cdot f_\ell \cdot f_t \left[q' \times B^{0.7} \times \frac{I_c}{3} \right]$$

NORTH FOOTING:

$$\rho = (1.0)(1.0)(1.0)\left[(105\ kN/m^2)(6.5m)^{0.7}\ 9.2\times10^{-3} \right]$$

$\rho = 3.6mm$; $\boxed{\rho = 0.14\ inch}$

SOUTH FOOTING

$$\rho = (1.0)(1.0)(1.1)\left[(75\ kN/m^2)(6.5m)^{0.7}\ 9.2\times10^{-3} \right]$$

$\rho = 2.8mm$; $\boxed{\rho = 0.11\ inch}$

II - D'APPOLONIA METHOD

Refer to sheet C8 of West Abutment calculation for detailed discussion of D'Appolonia equation.

Refer to sheet C27, figure 11 for subsurface information and data.

Procedure

1) Determine embedment influence factor (μ_0)

compute D/B and obtain μ_0 from Figure 6 of Christian & Carrier (3)

$D = 5.0$ ft $D/B = 0.24$; $\boxed{\mu_0 = 0.95}$
$B = 21.0$ ft

2) Determine compressible strata influence factor (μ_1)

Compute H/B and L/B and obtain μ_1 from Figure 6 of Christian & Carrier (3)

	NORTH	SOUTH
$H = 150$ ft	$H/B = 7.1$	$H/B = 7.1$
$B = 21.0$ ft	$L/B = 1.0$	$L/B = 1.4$
$L = 21.0$ ft (N)	$\boxed{\mu_1 = 0.65}$	$\boxed{\mu_1 = 0.7}$
$L = 30.4$ ft (S)		

3) Determine Modulus of Compressibility (tsf)

@ Pier 1; $\bar{N} = 18$ to a depth corresponding to "B" below the base of the footing

from Fig 44 - D'Appolonia (2) for preloaded sand

$\boxed{M = 625 \text{ tsf}}$ (1250 Ksf)

4) Calculate Settlement

NORTH: $s = \dfrac{(0.95)(0.65)(2.1 \text{ Ksf})(21.0 \text{ ft})}{1250 \text{ Ksf}}$; $s = 0.02$ ft
$\boxed{s = 0.26 \text{ inch}}$

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SOUTH :

$$s = \frac{(0.95)(0.7)(1.5\ \text{ksf})(21.0\ \text{ft})}{1250\ \text{ksf}}$$

$s = 0.017$ ft

$\boxed{S = 0.20\ \text{inch}}$

III PECK & BAZARAA

Refer to sheet C14 of the West Abutment calculation for detailed discussion of Peck & Bazaraa equation

Refer to sheet C27 figure 11 for subsurface information and data.

Procedure

1) Calculate the groundwater correction factor (if applicable)

Measured groundwater = El. 17.0±
0.5 (B) = El. 19.5

∴ Existing groundwater is below depth = 0.5 B
∴ $\boxed{K = 1.0}$

2) Calculate the embedment correction factor (if applicable)

@ Pier 1, base of the footing is approximately 5.0 ft below the previous existing grade.

$D = 5.0$ ft ; outwash sand $\gamma = .113$ kcf

$$C_d = 1.0 - 0.4\left[\frac{\gamma D}{q}\right]^{1/2}$$

NORTH FOOTING : $q = 2.1$ ksf

SOUTH FOOTING : $q = 1.5$ ksf

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NORTH : $C_d = 1.0 - 0.4\left[\frac{(.118\ kcf)(5.0ft)}{2.1\ ksf}\right]^{1/2}$

$\boxed{C_d = 0.79}$

SOUTH : $C_d = 1.0 - 0.4\left[\frac{(.118kcf)(5.0ft)}{1.5\ ksf}\right]^{1/2}$

$\boxed{C_d = 0.75}$

3) N-value Correction - depth of influence = B

@ depth = B $\quad p_o = (18\ ft)(.118kcf) - (8\ ft)(.118\ kcf - .062kcf)$

$p_o = 1.7\ ksf$

$p_o \geq 1.5\ ksf$; $\bar{N} = 18$ to depth = B

$\therefore N_c = \frac{4N}{3.25 + 0.5p_o} = \frac{4(18)}{3.25 + 0.5(1.7ksf)}$

$\boxed{N_c = 18}$

4) Calculate Settlement

$$S = K C_d \frac{2q}{N_c}\left(\frac{2B}{B+1}\right)^2$$

NORTH FOOTING
($q = 1.05tsf$)

