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SYNTHESIS ON FAULTED PAVEMENTS AT BRIDGE ABUTMENTS

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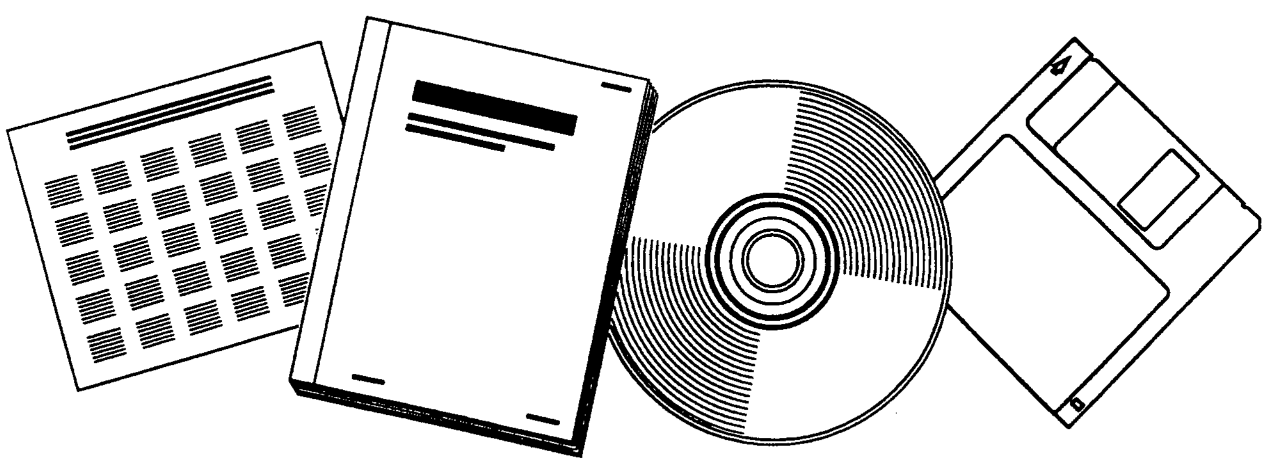

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Synthesis on
Faulted Pavements at Bridge Abutments

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May 1997

Prepared in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

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This synthesis includes a review of the causes of settlement-related problems in bridges, a review of methods for mitigation, a compilation and analysis of data on bridge settlements, an assessment of methods for the prediction of settlements, a proposal for a method of estimating differential settlements based on the variability in total settlement, an examination of spatial correlation in settlements, and a review of plastic rotation capacity in steel beams and its use in design of bridge beams to tolerate relatively large settlements.

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Section 1 SYNTHESIS ON FAULTED PAVEMENTS AT BRIDGE ABUTMENTS

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ABSTRACT

This is a synthesis of the literature on the problem of faults in roadway pavements at bridge abutments. Faults are differences in elevation of approach pavements and bridge decks caused by unequal settlement of embankments and abutments. There has been considerable interest in this problem in the past, and previous studies have recommended or demonstrated a number of methods for mitigation of pavement faults. Past efforts have focused on particular construction methods or materials to reduce total settlements and thereby reduce differential settlements. This synthesis attempts to collect and to evaluate the computational tools needed for an engineering practice to evaluate the potential for pavement faults, select appropriate methods for mitigation, and assess the expected performance of embankments and abutments under the action of a mitigation. An engineering practice for pavement faults requires adequate analysis of expected differential settlements, quantitative data on the improvements achieved by methods of mitigation, and realistic limits on the tolerance of structures for settlement. This synthesis attempts to assemble the analyses, the mitigations and limits that are needed. By this process, the synthesis is able to discover what is lacking, and what must be developed in continuing research.

This synthesis includes a review of the causes of settlement-related problems in bridges, a review of methods for mitigation, a compilation and analysis of data on bridge settlements, an assessment of methods for the prediction of settlements, a proposal for a method of estimating differential settlements based on the variability in total settlement, an examination of spatial correlation in settlements, and a review of plastic rotation capacity in steel beams and its use in design of bridge beams to tolerate relatively large settlements.

GOALS OF THE RESEARCH

The Colorado Transportation Institute and the Colorado Department of Transportation are developing the means to reduce faults in pavements at bridge abutments. Pavement faults are a hazard for DOT maintenance vehicles, especially snowplows, and are a cause of public dissatisfaction. CTI has undertaken this research because previous efforts, though numerous, have not succeeded in eliminating pavement faults.

The CTI effort will be executed in three phases. The first is a synthesis of previous work on pavement faults and an assessment of the research and development needs in the area. The second phase will be the execution of the research and development programs identified in the synthesis. The third phase will be a field demonstration of methods to mitigate pavement faults. This synthesis is the completion of the first phase of the CTI effort.

GOALS OF THE SYNTHESIS

The synthesis collects existing knowledge and identifies needs for new or expanded knowledge on causes of faults and existing technologies for the mitigation of faults. Sections in the synthesis include:

OCCURRENCE OF PAVEMENTS FAULTS. REPORTED CAUSES.

The first, basic step is a review of the reported causes of pavement faults. Mitigation of any sort from an improvement of fill material, to a change in the design basis for bridge foundations, can be chosen only in response to an identification of the causes of faults. If there are one or a few predominant causes, then selection of specific methods for mitigation is possible. If the causes are diverse and differ from project to project, then the primary need is for methods of analysis and design.

MITIGATION OF PAVEMENT FAULTS.

The synthesis reviews previous work on pavement faults. Mostly, this is work on pavement distress in approaches, and is better described as studies of problems associated with settlements of embankments and bridge substructures. A significant part of the literature on settlement problems addresses the development and demonstration of methods to reduce settlements of foundations and of embankments. Many methods available are to reduce settlements of embankments. These methods are often named as promising means for the reduction of pavement faults. Soil improvement can indeed be a method for the reduction of faults, but only if faults are caused by settlements of embankments that are greater than settlements of abutments. Thirteen distinct methods of soil improvement are collected from more than 40 literature sources.

An engineering design approach to the mitigation of pavement faults requires an ability to make quantitative predictions of the settlements that will occur at bridges. Moreover, there is the implication that the magnitude of settlements can be controlled, even selected, depending on the type of fill, the improvement of the fill foundation and the design of bridge foundations. Towards this end, methods for the prediction of total settlements and differential settlements are reviewed, the relation of differential settlement to variability in total settlements is examined together with spatial correlation of settlements. A method of prediction of differential settlement is proposed. In addition, tolerable settlement criteria for bridge beams are reviewed, and new criteria for compact steel beams are proposed on the basis of inelastic bending capacity.

OBSERVED TOTAL SETTLEMENTS.

Data on settlements of bridges, and embankments and embankments are reviewed. Differences in total settlement among embankments, structural foundations on deep foundation and structural foundations on shallow foundations are examined. Settlements of embankments are somewhat greater than settlements of structural foundations, but the data do not indicate that the use of shallow foundations for structural foundations will eliminate pavement faults. The database of observed settlements collected here contains data from more than 700 structural foundations and more than 100 embankments. These include studies of settlements during construction, settlements from one-time in service surveys of structures and from long term monitoring programs for settlements of structures.

PREDICTION OF TOTAL SETTLEMENTS.

The prevention of pavement faults in new projects requires an ability to accurately predict settlements of embankments and foundations. Studies that compare predicted and observed settlements are collected and examined. Predictions of settlements are conservative, and there can be large differences between prediction and observation for individual foundations even where mean values of predicted and observed settlements agree well. More than 1500 data points are collected from 36 studies of accuracy of settlement predictions.

DIFFERENTIAL SETTLEMENTS IN BRIDGES.

Differences in settlements cause pavement faults. Mitigations for faults must be evaluated for their potential to reduce or eliminate differential settlements. Methods for prediction of differential settlements are reviewed and a new method for prediction based on variability in total settlements is reported. Predictions of differential settlements are compared to observed differential settlements for 33 bridges. Prediction of differential settlements based on variance is accurate. Spatial correlations in settlements are also examined and it is shown that differential settlements are lesser for nearby foundations if settlements are correlated.

LIMITS ON TOLERABLE SETTLEMENTS FOR BRIDGES.

Reported limits on tolerable settlements for bridges are reviewed. Limits on tolerable settlements that are expressed as limits on angular distortion are recognized as limits on differential settlements. The new method for prediction of differential settlements is used to related limits on angular distortion to limits on total settlement. The effect of spatial correlation on limits on tolerable settlements is also examined. The tolerance of structures for differential settlements is reviewed and the important of inelastic response in structures is discussed. New methods for the estimation of tolerance for settlements of compact steel bridge beams are proposed. These estimates are based on the research in plastic rotation capacity of compact steel beams that had been performed as a part of the development of autostress design method proposed by AASHTO.

PAVEMENT FAULTS

Pavement faults will be prevented by the selection of designs that avoid or prevent differences in settlement of embankments and abutments, by the competent execution of these designs in construction, and by timely maintenance to offset deterioration that might produce faults at otherwise sound bridges.

There are two basic mechanisms in the occurrence of faults. Each mechanism leads directly to the identification of appropriate mitigations. The mechanisms are illustrated in Figure 1-1.

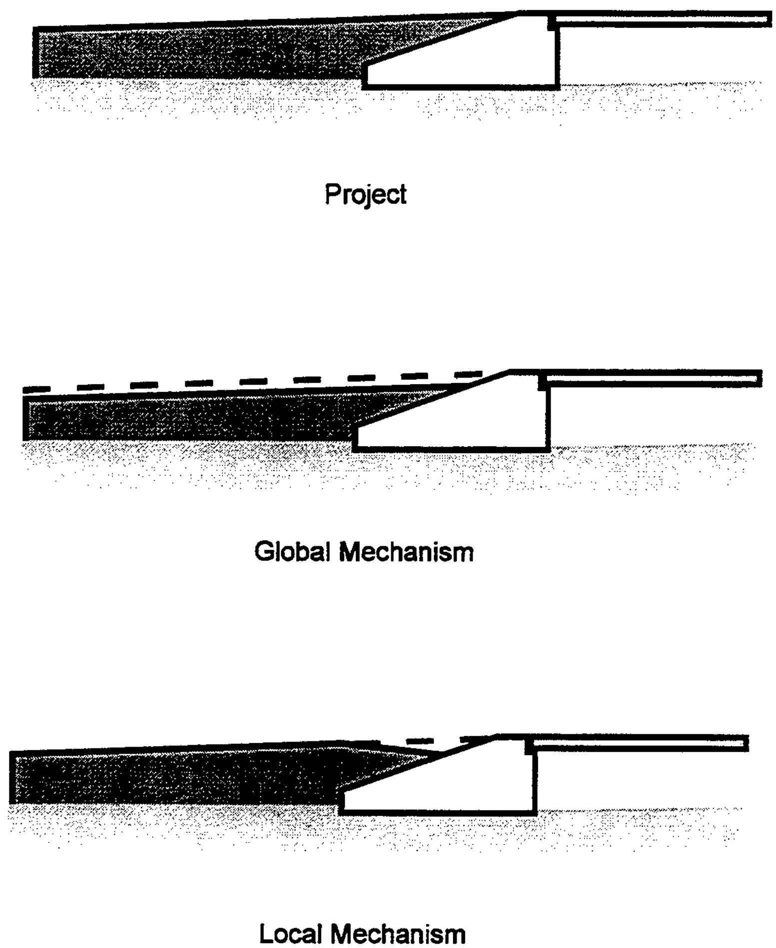

Figure 1-1 Mechanisms of Pavement Faulting

The first mechanism is global settlement of the embankment. The second is local settlement or loss of fill at the abutment. The global mechanism is mitigated by methods that reduce total settlements in embankments. Improvement of fills or fill foundation can be effective in mitigating faults due to the global mechanism. The local mechanism is associated with loss or shifting of fill due to (improper) drainage or due to movement of abutments. For the local mechanism, it is necessary to control drainage, prevent abutment movements and/or provide fills that will not erode or shift. The mechanisms, the observations of settlement that are consistent with the mechanism, and the general forms of mitigations are listed in Table 1-1. The table is an example of how mitigation methods might be selected. It is important to note that each method of mitigation may be effective for only one of the two basic mechanisms.

Mechanism	Cause / Observation	Strategies for Mitigation
Embankment settlements are greater than abutment settlements. Embankment settlements are uniform or proportional to embankment height. This is a global behavior of the embankment.	Abutments on deep foundations have near zero settlement	Use shallow foundations for abutments where possible. Footings directly on compacted fills are expected to settle an amount that is nearly equal to the settlement of the approach pavement.
		Abutments on spread footings may allow more settlement than a structure can tolerate. Settlements of embankments must be offset by mudjacking of approach pavements.
	Embankments, like any fill system, have a nonzero settlement.	Improve fills and fill foundations to achieve reduced settlements of embankments. Many methods of improvement are available including in-situ compaction, preloading, chemical stabilization, and mechanical stabilization.
Settlements of embankments and abutments are compatible overall. Pavement faults are a local problem of loss or movement of fill right at the abutment.	Drainage at abutment allows runoff into the embankment and erosion of fill	Control drainage. Eliminate pavement joints at abutments, or vigorously maintain joints and seals.
	Seasonal movement of the abutment allows small, cumulative movement of fill.	Provide chemically or mechanically stabilized fills that will not shift due to movement of the abutment. Also prevent the introduction of incompressibles in the void at the abutment backwall.
	Expansion of concrete approach pavement forces a movement of the abutment and allows a movement of fill in the embankment	Provide for the expansion of approach pavements by use of joints with compressible elements. Prevent a build-up of force at the abutment backwall.
Embankment settlements are greater than settlement of abutments, but both local and global mechanisms are observed.	The difference in settlements between embankments and abutments can be mitigated.	Provide, and maintain approach slabs.
		Provide periodic mudjacking to align approaches and bridge decks.
		Provide for the adjustment of the elevation of the end of the bridge deck, in addition to repairs or adjustment of the approach pavement.
Embankments and approaches are performing well at most bridges. Pavement faults, where they exist, are the result of unavoidable randomness in settlements.	Some bridges are affected by pavement faulting while others of nearly identical construction are not affected.	Provide for adjustment and maintenance. The occurrence of faults at individual sites is not predictable. It is however unavoidable.

Table 1-1 Mechanisms, Observations, and Mitigation of Pavement Faults

DESIGN TO PREVENT PAVEMENT FAULTS

An engineering practice for the mitigation of pavement faults introduces a new limit state that engineers will apply to the design of bridges and approaches. In addition to the existing concerns for adequate strength and for a tolerable limit on settlements, an engineering practice for faults seeks to enforce a limit on the difference in total settlements of approaches and abutments. This limit can be stated as

$$\begin{aligned} &Fault \leq Limit \\ &Fault = |S_{Abut} - S_{Emb}| \end{aligned} \qquad \textbf{Eq. 1-1}$$

where *Fault* is the difference in settlements, *Limit* is the maximum tolerable difference in settlement for a structure or a roadway, S_{Abut} is the total settlement of the abutment and S_{Emb} is the total settlement of the embankment in the vicinity of abutments. This limit is not identical to a requirement for zero or near-zero settlements of abutments and embankments, though such a requirement, if met, would prevent pavement faulting. The limit in Eq. 1-1 admits either zero settlements or non-zero settlements. It demands that for any value of settlement, embankments and abutments must settle the same amount within a *Limit* tolerance.

Equality of settlements is not easily achieved and it appears that there is no standard engineering design practice that serves this criterion. Instead, engineers today seek to keep total settlements below an upper bound. If a foundation performs 'better', that is if it exhibits less settlement than the maximum computed in the design, this is rarely considered to be a problem. Where similar settlements of separate foundations must be achieved, the usual strategy is to seek similarly in the conditions of foundations. In difficult soil conditions, an equality of settlements is achieved principally through construction-time or in-service adjustments of the structure. This includes preloading and/or the design of joints and expansion locations, and/or provision of jacking, shimming and other adjustments of structures.

The creation of designs for embankments and bridges that are not susceptible to pavement faulting must follow one of two strategies. One strategy seeks zero or near-zero settlements in both approaches and abutments. For abutments, this is probably achieved if deep foundations are used. For embankments, a criterion of zero settlements is met using some form of improvement for soils.

The second strategy allows settlements, but tries to ensure equality of settlements for abutments and embankments. Here, there are two ways to proceed. If the settlements of the embankment and the abutment are independent, then the engineer must select designs for both that yield equal settlements. A simpler, and more accessible, strategy is to seek a link in the total settlements of embankments and abutments. In this way the total settlements are not known explicitly but bounds on settlement are maintained, and settlement values are equal. For this reason, the use of footings for abutments, sometimes bearing on compacted embankment fill, is proposed as a remedy for pavement faulting.

THE SYNTHESIS ON PAVEMENT FAULTS

To prevent pavement faults, there is a need for methods of analysis of pavement faulting, for the assessment of the performance of soil improvements, for selection of foundation designs that avoid pavement faults, and for a decision process to select strategies to control faulting. The development of the practice must continually measure its success in terms of the ability to predict and to avoid pavement faults at bridges in service.

This synthesis gathers information on the occurrence pavement faults and on the methods that are recommended for the mitigation of faults, undertakes a first assessment of the analytical tools for the prediction of settlements, examines new methods for the prediction of differential settlements, and begins a reassessment of tolerable settlements of structures. In all aspects, this synthesis has sought to collect the tools needed for an engineering practice in mitigation of pavement faults.

Specific questions / concerns addressed in the synthesis include:

THE SEVERITY OF THE PROBLEM OF PAVEMENT FAULTING MUST BE IDENTIFIED.

Faulting and other settlement-related problems do not affect all bridges. A first step in the study is the collection of information on the prevalence of faults. To evaluation prevalence, this synthesis includes a review of earlier studies on settlement problems at bridges.

THE CAUSES OF PAVEMENT FAULTING MUST BE KNOWN.

Often, proposed methods for mitigation of pavement faults follow directly from assumptions about causes. The mitigations would work if assumptions about causes were correct. Indeed, the failure to achieve generally effective mitigation of pavement faults can be attributed to incorrect identification of causes. This synthesis examines data on settlements to see if mechanisms of faulting can be identified on the basis of the performance of abutments and embankments.

METHODS OF ANALYSIS OF THE POTENTIAL FOR PAVEMENT FAULTING AT INDIVIDUAL PROJECTS ARE NEEDED.

Like any engineering design, the design for mitigation of pavement faults relies on an ability to analyze needs at individual projects for the control of faulting. This is in large part an analysis of expected settlements, and the prediction of expected differential settlements. In this synthesis two tasks are undertaken. First, the accuracy of methods for prediction of total settlements is assessed. Settlement computations are a fundamental building block of an engineering practice in mitigation of faults.

Second, the prediction of differential settlements is reviewed. Beyond rule-of-thumb estimates, there are no common methods for the prediction of differential settlements among similar foundations in similar soils. This synthesis develops and demonstrates a method for the prediction of differential settlements among similar foundations. This method uses variability in total settlements to predict the occurrence and the magnitude of differential settlements. This prediction is critical to the analysis of pavement faulting.

The study of differential settlements is taken a step further into a consideration of spatial correlation in settlements. The recognition of correlation in space provides for better estimates of differential settlements and spatial correlation has an important role in the assessment of the tolerance of structures for settlements.

METHODS FOR MITIGATION OF PAVEMENT FAULTS MUST BE APPROPRIATE TO CAUSES.

The mitigation of pavement faults, and in particular the attempt to alter the practice of construction of bridges will be effective only if they address the correct causes of pavement faulting. Methods that have been recommended for the mitigation of pavement faults are reviewed, and their ability to remedy pavement faulting is discussed along with their relation to mechanisms of the causes of pavement faults.

THE PERFORMANCE OF MITIGATIONS MUST BE KNOWN.

The mitigation of pavement faults will exist in the context of a process for the selection and design of methods for mitigation. Such a practice is supported by information on the use, design and performance of mitigations. Methods that have been recommended must be supported by methods for their analysis.

MITIGATIONS MUST BE COMPATIBLE WITH STRUCTURES.

For mitigations that rely on increased settlements of abutments, bridges must be able to tolerate increased settlements. This synthesis reviews tolerable settlement criteria for bridges.

The mitigation of pavement faults may include the use of shallow foundations for bridge abutments, or the use of details that allow for the adjustment of the elevation of bridge decks to meet settlements of approach pavements. The tolerance of bridges for such adjustments limits the possible use of these mitigations. In this synthesis, the criteria for tolerable settlements of bridge beams are reviewed, and new criteria for the tolerable settlement of compact steel I-beams are proposed. These new criteria are adaptations of autostress design procedures.

OCCURRENCE OF PAVEMENT FAULTS

Information on the occurrence of pavement faults is found in studies of bridge settlements and of performance of approach pavements. Many studies deal with the development or demonstration of particular methods for the improvement of soils. Too often, individual studies of settlement problems in approaches deal with only a few bridges, and therefore it is not possible to extract a picture of the prevalence of pavement faults, or the prevalence of a particular mechanism of faulting.

Three studies however that did gather data for populations of bridges. These studies give an idea of the prevalence of pavement faults, and two of the studies have attempted to measure the magnitude of unevenness in approaches. These three studies are listed in Table 1-2.

Stewart [1985] is one of the few authors to measure approach pavement distress. He found that 59% of 820 bridge approaches in California needed repair. James [et al. 1991] used a method to calculate the average magnitude of pavement faults but did not document the prevalence of intolerable faults. Moulton [et al. 1985] and Laguros [1989] measured settlements at the bridge-approach interface, but did not explicitly measure faults. In Moulton's study, 439 out of 580 abutments supporting bridges throughout the US experienced movement. Of 758 bridges in Oklahoma, 83% of the approaches have experienced settlement [Laguros 1989].