$$S = (1.0)(0.79)\frac{2(1.05tsf)}{18}\left(\frac{2(21')}{21'+1}\right)^2$$

$S = 0.33$ inch

Preloaded Sand Condition - divide settlement prediction in half

$\boxed{S = 0.16\ inch.}$

SOUTH FOOTING
($q = 0.75\ tsf$)

$$S = (1.0)(0.75)\frac{2(0.75\ tsf)}{18}\left(\frac{2(21')}{21'+1}\right)^2$$

$S = 0.23$ inch

Preloaded Sand Condition: $\boxed{S = 0.12\ inch}$

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IV HOUGH METHOD

Refer to sheet C11 of the West Abutment calculations for a detailed discussion of Hough equation.

Refer to sheet C27, figure 11 for subsurface information and data.

Procedure

Calculate the following parameters for the table below:

a.) $P_{applied}$ NORTH : P_a = 940 Kips
SOUTH : P_a = 940 Kips

b) use Figure 12 (Hough) - ratio N'/N
use Figure 13 (Hough) - C'

assume : "clean uniform fine sand" for the outwash material - approximate C' between "clean uniform medium sand" and "clean well graded fine to coarse sand"

NORTH FOOTING

TABLE 3

	(1) MID DEPTH (ft)	(2) P_o (ksf)	(3)* ΔP (ksf)	(4) $\frac{\Delta P}{P_o}$	(5) $\log\left(\frac{\Delta P}{P_o}+1\right)$	(6) N'/N	(7) N	(8) N'	(9) C'	(10) H (ft)	(11) ΔH (in)	
	6.5	0.78	1.24	1.58	0.41	1.55	20	31	111	13	0.58	
@ 13.0 ∇	15.0	1.65	0.72	0.44	0.16	0.99	17	17	64	4	0.12	
	22.0	2.00	0.93	0.46	0.16	0.94	24	22	84	10		0.22
		2.00	0.51	0.25	0.09	0.94	24	22	84	10	0.12	
	32.0	2.60	0.61	0.23	0.09	0.88	23	20	78	10		0.14
			0.33	0.12	0.05	0.88	23	20	78	10	0.07	
	42	3.16	0.44	0.14	0.06	0.83	20	17	64	12		0.13

* All underlined numbers take stress interaction into account

Σ = 0.89" 1.19

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NORTH FOOTING (CONTINUED)

Considering stress interaction between the two footings the calculated settlement is 1.19". At Pier 1, the subsurface outwash deposits have been preloaded.

$$\therefore \quad \Delta H = (0.5)(1.20\text{ inch}) = \boxed{0.60\text{ inch}}$$

Neglecting stress interaction the calculated settlement is 0.90". Considering pre-loading conditions:

$$\Delta H = 0.5(0.9\text{ inch}) = \boxed{0.45\text{ inch}}$$

SOUTH FOOTING

TABLE 4

(1) MID DEPTH (ft)	(2) P_o (ksf)	(3)* ΔP (ksf)	(4) $\frac{\Delta P}{P_o}$	(5) $\log\left(\frac{\Delta P}{P_o}+1\right)$	(6) N'/N	(7) N	(8) N'	(9) C'	(10) H (ft)	(11) ΔH (in)
6.5	0.78	0.93	1.19	0.34	1.55	20	31	111	13	0.48
15.0	1.65	0.58	0.35	0.13	0.99	17	17	64	4	0.10
22.0	2.00	0.93	0.46	0.16	0.94	24	22	84	10	0.22
	2.00	0.42	0.21	0.08	0.94	24	22	84	10	0.11
32.0	2.60	0.61	0.23	0.09	0.88	23	20	78	10	0.13
		0.28	0.11	0.04	0.88	23	20	78	10	0.06
42.0	3.16	0.44	0.14	0.06	0.83	20	17	64	12	0.13

$\Sigma = 0.75"$ 1.06

@ Pier 1, subsurface outwash deposits have been preloaded

$\Delta H = (0.5)(1.06") = \boxed{0.53\text{ inch}}$ (considering preload)

$\Delta H = (0.5)(0.75") = \boxed{0.38\text{ inch}}$ (neglecting preload)

V SCHMERTMANN METHOD

Refer to sheet C17 of the West Abutment calculations for a detailed discussion of Schmertmann equation

Refer to sheet C39 figure 13 for CPT subsurface information This figure illustrates both the measured CPT and the approximated CPT data for the pier

Procedure

1.) Construct the strain influence triangle

a.) Consider the spacing between the two footings and the resulting stress interaction and equivalent dimensions.