The literature offers fewer than 1000 bridge approaches where settlements were measured. Among these, more than half exhibited faults. If these bridges are representative of the larger population of bridges, then possibly half of all bridges might be affected by settlement problems that can include pavement faults. There are several cautions in the extrapolation of these data. Few approaches have been studied, and these may all be 'problem' bridges and not representative of bridges in general. Stewart [1985] studied the need for repairs in bridge approaches. Distress in approaches is nearly always related to settlements, but these settlements may not be at the abutment so they are not all pavement faults of the sort being examined in this synthesis. Second, the study by James [1991] was based on rideability and may include pavement distress other than faults.

Source	Criteria	No. of Bridges	No. Deficient
Stewart 1985	Measured fault. Maintenance need.	820	59% of all bridge approaches had or were in need of repair 74% of asphalt approaches had or were in need of repair 43% of concrete approaches had or were in need of repair
James 1991	Ride (fault) meter	165	Bumps rated by driver from 0 (no bump) to 5 (loss of control) Average rating = 1.2, standard deviation = 0.7 One observer (83 bridges) had no ratings over 2
Laguros 1989	Settlement of approach (questionnaire)	758	83% of approaches settled (not necessarily causing a bump)

Table 1-2 Studies of Prevalence of Approach Settlement

CAUSES OF PAVEMENT FAULTS

The causes of settlements of approaches named in studies include consolidation in fills or fill foundations, erosion due to drainage through fills, movement of fills (slumping), movement of abutments, movement (growth) of approach pavements with a resulting movement or loss of fill in the embankment. Several notable studies of cause approach settlement are listed in Table

The causes of settlement listed in Table 1-3 can be grouped into the global and local mechanisms introduced earlier. Settlements of fills and fill foundations, and sliding or instability of fills are global. The performance of embankments may be adequate, even good, but larger embankment settlements compared abutment settlements result in pavement faults. Erosion, abutment movement with associated loss of fill, and pavement growth leading to loss of fill are all local mechanisms. The loss of fill in a relatively contained region of the embankment produces a depression in flexible pavements and cracking in rigid pavements. Problems resulting from poor compaction of fills can be global or local depending on the project. Most often, however poor compaction is related to difficult access at abutment backwalls. This makes it a local mechanism problem.

The finding here is that a single mechanism is not apparent in the studies of settlement-related problems in bridges and approaches. Causes identified for individual bridges include both global and local mechanisms. The local mechanism is often related to drainage and erosion of fills. But drainage may be an occurrence that is the result of some first cause such as abutment movement or pavement growth. In addition, drainage may cause severe local erosion of embankments if it is allowed to start at all. This means that a mechanism of moderate (and tolerable) global settlement in the embankment might open one or more joints in the approach pavement, and these areas are then undermined by erosion. This situation is a global (primary) mechanism in which the severe effects were produced by local erosion.

Source	Bridges	Causes of Settlement	Notes
Ardani, 1987	10	Consolidation in fill and fill foundation. Poor compaction and drainage in fill. Erosion of slope.	Causes deduced from the construction and observed drainage paths.
Meade & Allen, 1989	6	Embankment subsidence due to poor compaction, steep slopes, erosion, and secondary consolidation.	Embankments noted as having thick lifts (2′-3′) and poor compaction settled much more than those with 1′ lifts and good compaction. 80% or more of embankment settlement occurred before approach pavement was laid.
Kramer & Sajer, 1991	9	Primary and secondary consolidation and creep of embankment foundation soils. Subsidence of embankment fills due to creep, frost, erosion, and truck traffic. Failure of expansion joints and movement of abutments.	The bridge-approach interface was excavated and checked for voids and irregularities. Construction records were reviewed.
James et al. 1991	7	Longitudinal growth of pavement. Water infiltration into embankment fills.	Approach pavement cores taken to determine growth of pavement due to chemical reactions. Heavy traffic areas investigated.
Moulton, 1986	204	Movement of abutments due to poor pile foundations. Inadequate lateral resistance movement of approach embankment due to consolidation of foundation soils. Settlement of fill. Sliding (due to slope or foundation instability).	Data from questionnaire analyzed for correlations. Causes compiled from questionnaire responses.
Wahls, 1990	-	Consolidation of fill and fill foundation. Poor compaction and/or drainage erosion.	Synthesis of literature.
Hopkins & Deen, 1970	6	Primary consolidation of embankment foundation. Secondary compression and shear strain of fill foundation. Improper compaction of embankment fill. Loss of material from and around abutment. Lateral and vertical deformation of abutment.	Long-term monitoring (8 - 14 years).
Emmanuel 1978	-	Abutment movement due to growth of approach slab and settlement of fill.	From literature
McNulty 1979	1	Embankment movement due to erosion.	Fixed by replacement of fill with lightweight fill

Table 1-3 Studies of Causes of Approach Settlement

Mechanism	Named Cause
Global	Settlement / consolidation in fill. Settlement / consolidation in fill foundation. Sliding / instability of embankment.
Local	Erosion. Abutment movement (tilt). Growth of pavement.

Table 1-4 Mechanism and Causes of Pavement Faults

MITIGATION OF PAVEMENT FAULTS

Pavement faults are mitigated if differences in total settlements of abutments and approaches are reduced. Total settlements of fills and fill foundations may be reduced by soil improvement by any of several methods. Settlements of abutments may be made to match settlements more closely of embankments by use shallow foundations. Erosion may be reduced by use of highly competent fills such as thin grouts and flowable cementitious fills. Yet another type of mitigation is an attempt to bridge differences in settlements with approach slabs.

This synthesis forms a guide to the literature for a range of methods that may be useful for the reduction of pavement faults at abutments. These methods can be grouped as follows:

I. **Quality assurance / quality control during construction.**
Mitigation by QA/QC follows from the belief that pavement faults are the result of poor materials or workmanship. Mitigation measures include:

A. Adequate compaction, especially at backwalls and wingwalls of abutments.

B. Adequate inspection and control of fill materials and compaction operations.

II. **Consolidation and/or compaction of fill foundations.**
These methods improve the soils underlying the embankment without the use of mechanical or chemical inclusions. These mitigations are based on the belief that inadequate fill foundations cause pavement faults. Under this heading methods such as

A. Preloading.

B. Sand drains, wick drains.

C. Vibrocompaction, dynamic compaction.

III. **Chemical improvement of fills and/or fill foundations.**
These methods are similar in effect to methods for consolidation and compaction. Here grouts, or lime additions cement the soil together to reduce settlements. Chemical improvements include:

A. Flow Fill.

B. Soil Mixed Wall, Deep Soil Mixing.

C. Grouting and lime additions.

IV. **Mechanical improvements of fills and/or fill foundations.**
These methods reinforce fills or fill foundations with metal, polymeric or natural fiber inclusions, or with the installation of a grid of columns that may be steel piles, sand columns, or stone columns. Inclusions may be horizontal layers, or random inclusions.

Columns are vertical and are often designed to carry loads through a compressible layer to a more competent soil layer at depth.

A. Mechanically stabilized fills (by any of the methods of metal grids, metal strips, geotextiles, geogrids, etc.).

B. Piles, micropiles, and soil nails, often applied to a fill foundation.

C. Sand columns, Stone columns, in fills or fill foundations.

V. **Shallow foundations for bridge abutments.**
If abutments are supported on footings that bear on embankment fills, then there will be similar settlements of approaches and bridge decks, and no faults will occur.

VI. **Maintenance, especially maintenance joints in pavements.**
Pavement faults may be results from erosion of fills at abutments due to failed pavement joints. Aggressive maintenance joints could prevent pavement faults.

VII. **Adjust of pavement elevations in service.**
Pavement faults may be due to random differences in settlements of some abutments and some embankments. For random settlements, the remedy is an ability to adjust elevations of pavements after faults occur. Adjustments include:

A. Mudjacking for approach slabs.

B. Adjust of bridge bearings to change elevation of bridge decks.

Soil improvements to reduce pavement faults at abutments are useful if the faulting is the result of the global mechanism. Moreover, soil improvements applied to fills and fill foundations are effective only where the embankment settlements are greater than the settlements of abutments. Where the local mechanism is the cause, many of the soil improvements are not appropriate.

Section 3 of this synthesis reviews methods for soil improvement. The presentation consists of two parts. The first identifies the methods of soil improvement. The second lists literature sources, use of methods, and performance of methods.

OBSERVED SETTLEMENTS OF BRIDGES AND EMBANKMENTS

Total settlements of bridges and approaches, and the differences in settlements of bridges and approaches are the source of faults in pavements. It should be possible then to discern the occurrence of pavement faults, the magnitude of faults, and the evidence for causes of faults in terms of different total settlements of approaches and bridges

This synthesis gathers from the literature data on settlements and differences in settlements of bridges and approaches. From these sources it should be possible to discern the occurrence and magnitude of faults. In addition, it should be possible to compare the performance of bridge substructures on shallow foundations and on deep foundations. This comparison is a first assessment of the potential for the use of abutments supported on spread footings as a means of mitigating pavement faults.

This synthesis has collected more than 350 data points on settlements of bridges, and 50 points on settlements of embankments. The remaining data points are obtained from building foundations and from foundations for industrial applications including tanks. Data on settlements are used to determine mean values of settlements, to compare settlements for different foundation types and different soil conditions, to compute basic statistics for settlements, to examine differential settlements and to examine the relation of differential settlements to total settlements.

In section 4 of this synthesis, data on total settlements are presented. The synthesis has examined the data, has examined the settlement of deep foundations compared to shallow foundations, and abutments compared to piers. The synthesis goes on to consider settlements of embankments, examines settlements a function of the height of embankments, and for a few embankments, has been separated total settlements from settlements after bridge decks were completed. Settlement data, and the subsets of settlement data are detailed in section 4.

Nearly 60% of the settlements of bridge substructures are less than 1 inch. Higher values of settlement occur less frequently. Even the population of settlements between 1 inch and 2 inches is roughly one sixth of the population of settlements below 1 inch.

The data in Table 1-5 are a summary of total settlements. These data are a small set, and the data may be biased in the sense that the projects that have merited a report are inherently atypical. But given these data, there are several things to note. Bridge substructures supported on footings settle somewhat more than bridge substructures supported on deep foundations. The median settlements of approach embankments less than 30 feet in height is 1.6 inches, and the median settlement of abutments is 1.2 inches. The difference 0.4 inches is the magnitude of pavement faulting that can be supported on the basis of reported data on total settlements.

Foundation Type	**Mean Settlement μ, in**	**Median Settlement m, in**	**Standard Deviation σ, in**
Footings on Cohesionless Soil	1.5	0.6	2.2
Footings on Cohesive Soil	12.2	2.3	13.2
Single Piles and Pile Groups	3.2	1.0	6.2
Bridges	μ	m	σ
All Abutments	2.1	1.2	2.5
Abutments on Piles	2.0	0.8	2.6
Abutments on Footings	2.1	1.3	2.5
All Piers	3.2	1.2	3.3
Piers on Piles	2.7	1.2	2.9
Piers on Footings	3.5	1.7	3.4
Embankments (total)	μ	m	σ
All Embankments	23.9	15.4	29.0
Highway and Approach Embankments	23.3	7.1	31.8
Approach Fills less than 30 feet high	4.4	2.5	6.3
Approach Fills greater than 30 feet high	14.2	16.3	8.9
Embankments (post-construction)	μ	m	σ
All Embankments	3.4	2.5	5.1
Highway and Approach Embankments	3.4	2.5	5.1
Approach Fills less than 30 feet high	3.4	1.6	6.5
Approach Fills greater than 30 feet high	4.0	3.6	3.2

Table 1-5 Summary of Data on Total Settlements

In section 4, the summary of settlements prepared in this synthesis is compared to the summary in Moulton [et al. 1985]. The comparison reveals a pitfall in the use of the small data set of total settlements. In this synthesis, extreme values of settlement are not included in the computation of mean values of total settlement. In Moulton, it appears that extreme values were included. The effect in the two summaries is to reverse some of the apparent trends in the data. For example, this synthesis finds that abutments on shallow foundations have a mean value of settlements that slightly greater than abutment on deep foundations. Both mean values are around 2 inches. Moulton's summary indicates that abutments on deep foundations settle more than abutments on shallow foundations and that both settle nearly 4 inches in the mean. This indicates that the set of data is so sparse that it does not support conclusions on cause of faults.

The findings of this synthesis concerning observed settlements are:

- Data may include a disproportionate representation of bridges with settlement problems. This increases all mean and median values of settlement, and limits the usefulness of the data.
- Comparisons of settlement data from bridge abutments and from embankments should be made using median values of settlement to minimize the distortion in mean values due to a few large settlements.
- There is a difference of 0.4 inches in the median settlements of embankments and abutments. The embankment settlement is greater than the abutment settlement.
- The review of settlements provided by this synthesis uses data that is representative of data used in other, earlier summaries of bridge settlements.
- There is a need to carefully sift the data before averaging, and before seeking conclusions. Overall, these data are sparse. Apparent trends in the data must be used with caution.

PREDICTIONS OF TOTAL SETTLEMENTS

A concern closely related to observed settlements of bridges and embankments is the accuracy of predictions of total settlement. Predictions of total settlements are needed both to determine the potential for pavement faults at individual bridges and to assess the effectiveness of mitigations that might be employed. A basic computation in pavement faults is the comparison of expected values of total settlement of embankments and abutments. If these values are different, then a fault is possible. But if the settlement predictions themselves are not accurate, then the potential for faults can not be evaluated in advance, and the effectiveness of mitigations can not be determined.

This synthesis undertook an examination of the accuracy of predictions of total settlements. There are many studies available that have proposed methods of computation of settlements and have used data from laboratory experiments or from constructed projects to make comparisons of observations with predictions. This synthesis compiles the results of these studies. More than 1500 comparisons of predicted and observed settlements have been collected. The majority of the comparisons are for settlements of footings.

The accuracy of predictions of settlements is examined by forming the ratio of predicted settlement S_{Pre} over observed settlement S_{Obs}

$$Rp = \frac{S_{Pre}}{S_{Obs}} \qquad \textbf{Eq. 1-2}$$

For this ratio, a value of 1.0 indicates that the prediction matched the observation. Values greater than 1.0 indicate that predictions are larger than observations. Settlement ratios are examined for many methods of prediction and for many categories of foundation type and soil type. It is found that all methods of prediction have similar performance. Methods are generally conservative though some individual results may be unconservative. In the mean, predicted settlements are more than 150% of observed settlements, and there is a substantial scatter in results. Settlement ratios are summarized in Table 1-6. The range and scatter of settlement ratios are shown in Figure 1-2 and Figure 1-3.

It is clear that the computations are successful in making conservative estimates of settlements in most cases. But conservative estimates, though useful as limits in design of structures, are not useful in the prediction of pavement faults. For faults, it is necessary to be accurate. The general finding here is that the ability to predict settlements in the course of normal design process is not sufficiently accurate for the assessment and mitigation of pavement faults.

Section 5 of this synthesis provides detail on the sources that are used to compute settlement ratios, detail on the individual methods, and a set of figures that examine subsets of the data. The examination of subsets seeks a correlation of settlement ratios with foundation type or soil type. There is no clear correlation with types of foundations or soils. There is a strong correlation between the scatter in settlement ratios and the number of comparisons. A greater number of comparisons is always associated with a greater scatter in settlement ratios. Methods that appear to give relatively narrow ranges of settlement ratios are those methods that have been compared with relatively few observations.

Category	Source / Method	Data Points	R_P Mean	R_P Median	R_P COV
All sources	All	1522	1.64	1.20	0.83
Overall method performance	Observational	23	0.91	0.85	0.45
	Strain factor	144	1.36	1.12	0.54
	Compressibility coefficient	152	1.65	1.42	0.75
	Elasticity	440	1.28	1.05	0.72
	Empirical†	478	1.24	1.09	0.58
	Empirical	764	1.92	1.36	0.84
Methods for footings	Oweis, 1979	10	1.65	1.59	0.36
	D'Appolonia, 1971	11	1.20	1.03	0.35
	Parry, 1977	13	0.88	0.85	0.37
	Schmertmann, 1986	16	0.97	1.13	0.32
	Skempton-Bjerrum, 1952	17	1.18	1.10	0.22
	Hough, 1959	20	1.78	1.85	0.35
	Asaoka, 1978	23	0.91	0.85	0.44
	Peck & Bazaraa, 1969	30	0.73	0.64	0.56
	Menard, 1975	31	1.07	1.03	0.38
	Wahls-Gupta, 1994	31	1.13	0.98	0.53
	Bowles, 1987	48	2.25	2.28	0.44
	Schultze & Sherif, 1973	56	0.97	0.97	0.33
	Papadopoulos, 1992	61	1.08	0.99	0.32
	Meyerhof, 1965	74	1.52	1.50	0.32
	Schmertmann, 1970	113	1.43	1.18	0.53
	Alpan, 1964	118	3.44	3.02	0.44
	Berardi & Lancellotta, 1994	125	1.06	0.83	0.76
	Buismann-DeBeer, 1957	132	1.63	1.32	0.80
	D'Appolonia, 1970	135	1.28	1.05	0.87
	Burland & Burbidge, 1985	145	1.45	1.24	0.63
	Terzaghi & Peck, 1948	168	2.77	2.43	0.66
Elasticity methods for footings	D'Appolonia, 1971	11	1.20	1.03	0.35
	Skempton-Bjerrum, 1952	17	1.18	1.10	0.22
	Bowles, 1987	48	2.25	2.28	0.44
	Papadopoulos, 1992	61	1.08	0.99	0.32
	Berardi & Lancellotta, 1994	125	1.06	0.83	0.76
	D'Appolonia, 1970	135	1.28	1.05	0.87
Empirical methods for footings	Oweis, 1979	10	1.65	1.59	0.36
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	Meyerhof, 1965	74	1.52	1.50	0.59
	Alpan, 1964	118	3.44	3.02	0.44
	Burland & Burbidge, 1985	145	1.45	1.24	0.63
	Terzaghi & Peck, 1948	168	2.77	2.43	0.66

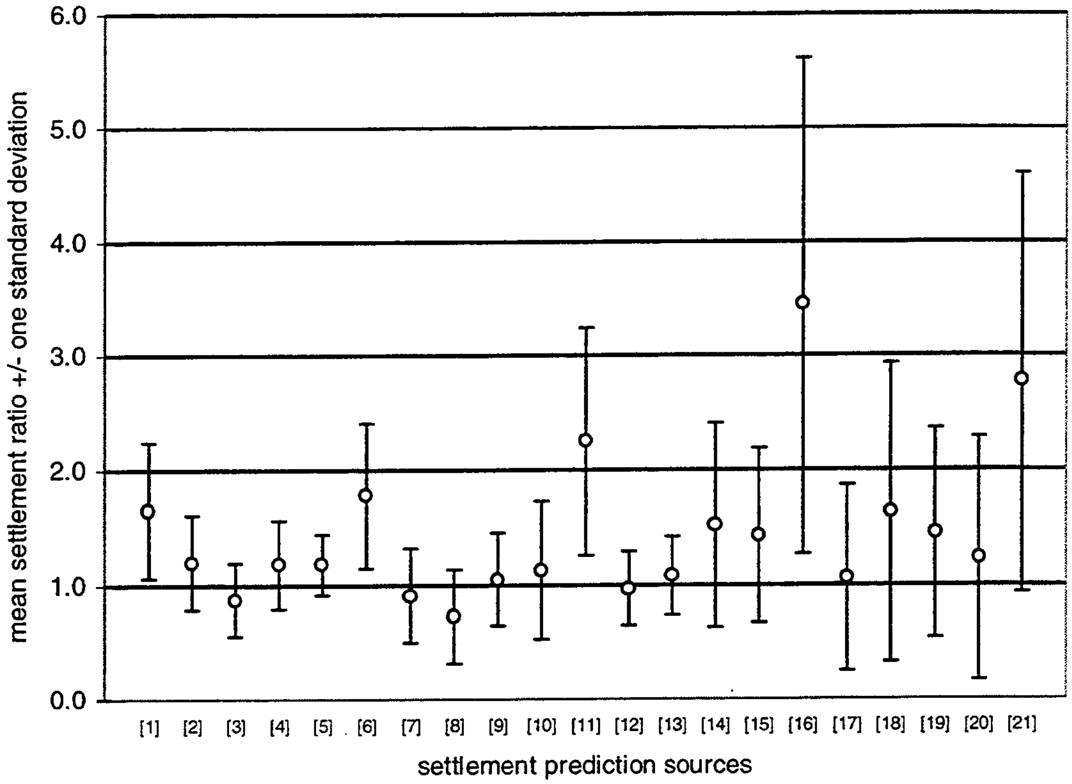
† without Alpan and Terzaghi & Peck source data

Category	Source / Method	Data Points	R_P Mean	R_P Median	R_P COV
Methods for piles	Meyerhof, 1976[CPT]	15	1.51	1.58	0.32
	Meyerhof, 1976[SPT]	16	1.21	1.08	0.30
	Yamashita, 1987	22	1.37	1.29	0.36
	Bazaraa & Kurkur, 1986	72	1.01	1.00	0.34