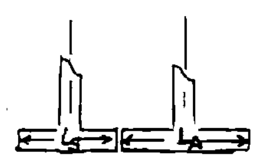
The author suggests that applied loads from adjacent footings act independent of each other if the intersection of 45° lines from the edges of each abutment occurs below the depth equal to the smallest footing width.

However, if stress interaction is evident, consider increasing the minimum width "B" to account for the added stresses.

The increase is dimension is as follows:

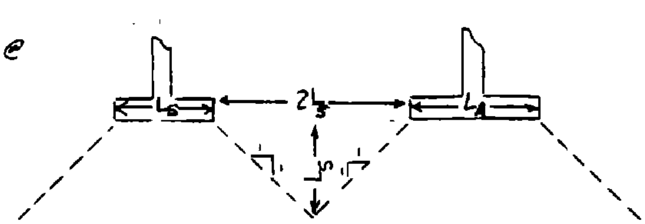
SAY: $L_S < L_A$; settlement calculation for footing B_S
S = clear space between the adjacent footings

case 1) $s = 0$
$L_{equiv} = L_S + L_A$

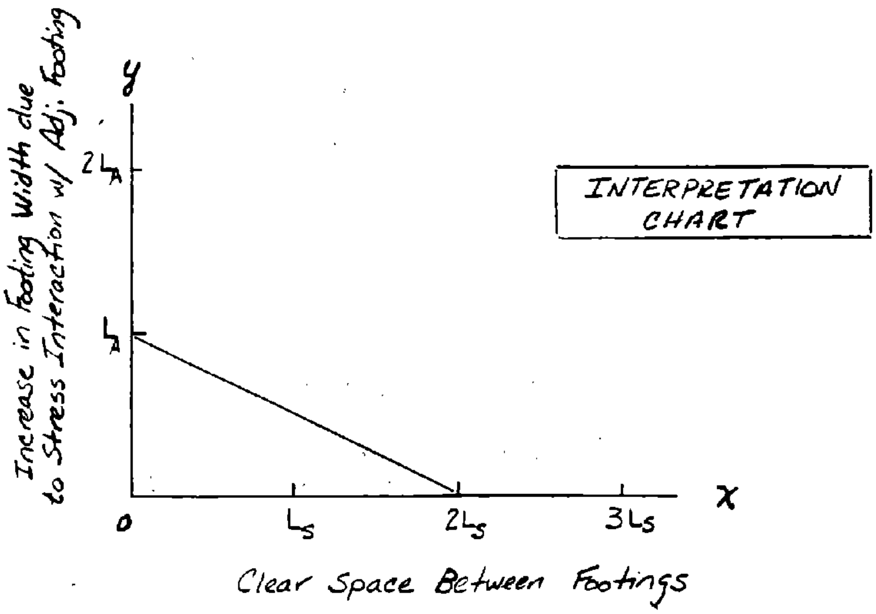


case 2) $s \approx 2L_S$
stress overlap @ depth = L_S

$L_{equiv} = L_S$



for intermediate cases use the chart below:



where;

L_S = dimension of the smallest of two or more footings whose stresses interact (assume 45° lines from edges of abutment) above a depth of B_S. B_S is measured as the dimension of the smallest footing along a line between the two footings.

L_A = dimension of the footing adjacent to the footing under which settlement is to be calculated in the direction of a line between the two footings.

$L_{equiv} = L_\alpha + y$

α - the width of the footing under consideration for settlement calculation

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@ Pier 1, calculate the settlement of the north footing.