Table 1-6 Summary of Settlement Ratios

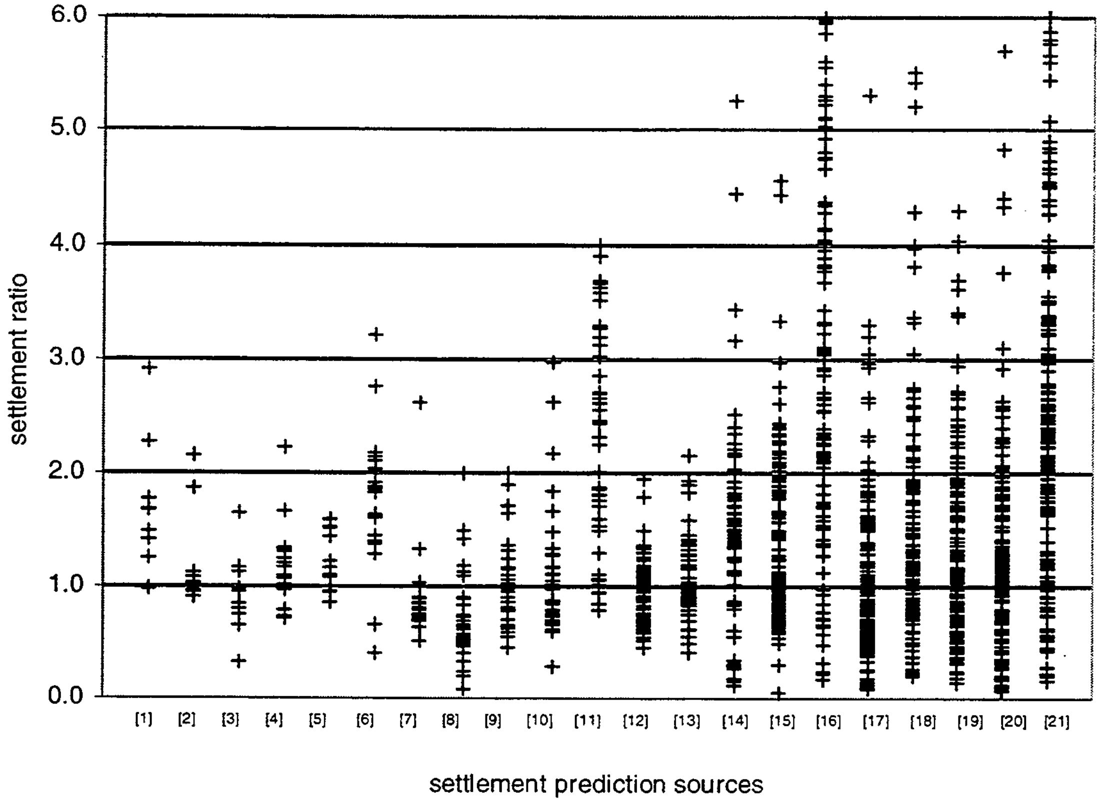
[CPT] Uses CPT data

[SPT] Uses SPT data



[1] Oweis, 1979
[2] D'Appolonia, 1971
[3] Parry, 1977
[4] Schmertmann, 1986
[5] Skempton-Bjerrum, 1952
[6] Hough, 1959
[7] Asaoka, 1978
[8] Peck & Bazaraa, 1969
[9] Menard, 1975
[10] Wahls-Gupta, 1994
[11] Bowles, 1987
[12] Schultze & Sherif, 1973
[13] Papadopoulos, 1992
[14] Meyerhof, 1965
[15] Schmertmann, 1970
[16] Alpan, 1964
[17] Berardi & Lancellotta, 1994
[18] Buisman-DeBeer, 1957
[19] Burland & Burbidge, 1985
[20] D'Appolonia, 1970
[21] Terzaghi & Peck, 1948

Figure 1-2 Performance of Settlement Prediction - All Methods



[1] Oweis, 1979
[2] D'Appolonia, 1971
[3] Parry, 1977
[4] Schmertmann, 1986
[5] Skempton-Bjerrum, 1952
[6] Hough, 1959
[7] Asaoka, 1978
[8] Peck & Bazaraa, 1969
[9] Menard, 1975
[10] Wahls-Gupta, 1994
[11] Bowles, 1987
[12] Schultze & Sherif, 1973
[13] Papadopoulos, 1992
[14] Meyerhof, 1965
[15] Schmertmann, 1970
[16] Alpan, 1964
[17] Berardi & Lancellotta, 1994
[18] Buisman-DeBeer, 1957
[19] Burland & Burbidge, 1985
[20] D'Appolonia, 1970
[21] Terzaghi & Peck, 1948

Figure 1-3 Performance of All Settlement Predictions

DIFFERENTIAL SETTLEMENTS

Section 6 of the synthesis examines differential settlements of bridge foundations. Differential settlement, specifically differences in total settlements between abutments and approaches cause pavement faults. Differential settlements, their occurrence and magnitude, and the ability to predict differential settlements are central to the study of pavement faults. However data in this area are limited. There are some data on differential settlements of discrete foundations, but little information on differences in settlement between approaches and abutments. Methods to predict differential settlements are also lacking. Often, differential settlements are estimated as a fraction of total settlements. One common rule-of-thumb estimates differential settlements as 50% of total settlement among similar foundations, and 75% of total settlements for dissimilar foundations.

This synthesis compiles data on total settlements from thirty-three bridges and two building projects, and uses these data to compute differential settlements. The bridges are from projects in North America and from Europe. Most are moderate span bridges, but two long span bridges are included. The settlements are all final or near-final settlements. For each project, differential settlements are computed, and mean values and standard deviations of total settlements are computed.

This synthesis examines the correlation of differential settlements with standard deviations and coefficients of variation (COV) in total settlements. This is an application of the simple idea that differential settlements result from variability in total settlements. A strong correlation is found. Normalized differential settlements can be related to COV of total settlements as follows.

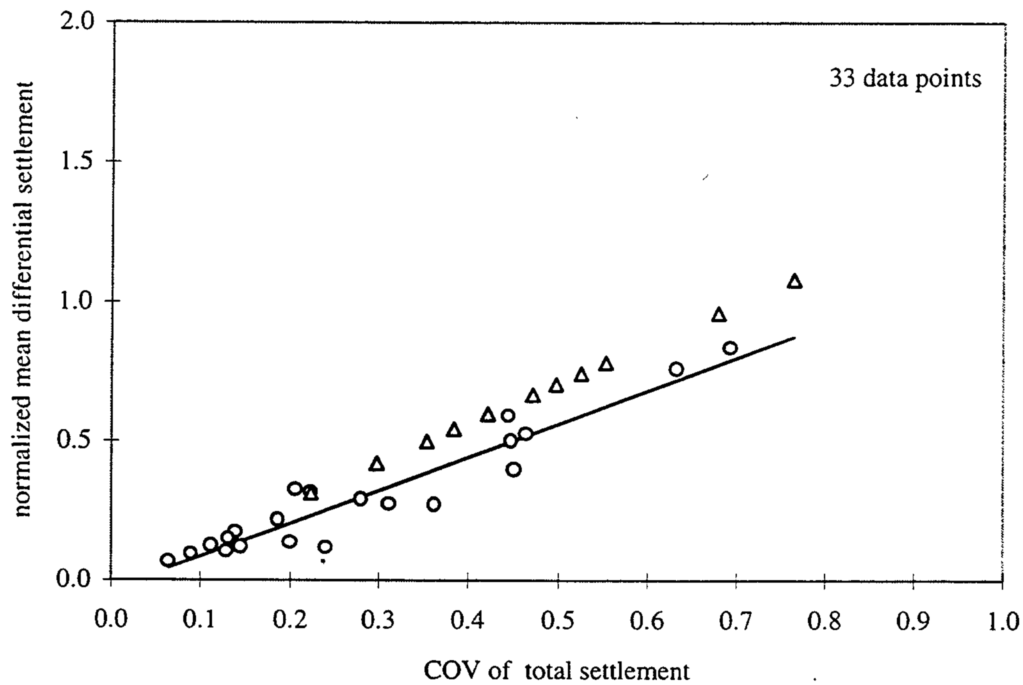
$$N\mu = \frac{\mu_D}{\mu_S}$$
$$COV_S = \frac{\mu_S}{\sigma_S}$$
$$N_\mu = -0.03 + 1.19 COV_S \qquad \textbf{Eq. 1-3}$$

where

μ_D = Mean value of differential settlement.
μ_S = Mean value of total settlement.
N_μ = Normalized differential settlement.
COV_S = Coefficient of variation of total settlement.

The relation is plotted in Figure 1-4. Notice that differential settlements are a function of only the variance in total settlements. The zero intercept for the relation shows that as COV_s approaches zero, differential settlements also approach zero. This finding has two important implications. First, differential settlements are reduced if the variability in total settlement is reduced. An attempt to mitigate pavement faults, a differential settlement problem, should address variability in settlements as directly as it addresses magnitude of settlements. Soil improvements, in particular, may be more valuable for their ability introduce uniformity in settlements rather than for their ability to reduce total settlements.

Second, the variability in total settlements indicates that differences in settlement are expected to occur even when the mean values of total settlement are equal for different foundations. This has a direct importance for the mitigation of pavement faults. If bridge abutments and bridge approaches are designed to have equal settlements in the mean, then the difference in settlements could easily be 0.5 inches to 1.0 inches for the magnitude and COV of total settlements observed in the data. An equality of mean value of settlements does not ensure the absence of differential settlements and may not be effective in the mitigation of pavement faults.



$$N_\mu = -0.03 + 1.19\ COV_s$$
$$r = 0.95$$
$$\sigma_{est} = 0.07$$

o data used in calculating regression equation
Δ bridges with only two known settlements

Figure 1-4 Normalized Mean Differential Settlement versus COV of Total Settlements

SPATIAL CORRELATION IN SETTLEMENTS

Data on differential settlements of bridge foundations are also used in this synthesis to examine the correlation in space of total settlements. While foundations can each settle different amounts despite a similarity of soil conditions, it is likely that foundations near each other will have similar settlements. The mean value of difference in total settlements can exhibit a dependence on the distance that separates foundations. The idea of a correlation of soil properties in space is not new. Previous work has demonstrated a correlation of standard penetration values, and of elastic modulus. Settlements depend on soil properties, and it is not surprising that settlements can be correlated in space.

The data on differential settlements for a bridge are used to examine spatial correlation. The dependence of differential settlements on distance between foundations is examined first in Figure 1-5. Differential settlements are grouped according to the distance between foundations. There is a general, though scattered trend of increasing difference in settlement with increasing distance. Given this trend, empirical correlation functions are fit to the data. For this same bridge, the resulting autocorrelation fits are shown in Figure 1-6. The four functions are similar to functions for correlation of soil properties that have been used in previous studies. Their forms all include exponential decay terms that reflect the expected loss of correlation of settlements as the distance separating foundations becomes large. Correlation functions for other projects are shown in section 6.