$L_S = 21.0$ ft
$L_A = 30.3$ ft
$x = 26.0$ ft

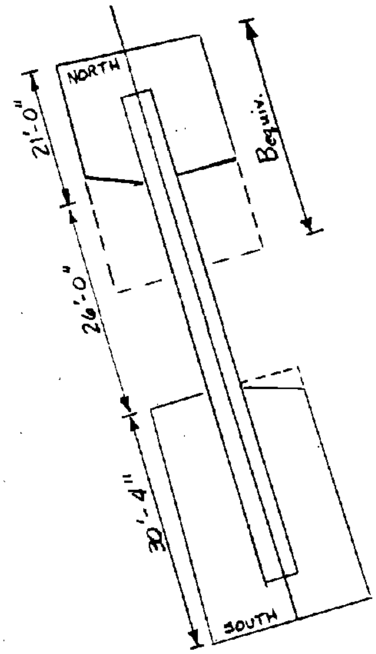

Figure 12
(NTS)

from Interpretation Chart

$$x = \frac{26.0}{21.0} L_S$$

$$x = 1.24\ L_A$$

$$\therefore\ y = 0.4\ L_A$$

$$L_{equiv} = L_S + 0.4 L_A$$
$$L_{equiv} = 21.0\text{ft} + 0.4(30.3\text{ft})$$
$$L_{equiv} = 33.1\text{ ft}$$

$$\therefore\ \frac{L}{B} = \frac{33.1\text{ ft}}{21.0\text{ ft}} = 1.6$$

1a.) For pt. A of the strain influence triangle

$$A = 0.1 + \frac{\frac{L/B - 1}{9}}{10}$$

$$A = 0.1 + \frac{\frac{1.6-1}{9}}{10}$$

$$\boxed{A = 0.11}$$

1b.) Peak influence factor and corresponding depth

$$I_{zp} = 0.5 + 0.1\left(\frac{\Delta P}{\bar{\sigma}_{vp}}\right)^{1/2}$$

$$\Delta P = P - P_0$$
$$\Delta P = 2.1\text{ ksf} - (4.5\text{ft})(.118\text{ kcf})$$
$$\Delta P = 1.6\text{ ksf}\quad (0.8\text{ kg/cm}^2)$$

Depth to I_{zp} ; interpolate between $B/2$ and B

$$Depth = \left(0.5 + \frac{\frac{L/B-1}{9}}{2}\right) B$$

$$Depth = \left(0.5 + \frac{0.07}{2}\right) B$$

$$Depth = 0.54\,B$$

$$Depth = 0.54\,(21.0\,ft)$$

$\underline{Depth = 11.3\ ft}$ below the base of the footing

∴ At the toe of the footing .... 4.5 ft of fill placed
Therefore, 15.8 ft of outwash sand above the depth of I_{zp}. Groundwater level @ approximately El. 17.0±
(i.e. 13 ft. below base of footing)

$$\sigma'_{vp} = (15.8\,ft)(.118\,kcf)$$

$$\sigma'_{vp} = 1.9\,ksf$$

$$I_{z_p} = 0.5 + 0.1\left(\frac{1.6\,ksf}{1.9\,ksf}\right)^{1/2}$$

$$\boxed{I_{zp} = 0.59}$$

1c.) Relative depth below base of footing (Point C)

$$C = 2B + 2\left[\frac{L/B-1}{9}\right]B$$

$$C = 2.13\,B$$

$$\boxed{C = 44.8\,ft}$$

2.) Determine Equivalent Young's Modulus (E_s)

$$E_s = 2.5q_c + \left[\frac{L/B-1}{9}\right]q_c$$

$$E_s = 2.5q_c + 0.07\,q_c$$

$$\boxed{E_s = 2.6q_c}$$

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3) Calculate the Correction Factors C_1 C_2

$$C_1 = 1.0 - 0.5\left(\frac{P_0}{\Delta P}\right)$$

$$C_1 = 1.0 - 0.5\left(\frac{0.5\ ksf}{1.6\ ksf}\right)$$

$$\boxed{C_1 = 0.84}$$

$$C_2 = 1.0 + 0.2\log\left(\frac{t}{0.1}\right)$$

$t =$ 897 days, (2.5 yrs)

$$C_2 = 1.0 + 0.2\log\left(\frac{2.5}{0.1}\right)$$

$$\boxed{C_2 = 1.23}$$

4) Plot measured and/or approx. q_c (kg/cm^2)