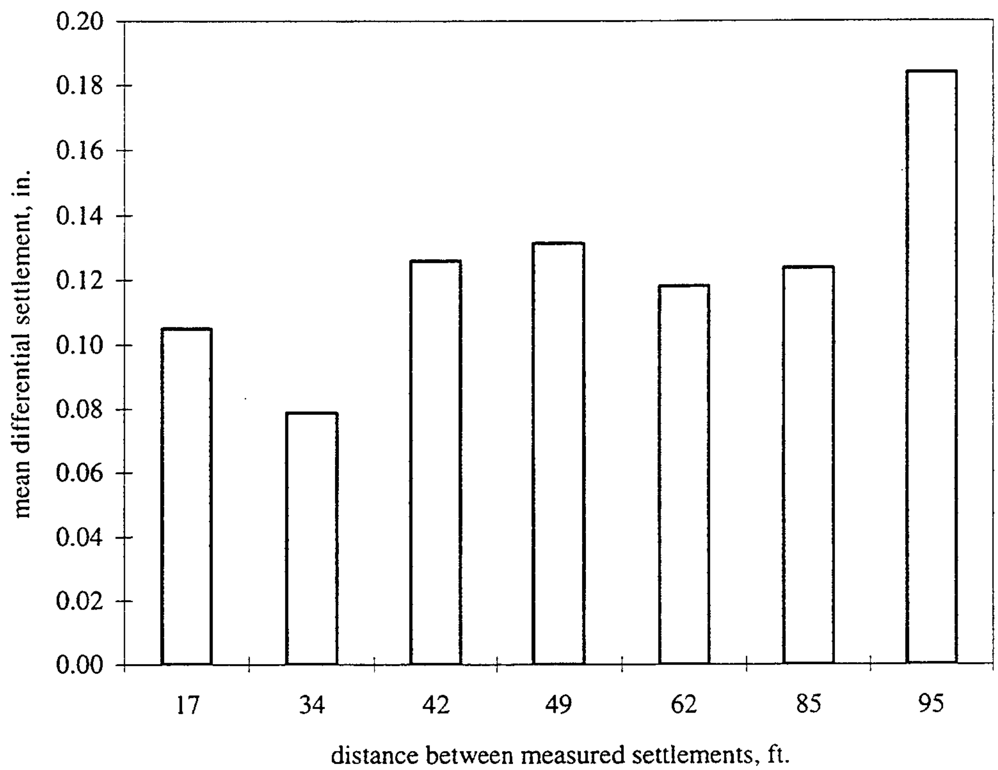


Figure 1-5 Differential Settlement versus Distance - Loppem Bridge

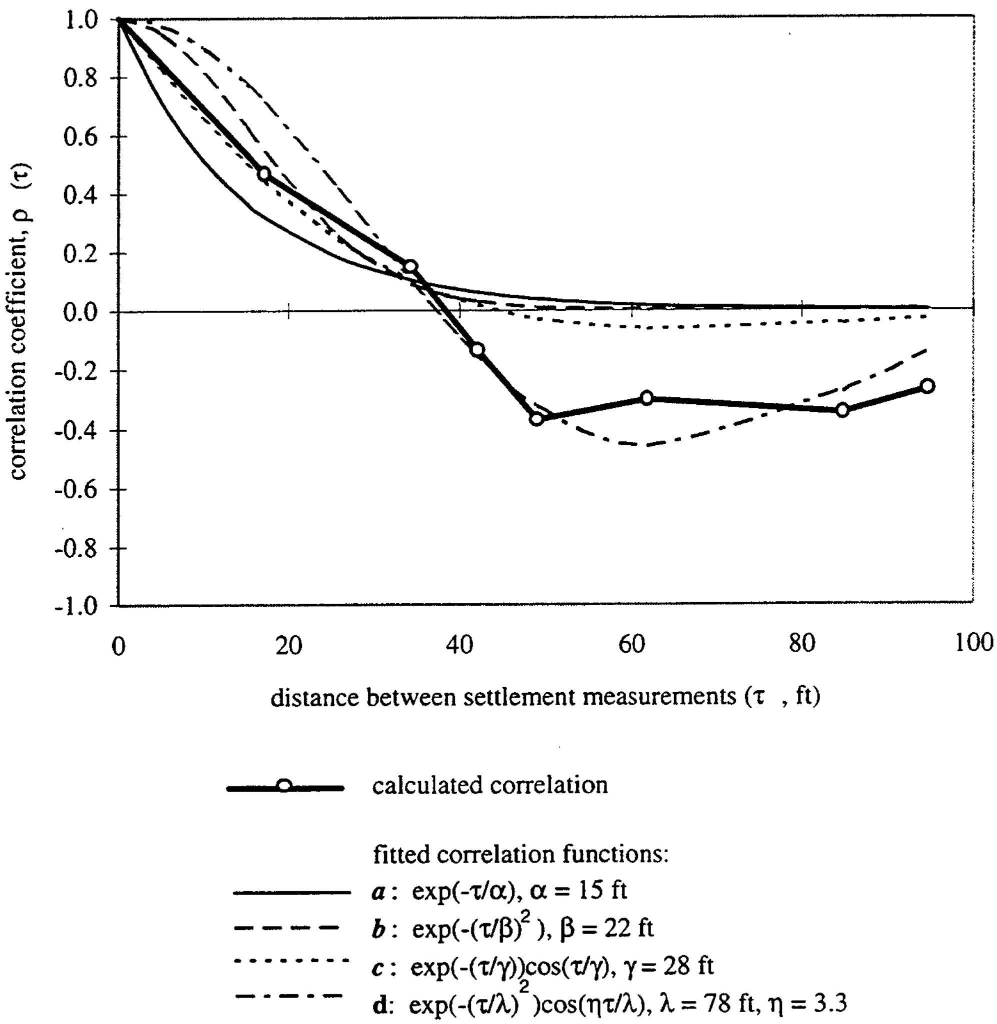


Figure 1-6 Spatial Correlation of Settlements - Loppem Bridge

PREDICTING DIFFERENTIAL SETTLEMENTS

The spatial correlation of total settlements affects the expected value of differential settlements. Given the relation between differential settlement and COV of total settlements, it can be shown that the expected values of differential settlements E[| D |] in the absence of spatial correlation is

$$D = S_i - S_j$$
$$E[|D|] = 1.13\,\sigma_s \qquad \textbf{Eq. 1-4}$$

where S_i and S_j are the values of total settlement at foundations i and j. If total settlements are correlated in space, then differential settlements among many foundations will be less than $1.13\sigma_s$, and this large value of differential settlement will be approached only for foundations that are well separated. When spatial correlation exists, differential settlements may be estimated as

$$\begin{aligned} E\left[D^2(\tau)\right] &= E\left[(\delta(\xi) - \delta(\xi+\tau))^2\right] \\ &= 2\sigma_S{}^2(1-\rho(\tau)) \\ &= 2\sigma_S{}^2\left(1-e^{-(\tau/\beta)^2}\right) \\ \sqrt{E\left[D^2(\tau)\right]} &= \sqrt{2\sigma_S{}^2\left(1-e^{-(\tau/\beta)^2}\right)} \end{aligned} \qquad \textbf{Eq. 1-5}$$

where

τ = Distance between foundations.
β = Characteristic distance that depends on the significance of the spatial correlation.

As the distance, τ, between foundations decreases the expected value of differential settlements is smaller. As τ increases, expected value of differential settlements approaches a constant. Distances are 'long' or 'short' in relation to length β that is characteristic of the spatial correlation at a site.

Spatial correlation of settlements and the resulting decrease in differential settlements for nearby foundations contribute to the identification of limits on settlements to protect structures. Bridges offer a limited tolerance for differential settlements. This tolerance is translated into project-level limits on total settlements using relations between total and differential settlements. At present, these relations may be as simple as the 50% rule-of-thumb. A better relation recognizes the dependence of differential settlements on variability in total settlements. In this way a more representative, and potentially more workable limit on total settlements is obtained. An even better approach recognizes spatial correlation of total settlements to compute limits on total settlement that are a function of variability in settlements and of span lengths.

TOLERABLE SETTLEMENTS OF BRIDGE BEAMS

Pavement faults will be reduced if there is the possibility to allow some settlement of bridge beams at abutments. In-service correction of pavement faults might be achieved partly by jacking of approach slabs or by adjustment of the elevation of bridge beams. Mitigation of this sort is limited by the tolerance of bridge beams for settlement.

The tolerance of bridge beams has been studied on the basis of performance of bridges, and it is performance only that is the basis of the commonly used limits on structural settlements (Table

1-7). An elastic analysis of stresses that result from settlements leads to much lower limits on tolerable settlements [Mouton et al. 1985]. The results of elastic analysis do not agree with the performance of real bridges.

Category	Angular Distortion *a/l* Observed Mean Tolerable	Observed Mean Intolerable	Proposed Limit
All Bridges	0.0025	0.0161	
Simple Bridges	0.0031	0.0241	0.005
Continuous Bridges	0.0022	0.0129	0.004
Concrete Bridges	0.0024	0.0232	
Steel Bridges	0.0026	0.0138	

Table 1-7 Field Data from Moulton [et al. 1985]

Real bridges are able to respond to settlements with inelastic deformations. It is apparently this potential for inelastic response that accounts for the difference between elastic stress analysis and observed bridge performance. Moulton was aware of this, and his study included creep effects in prestressed concrete beams to show that creep could increase the tolerance for differential settlement by as much as 300% compared to elastic analysis. For steel beams, Moulton did not propose an inelastic analysis. This synthesis performs a first examination of the role of inelastic response for steel bridge beams and the potential for the creation of a design basis that relates tolerance for settlement to the capacity for inelastic deformation.

Inelastic response in steel beams is available as a plastic rotation capacity. AASHTO recognizes the use of plastic rotation capacity in the design of braced, compact continuous steel beams. This synthesis proposes a method of design that uses compact steel beams, selects these beams to carry all loads and overloads elastically, and uses the plastic rotation capacity of to accommodate differential settlements. In this approach, it is necessary to identify an analysis of the plastic rotation capacity of compact steel beams and to select an appropriate upper bound on plastic rotations.

This synthesis examined several proposed models for predicting the inelastic rotation capacity of steel beams. This synthesis also accumulated a database of tests of steel beams for plastic rotation capacity. These data are used to check the models. There are good sets of data available both from tests in the 1970s and 1980s conducted as a part of the development of autostress design methods for bridges, and tests in the 1950s and 1960s conducted for investigations of plastic design methods for steel frames.

The model proposed by Kemp and Dekker [1991] offers a generally accurate and often conservative estimate of the ultimate rotation capacity of steel beams. Since the model estimates ultimate rotation, a lesser tolerable rotation capacity must be identified. In some of the tests of steel beams, it is possible to identify the onset of local flange buckling. For compact flanges, local buckling occurs at a rotation that is at least 20% of the ultimate rotation capacity, if it occurs at all. In addition, the value of 20% of the ultimate rotation capacity corresponds to a moment value that is significantly below the peak moment. The use of a design limit on plastic rotations equal to 20% of the ultimate rotation capacity is proposed.