Plot strain influence triangle

Use approximated q_c values

0-20 ft q_c = 90 kg/cm^2
20ft-34ft q_c = 101 kg/cm^2
34ft-44ft q_c = 52 kg/cm^2

DEPTH	Measured q_c (kg/cm^2)	Approximate q_c (kg/cm^2)
0-12	90	84
12-18	85	84
18-21	107	84 / 101
21-25	78	101
25-34	-	101
34-44	-	52

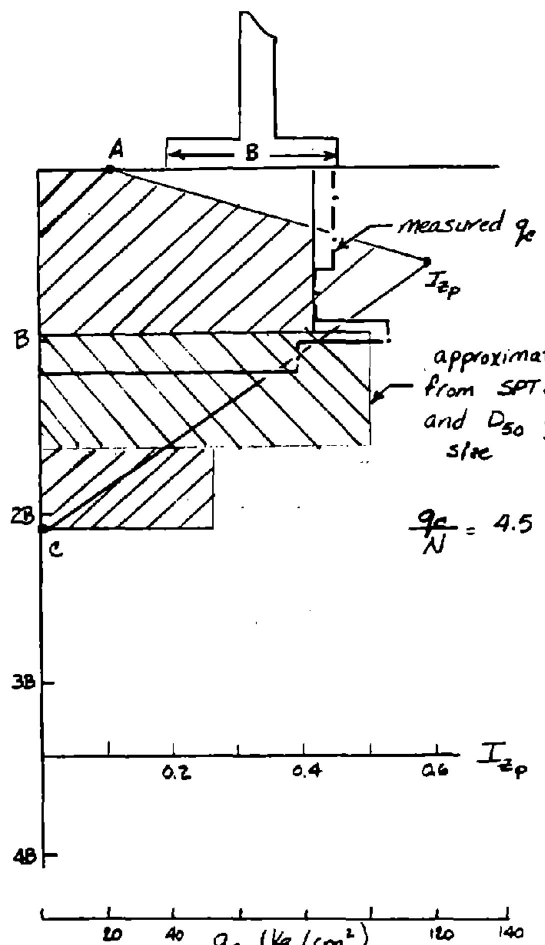

Figure 13
Scale 1" = 40.0'

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5) Construct table 5 for settlement calculation

NORTH FOOTING

TABLE 5

(1) Layer	(2) Δz (ft)	(cm)	(3) $\bar{q}_c$ [(a)] Kg/cm²	(4) E_s [(b)] Kg/cm²	(5) $z_{¢}$ (ft)	(6) I_z [(c)]	(7) $\left(\frac{I_z}{E_s}\right)\Delta z$ cm/ Kg/cm²
1a.	11.3	344	84	210	5.6	0.34	0.56
1b.	8.7	265	84	210	15.7	0.52	0.66
2	14.0	427	101	263	27.0	0.31	0.50
3	10.0	305	52	135	39.0	0.09	0.20
Total							1.92

a $\bar{q}_c$; see figure 13, approximated values
b E_s = $2.6 q_c$
c. I_z ; see figure 13

6) Calculate Settlement

$$S = (0.84)(1.28)(0.8\ \text{kg/cm}^2)\left(1.92\ \text{cm}/\text{kg/cm}^2\right)$$

$$S = 1.53\ \text{cm}$$

$$S = 0.60\ \text{inch}$$

Preloaded conditions

$\therefore$ $\boxed{S = 0.30\ \text{inch}}$

SOUTH FOOTING

from Interpretation Chart

$x = \frac{26.0}{24.0} L_s$

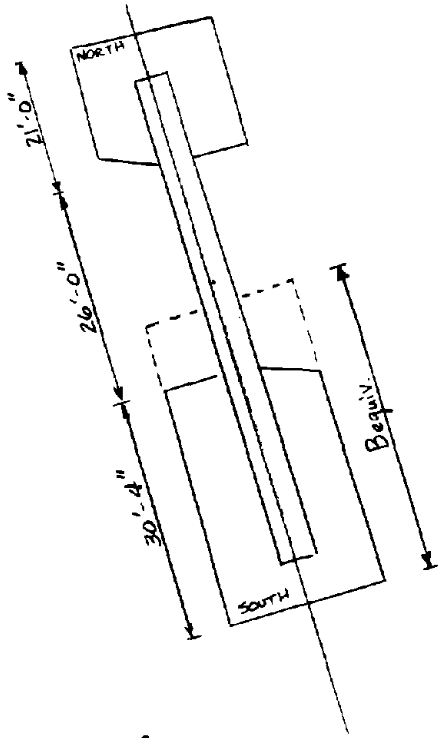
$x = 1.24\ L_s$

$\therefore\ y = 0.4\ L_s$
$y = 0.4(21.0ft)$
$y = 8.4\ ft$

$L_{equiv} = L_a + y$

$L_{equiv} = 30.3ft + 8.4ft$
$L_{equiv} = 38.7ft$

$\frac{L}{B} = \frac{38.7ft}{21.0ft} = 1.8$



1) Determine the geometry and relative limits of the strain influence diagram.

1a.) For pt. A of the strain influence triangle

$$A = 0.1 + \frac{\frac{L/B - 1}{9}}{10}$$

$$A = 0.1 + \frac{\frac{1.8 - 1}{9}}{10}$$

$$\boxed{A = 0.11}$$

1b.) Peak influence factor and corresponding depth

$$I_{zp} = 0.5 + 0.1\left(\frac{\Delta P}{\bar{\sigma}_{vp}}\right)^{1/2}$$

$\Delta P = P - P_o$
$\Delta P = 1.5\ ksf - (4.5ft)(.118\ kcf)$
$\Delta P = 0.97\ ksf\ \ (0.48\ kg/cm^2)$

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Depth to I_{zp} ; interpolate between $B/2$ and B

$$\text{Depth} = \left(0.5 + \frac{\frac{L/B-1}{9}}{2}\right) B$$

$$\text{Depth} = \left(0.5 + \frac{0.09}{2}\right) B$$

$$\text{Depth} = 0.54\,B$$

$$\text{Depth} = 0.54\,(21.0\text{ ft})$$

$$\underline{\underline{\text{Depth} = 11.3\text{ ft}}}$$ below the base of the footing

∴ At the toe of the footing .... 4.5 ft. of fill placed.
As was the case with the north footing -

$$\bar{\sigma}_{vp} = 1.9\text{ ksf}$$

$$I_{zp} = 0.5 + 0.1\left(\frac{\Delta P}{\bar{\sigma}_{vp}}\right)^{1/2}$$

$$I_{zp} = 0.5 + 0.1\left(\frac{0.97\text{ ksf}}{1.9\text{ ksf}}\right)^{1/2}$$

$$\boxed{I_{zp} = 0.57}$$

1c.) Relative depth below base of footing (Point C)

$$C = 2B + 2\left[\frac{L/B-1}{9}\right] B$$

$$C = 2.18\,B$$

$$\boxed{C = 45.8\text{ ft}}$$

2.) Determine Equivalent Young's Modulus (E_s)

$$E_s = 2.5q_c + \left[\frac{L/B-1}{9}\right] q_c$$

$$E_s = 2.5q_c + 0.09q_c$$

$$\boxed{E_s = 2.6q_c}$$

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3) Calculate the Correction Factors C_1, C_2

$$C_1 = 1.0 - 0.5\left(\frac{P_o}{\Delta P}\right)$$

$$C_1 = 1.0 - 0.5\left(\frac{0.53\,Ksf}{0.97\,Ksf}\right)$$

$$\boxed{C_1 = 0.73}$$

$$C_2 = 1.0 + 0.2\log\left(\frac{t}{0.1}\right)$$

$t = 2.5$ yrs

$$C_2 = 1.0 + 0.2\log\left(\frac{2.5}{0.1}\right)$$

$$\boxed{C_2 = 1.28}$$

4) Plot measured cone data and approximated cone data

approximate q_c values based on mean grain size D_{50} and SPT data

$$\frac{q_c}{N} = 4.5$$

DEPTH	Measured q_c (Kg/cm²)	Approx q_c (Kg/cm²)
0-5'	64	64
5-10'	135	64
10-17	89	64 / 114
17-26	79	114
26-46	—	69

Plot strain influence triangle.