Section 7 of this synthesis reviews the limits on tolerable settlements for highway bridges, examines inelastic rotation capacity for compact steel bridge beams and develops a simple example of the application of plastic rotation capacity to the design of bridge beam for settlement. It is found that compact steel bridge beams can have a tolerance for differential settlements that is easily 400% greater than the current accepted limit.

FINDINGS

Findings of the synthesis may be summarized as follows:

- Causes of pavement faults can be grouped under two mechanisms: A global mechanism and a local mechanism. The global mechanism is consistent with the idea that embankments settle more than abutments that are supported by deep foundations. The local mechanism is consistent with the idea that local erosion or movement of embankment fills results in pavement faults.
- The global mechanism can be mitigated by the use of soil improvements or by the use of spread footings to support bridge abutments.
- The local mechanism can be mitigated by careful control of drainage, and by the use of chemically stabilized fills that are able to resist erosion and shifting.
- An engineering practice of pavement faults can be formed by the introduction of an additional limit state that examines the difference in settlements of embankments and abutments. This practice requires adequate methods for the prediction of total settlements and differential settlements.
- Methods for the prediction of total settlements yield conservative but scattered results. Comparisons of predicted settlements with observed settlements indicate that methods of predictions are not accurate.
- A relation between differential settlement and variability in total settlements is demonstrated, and could be developed into a tool for the prediction of differential settlements. There is also evidence that a spatial correlation of total settlements exists.
- New, and more liberal limits for tolerable settle of bridges can be developed. This synthesis proposes an inelastic method for computing the tolerable settlement limits of compact steel bridge beams. Similar methods for other materials and for other bridge elements may be developed.
- Pavement faults may occur even without the action of either of the two mechanisms. The inherent variability of settlements can cause differences in settlement among similar foundations that have the same mean settlement. Variability in settlement would cause faults at some but not all bridges, even when the mean magnitudes of settlements of abutments and embankments are all within tolerable limits.
- Pavement faults that are the result of variability in total settlements might be more easily corrected by the adjustment of bridges in service rather than by the use of soil improvement or other mitigations in the original construction.

RECOMMENDATION

There is a need of more data on the performance of bridges in service to identify the causes of pavement faults. Previous studies involving a few bridges are too small to be representative of the performance of the larger population of bridges. A simple, basic survey of settlements at many thousands of bridges is needed to determine the causes of pavement faults.

A field survey has been proposed, ands the Colorado DOT has rejected the idea. The proposed form for field data collection is shown on the next page.

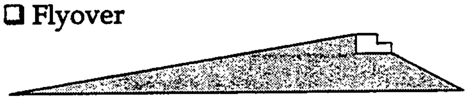
Pavement Faults Bridge ID:	University of Colorado at Boulder Colorado Transportation Institute
Inspector:	Weather:
Date: Time:	

Route Carried	No. of Lanes	Roadway Class
Route Under	No. of Lanes	Vertical Clearance
Other Under	Vertical Clearance	

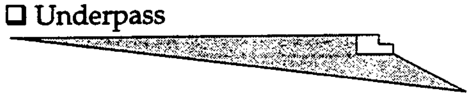
Abutment Type	Abutment Material	Abutment Foundation
❑ Full height	❑ Reinf. Concrete	❑ Piles
❑ Stub	❑ Steel	❑ Deep Footing
❑ Spill through	❑ Timber	❑ Footing on Fill
❑ Piles and lagging	❑ Other	❑ Other
❑ Other		❑ Unknown

Embankment Type

❑ Flyover



❑ Underpass



Embankment Height & Length

Approach Pavement					Wearing Surface		
❑ Reinf Concrete	❑ Asphalt	❑ Gravel	❑ Unknown	❑ Other	❑ None	❑ Asphalt	❑ Other

Bridge Deck

Deck Material	Wearing Surface	Skew	On Joint	Off Joint	Abutment Backwall
❑ Reinf. Concrete	❑ Concrete	❑ None	❑ Modular ❑ Finger	❑ Modular ❑ Finger	❑ Exposed
❑ P/S Concrete	❑ Asphalt	❑ On Deg. ____	❑ Gland ❑ Plate	❑ Gland ❑ Plate	❑ Under Deck
❑ Steel	❑ Timber	❑ Off Deg. ____	❑ Comp. Seal ❑ Cold Jt	❑ Comp. Seal ❑ Cold Jt	❑ Under App
❑ Timber	❑ Gravel		❑ Continuous Pavement	❑ Continuous Pavement	❑ Unknown
	❑ None				

Bridge Deck Condition

Deck Surface	On Joint	Off Joint	Abutment Backwall
❑ Good	❑ Good	❑ Good	❑ Good
❑ Cracks	❑ Uneven, App ↓ Mag. ____	❑ Uneven, App ↓ Mag. ____	❑ Cracks
❑ Spalls in Wearing Surface	❑ Uneven, App ↑ Mag. ____	❑ Uneven, App ↑ Mag. ____	❑ Spalls
❑ Spalls in Deck	❑ Cracked Pave.	❑ Cracked Pave.	❑ Chipped at Approach
	❑ Spalled Pave.	❑ Spalled Pave.	❑ Chipped at Deck

Remarks:

ON APPROACH PAVEMENT CONDITION

	Right Shoulder	Right Lane(s)	Left Lane(s)	Left Shldr
At Highway	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips
Middle Embankment	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips
At Abutment	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips

Remarks: ____________________

OFF APPROACH PAVEMENT CONDITION

	Right Shoulder	Right Lane(s)	Left Lane(s)	Left Shldr
At Highway	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips
Middle Embankment	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips
At Abutment	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips	❑ Good, No Dip ❑ Cracking, No Dip ❑ Dips

Remarks: ____________________

Notes: ____________________

Section 2 Occurrence of Pavements Faults. Reported Causes.

Previous studies of pavement faults and of other settlement problems at bridges have attempted to name causes. These causes include consolidation within fills and fill foundations that may be due to poor construction, inadequate inspection and control during construction of fills, or poor design of embankments that led to movement and slumping of fill. Causes also include local loss or disruption of embankment fills due to erosion. Erosion may be due to failure of joints in pavements, to seasonal movement of abutments, or to movement of abutments caused by expansion of rigid approach slabs. Relevant sources in the literature are listed in Table 2-1.

Table 2-1 Summary of Reported Causes of Pavement Faults

Source	Bridges	Causes of Settlement	Notes
Ardani, 1987	10	Consolidation in fill and fill foundation. Poor compaction and drainage in fill. Erosion of slope.	Causes deduced from the construction and observed drainage paths.
Meade & Allen, 1989	6	Embankment subsidence due to poor compaction, steep slopes, erosion, and secondary consolidation.	Embankments noted as having thick lifts (2′-3′) and poor compaction settled much more than those with 1′ lifts and good compaction. 80% or more of embankment settlement occurred before approach pavement was laid.
Kramer & Sajer, 1991	9	Primary and secondary consolidation and creep of embankment foundation soils. Subsidence of embankment fills due to creep, frost, erosion, and truck traffic. Failure of expansion joints and movement of abutments.	The bridge-approach interface was excavated and checked for voids and irregularities. Construction records were reviewed.
James et al. 1991	7	Longitudinal growth of pavement. Water infiltration into embankment fills.	Approach pavement cores taken to determine growth of pavement due to chemical reactions. Heavy traffic areas investigated.
Moulton, 1986	204	Movement of abutments due to poor pile foundations. Inadequate lateral resistance movement of approach embankment due to consolidation of foundation soils. Settlement of fill. Sliding (due to slope or foundation instability).	Data from questionnaire analyzed for correlations. Causes compiled from questionnaire responses.

Source	Bridges	Causes of Settlement	Notes
Wahls, 1990	-	Consolidation of fill and fill foundation. Poor compaction and/or drainage. Erosion.	Synthesis of literature.
Hopkins & Deen, 1970	6	Primary consolidation of embankment foundation. Secondary compression and shear strain of fill foundation. Improper compaction of embankment fill. Loss of material form and around abutment. Lateral and vertical deformation of abutment.	Long-term monitoring (8 - 14 years).
Emmanuel 1978	-	Abutment movement due to growth of approach slab and settlement of fill.	From literature
McNulty 1979	1	Embankment movement due to erosion.	Fixed by replacement of fill with lightweight fill

The causes proposed in the literature can be grouped under two headings: Global mechanisms and Local Mechanisms (Table 2-2). Each mechanism includes several causes.

Mechanism	Performance	Cause
Global	Consolidation of embankment fills.	Poor compaction.
		Inadequate construction inspection.
	Consolidation of fill foundation.	Poor (pre)compression.
		Inadequate subsurface exploration.
		Inadequate construction inspection.
	Slumping of embankment	Steep side slopes. Design error.
		Movement (forward tilt) of abutment due to high earth pressure.
		Movement of fill through open abutment face.
Local	Local erosion of fill at abutment.	Failure of joints in pavement.
		Cracking / leakage in pavement.
		Seasonal tilt of abutment, and shifting of fill.
		Tilt of abutment due to expansion of approach pavement.

Table 2-2 Mechanisms, Performance, and Causes of Pavement Faults

Mitigations are specific to mechanisms. The global mechanism and its related causes are mitigated by improvement of fills or fill foundations. For example, chemical or mechanical stabilization of embankment fills are appropriate mitigations for pavement faults due to global mechanism. The local mechanism and its related causes are mitigated by better joints, by designing for movements of abutments, by provide fills that resist erosion (such fills are required only near the abutment), or by designing maintainable transitions between approach pavements and bridge decks.

The difference in mechanisms must be seen in a difference in the performance of embankments. Consistently greater settlements in embankments than in abutments are evidence of the global mechanism. Local disruption of pavements near abutments, with similar settlements in embankments and abutments, is evidence of the local mechanism.

Previous studies identify diverse, site-specific causes of settlement problems. Often, causes of settlement problems are identified after the fact, and there is a tendency to name probable causes rather than to prove the importance of individual causes. The most significant characteristic of the literature on this area is the persistence of pavement faults. Clearly, if causes are known already then there should be no difficulty in preventing new faults. It appears that causes of faults have not been correctly or completely identified. There may be an interaction of several contributing causes making both investigation and prevention of faults so difficult.

Section 3 MITIGATION OF PAVEMENT FAULTS

Mitigation of pavement faults may be broadly divided into methods intended to prevent the occurrence of faults and methods that remedy existing faults. Preventive methods are applied to new structures during their construction. Remedies are applied to structures in service. Methods for mitigation may be further divided into methods that are effective in prevention or remediation of faults caused by a global mechanism, and methods effective for a local mechanism.

Methods for mitigation take five forms (Table 3-1).