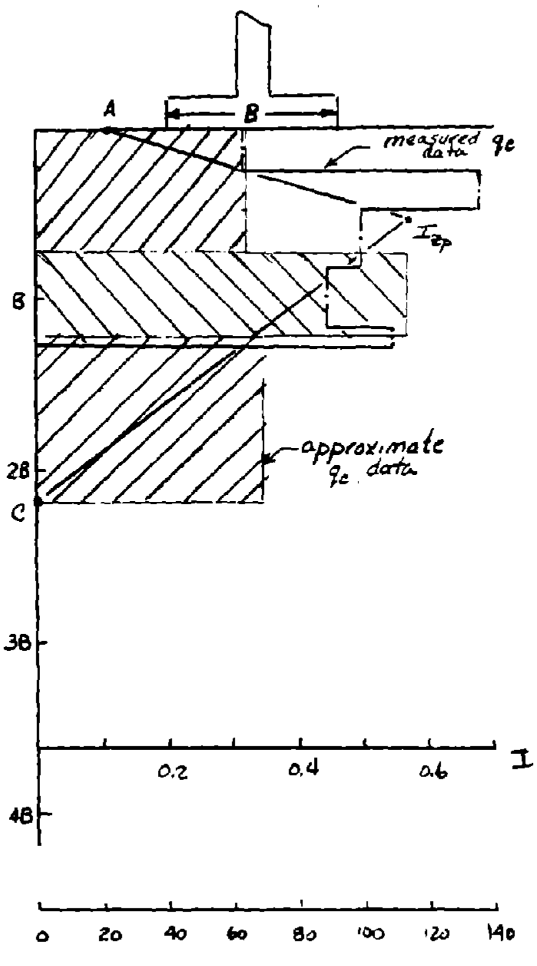

Figure 14
Scale 1" = 20.0'

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5) Construct table 6 for settlement calculation

SOUTH
FOOTING

TABLE 6

(1)	(2)		(3)	(4)	(5)	(6)	(7)
LAYER	Δz (ft)	Δz (cm)	$\bar{q}_c$ [a] (kg/cm²)	E_s [b] (kg/cm²)	z_c (ft)	I_z [c]	$\left(\frac{I_z}{E_s}\right)\Delta z$ cm/kg/cm²
1a	11.3	344	64	166	5.6	0.32	0.66
1b	3.7	113	64	166	13.1	0.54	0.37
2	12.5	381	135	351	21.2	0.42	0.46
3	18.3	558	69	179	37.5	0.14	0.43
Total							1.92

a $\bar{q}_c$; see figure 14 approximated values
b $E_s = 2.6\, q_c$
c I_z ; see figure 14

6) Calculate Settlement :

$$S = (0.73)(1.28)(0.5\ kg/cm^2)(1.92\ cm/kg/cm^2)$$

$S = 0.90$ cm
$S = 0.35$ in

Preloaded conditions

$\therefore \boxed{S = 0.18\ in.}$

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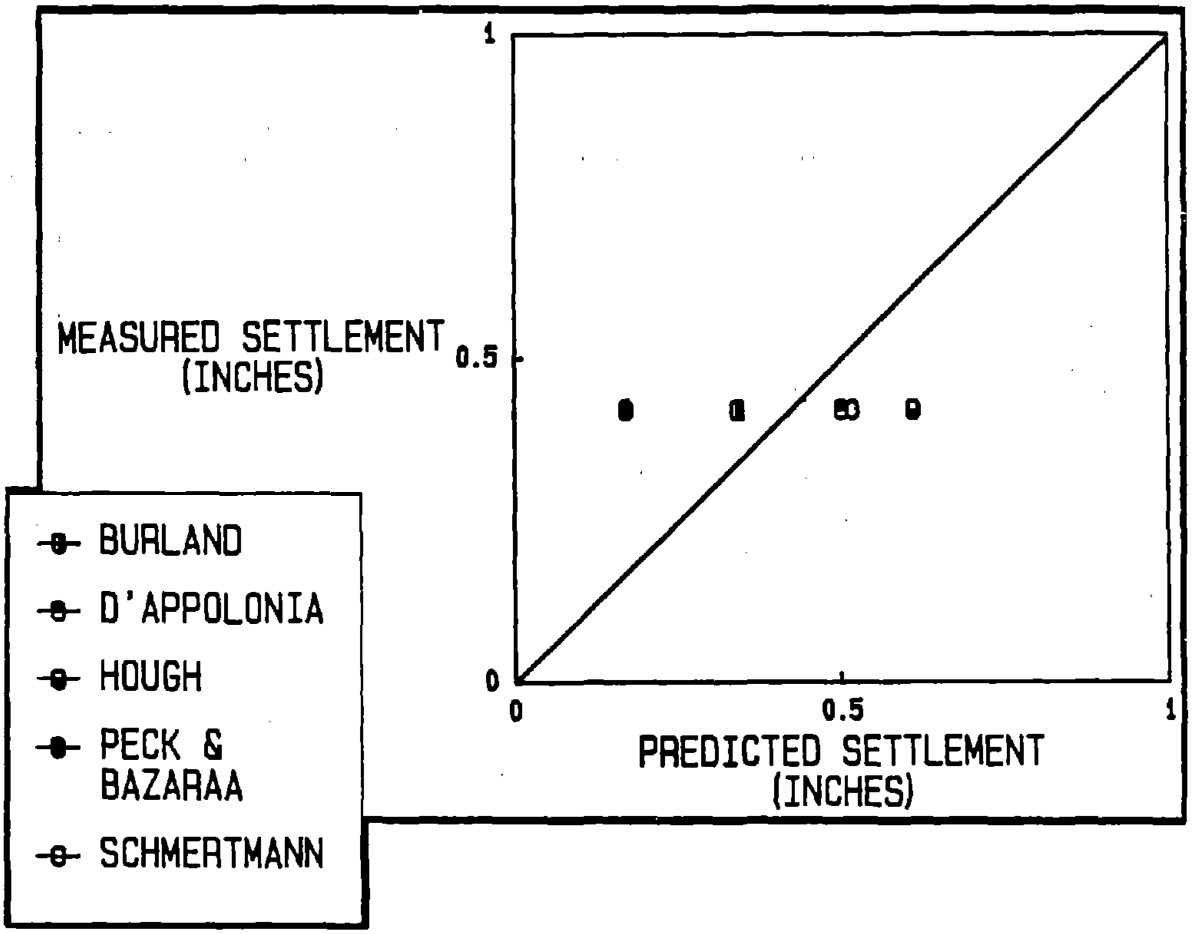
WEST ABUTMENT

(Predict vs. Measured Settlement)



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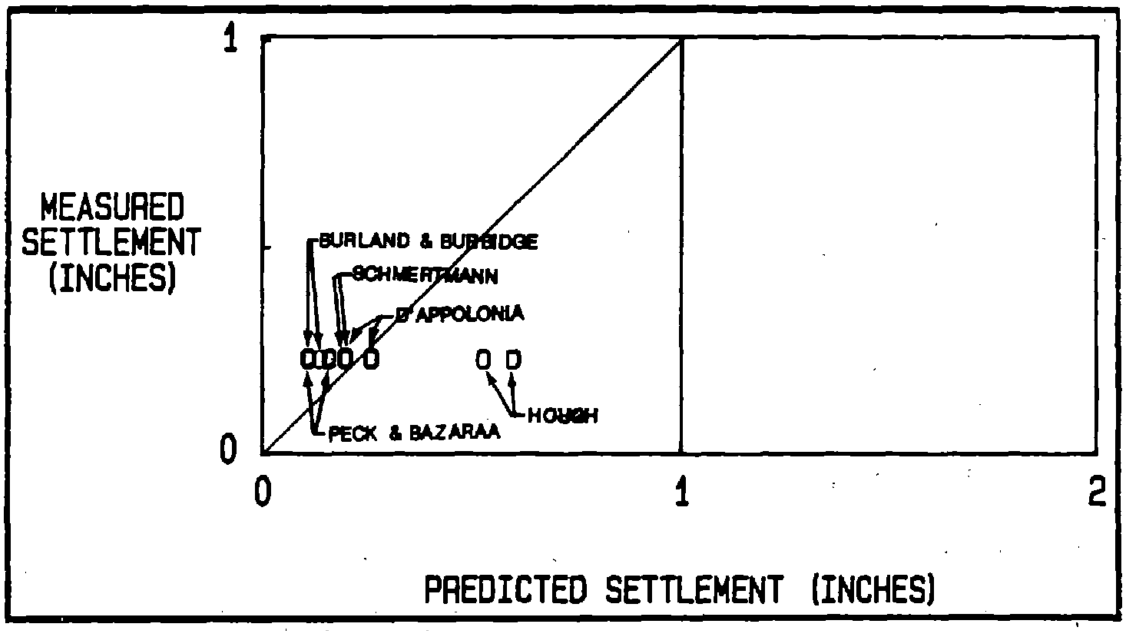
PIER 1

(Predicted vs Measured Settlement)



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