- Improvement of fills and fills foundations are preventive methods that mitigate faults due to global mechanisms by reducing the post-construction settlement of embankments.
- The use of abutments supported on embankments is a preventive method that mitigates faults due to global mechanisms by making abutment settlements similar in magnitude to embankment settlements.
- Approach slabs may be preventive or remedial and are effective for both global and local mechanisms of pavement faults.
- Maintenance activities are preventive, and are effective for faults caused by a local mechanism. Maintenance activities include the aggressive maintenance of pavements and joints, as well as newer strategies of removable/adjustable pavement sections.
- Mudjacking, fill replacement, repaving, and other rehabilitations of embankments or approaches are remedies for existing faults.

	Application	Mechanism
Soil improvement	Preventive	Global
Abutments on Footings	Preventive	Global
Approach Slabs	Preventive or Remedial	Global or Local
Maintenance	Preventive or Remedial	Local
Rehabilitation	Remedial	Global or Local

Table 3-1 Methods for Mitigation of Pavement Faults

Among these five categories of methods for mitigation of faults, most information is available for soil improvements. Abutments on footings are reported by DiMillio [1982] and by Grover [1978], though neither source provides a quantitative assessment of the effectiveness in preventing pavement faults. A survey of use of approach slabs is provided by Allen [1985]. Stewart [1985] reported on approach pavement types and their maintenance needs. Maintenance, as a means of controlling faults or other settlement-related problems, is not reported in the literature. Rehabilitation, short of replacement of pavement, is sometimes accomplished by mudjacking. More commonly, however, approach pavements are demolished and replaced with new pavement.

SOIL IMPROVEMENT

Soil improvements include compaction or drainage of soils, addition of grouts, or addition of mechanical stabilization. Compaction may be accomplished by surcharge, by dynamic loading, or by vibratory loading. Wicks or sand columns may be used to drain soils. Strips, grids, or textiles may be used to construct mechanically stabilized backfills. Stone, sand or lime columns, soil nails, or micropiles can be use for mechanical stabilization of unexcavated soils. Soil improvement may also include better construction practices and control, and more stringent specifications for fill materials. Methods of soil improvement are listed in Table 3-2.

Table 3-2 Soil Improvement Methods

Type	**Stone Columns**
Description	• Hole created by jetting or other methods and then backfilled with stone or sand compacted by impact and vibration. • 1.5′ - 4′ diameter depending on level of improvement, method of installation, stone size, soil strength. • Graded stones, 1.3 cm to 7.6 cm diameter [Juran et. al 1989]. • Layout in square or rectangular grids, on center. spacing of 5′-12′ depending on level of improvement [Welsh 1987]. • Blanket of sand and gravel or semi-rigid mat at top of soil mass transfers loads to columns which support by end bearing and/or side friction. • 10 - 35% of weak soil replaced, typical.
Installation	• Vibroflot "wet". Water jet with stone filled in behind and compacted as vibroflot is removed [Mitchell 1970]. Weak soil removed in this method. Most effective for soft to firm soils, high groundwater [Welsh 1987]. • Vibrodisplacement "dry". compressed air, smaller diameter holes. weak soil pushed aside. • Rammed. small hole drilled, stone rammed in with falling weight; recommended for diameters less than 80cm
Design	• Pile analysis must be performed (lateral support, skin friction, bulging, length) • Semi-empirical methods or finite element methods used to calculate strength of columns [Welsh 1987] • Bearing capacity may be 20 to 50 tons per column [Juran et al. 1989]
Use	• Improves bearing capacity, slope stability and decreases settlement [Welsh 1987] • Best for soils with unconfined compression strength of 15 - 50 kPa [Juran et al. 1989] • Wide acceptance in U.S. for soft cohesive soils • $45/m [Welsh 1992]
Projects	Meade and Allen [1985]
Limitations	• Not enough lateral support from highly compressible soils [Juran et al. 1989] • Not recommended for soils w/ sensitivity > 5 (soil becomes remolded during installation)

Type	**Sand Compaction Piles**
Description	Hole backfilled with compacted sand.
Installation	• Pipe driven in by vibration. • 4.5 to 6 ton hydraulic or electric vibrator. • Sand poured in and casing pulled out and pushed back in at intervals to compact sand. • 24" to 32" diameters, up to 80" diameters. • 50' piles installed in twenty minutes. • Strong sand piles installed with additional horizontal vibration.
Design	1. Assume required reinforcement and pick spacing. 2. Estimate initial and final void ratio of soil. 3. Find required volume of sand and diameter. 4. Determine diameter of casing - estimated as 1.5 times larger than pile diameter. 5. Iterate to find reasonable size.
Use	• In Japan for reclaiming land from the sea. • Improves stability. • Reduces settlement. • Protects soil from liquefaction. • Accelerates primary consolidation [Barksdale 1987].

Type	**Soil Mixed Wall (SMW) or Deep Soil Mixing (DSM)**
Description	Grout mixed in with soil to form soil-cement columns.
Installation	Auger w/ hollow stem of 22"-40" diameter [Welsh 1992].

Type	Soil Nailing
Description	Nails • Driven Nails - low cost, small diameter (#5-#11 Bars) 50 ksi some with annulus to shoot grout through after installation to seal [Welsh 1987]. • Grouted nails - high strength (150 ksi) #5-#11 bars. • Jet-grouted nails - composite inclusions (30-40cm diameter) [Welsh 1987] made of grouted soil with central rod. • Corrosion protected nails - (French) double encapsulated nails. • Solrenfor - steel bar and grout protected by steel or plastic casing • Intrforcolor - prestressed to keep grout under compression. • US contractors use resin bonded epoxy nails [Welsh 1987]. Facing • Facing: Ensures local ground stability, limits decompression (for tunnel excavation), protects ground in a continuous manner, flexible, conforms to irregularities [Welsh 1987]. • Shotcrete - most common facing, 10-25 cm thick, reinforced with welded wire mesh. Simple, cheap. steel fibers can be added to reduce cracking [Elias and Juran 1991]. • Welded wire mesh for facing of fragmented rocks or intermediate soils [Welsh 1987]. • Galvanized wire mesh for permanent structures [Elias and Juran 1991]. • Cast-in-place reinforced concrete or prefabricated concrete or steel panels.
Installation	• Driven nails using pneumatic or hydraulic hammers (no drilling) [Elias and Juran 1991, Welsh 1987]. 2-4 nails per 10sq.ft driven @ 3-5 nails/hr. • Grouted nails placed in boreholes (10-15cm diameter, 3.5"-12") w/ spacing of 1m-3m, 4'-6' [Welsh 1987, Elias and Juran 1991] conventionally cemented or resin grouted. • Jet-grouted nails - vibro-percussive driving of rod, grout injected through annulus in bar causing hydraulic fracturing of surrounding soil provides recompaction and improvement of surrounding ground. • Rods attached to facing by steel plates or cladding [Elias and Juran 1991].
Design	• Lateral friction along rods, lateral passive soil thrust on rods (similar to piles) [Welsh 1987] • Pullout strength formulas, pullout capacity table, design charts [Elias and Juran 1991]. nails may be prestressed to 20% of working load [Elias and Juran 1991]
Use	• Excavations and at toes of slopes. • Primarily temporary structures because of lack of durability of nails and facing.
Projects	• More cost effective than tiebacks (10-30% savings). • Permanent walls - $25-30/sqft (shotcrete facing), $40-54/sqft (precast/cast-in-place facing). • Temporary walls -$15-$28 • Shotcrete is 30% of total cost, 40%-50% for other facing types. • All costs 1985-1988 [Elias and Juran 1991].
Limitations	Advantages: • Low cost, adaptability, easy modification [Welsh 1987], withstands larger total and differential settlement than conventional structures [Elias and Juran 1991, Welsh 1987]. • Rapid installation, light construction, failure of one nail is okay because of

	redundancy [Elias and Juran 1991]. Limitations: • Not effective in clayey soils which are affected by creep and saturation; not durable in aggressive environments [Welsh 1987]. • Groundwater drainage may be difficult to construct. • Number of nails may interfere with utilities, etc. • Displacements may be larger than conventional tiebacks, causing movement of nearby structures. • Nail capacity may not be economically developed in cohesive soils. • Long-term performance of shotcrete facing is unknown. • No good for organic soils, cinder ash or slag fills, rubble fills, industrial wastes, acid mine wastes, nor for cohesive soils with LL>50 and PI<20 (creep must be checked) nor for cohesionless soils with N<10, cohesive soils with unconfined compression less than 0.5 Tsf.

Type	**Micropiles**
Description	Small diameter, cast-in-place piles.
Installation	• Drilling and grouting or displacement method. Drilled hole (cased), rebar placed, cement rich, small aggregate concrete grout poured, casing withdrawn while concrete pumped (sometimes casing stays). • Concrete or mortar forced into ground making a reinforced soil structure. • Some expanded bases (bells). • Driving method difficult with small diameter. • Wide spacings.
Design	• Micropiles act like regular piles with little end bearing. • Tension and compression taken, only small bending moments, buckling in weak soils. • Designed like regular piles. • In-situ pull-out test should be used to check capacity.
Use	• For stability and to reduce settlement, not intended for support (<250mm diameter) • Root-piles - used for underpinning existing structures, reinforcement of weak earth masses, stability of slopes, or strengthening of soil around excavations
Projects	Foundations for bridge piers [Blondeau, 1984] Foundation for embankment [Korfiatis, 1984]

Type	Grouting
Description	Cement-like substance pumped into voids.
Installation	• Jet grouting - used japan and Europe for 15+ years. • Water jets excavate soil, grout is pumped in and mixed with soil to make columns. • Total mixing or partial removal of soil. • Up to ten-foot diameters. • Rapid set cement grout, chemicals also used [Welsh 1987]. • Chemical grouting - 30% of soil volume replaced; 1-1.5m spacing [Welsh 1992]. • Compression grouting -- up to 5000psf pressure, used for slabjacking [Bruce 1992]. • Hydrofracture grouting -- stable high-mobility cementing grout injected at high rates to fracture ground. • French add polypropylene to provide tensile and flexural capacity.
Design	• Increases strength and/or decreases permeability. • Increases total stresses, fills voids, locally consolidates and densifies soil. • Chemical grouting "glues" granular soils together, seals cracks to prevent water infiltration; best for sands w/ <20% fines. • Compaction grouting good for thin, loose, deep strata overlain by very dense strata [Welsh 1987]; more controllable than slurry, more expensive than vibro-compression.
Projects	• Jet grouting - $250/m for 1m diameter column, $35000 to move equipment [Welsh 1992]. usually cheaper to do other things instead [Bruce 1992]. • Chemical grouting - $0.5-1.0/liter, $75/linear m, $10000 mobilization cost. • Compaction grouting - $200/cu.m of grout, $50-60/m of placement, $10-50000 for mobilization [Welsh 1992].
Limitations	• Expensive, for special problems (uncontrollable seepage, voids, foundation underpinning), complex, results difficult to evaluate [Mitchell 1970]

Type	**Mechanically Reinforced Fills**
Description	Layers of soil with tensile reinforcing elements in horizontal planes between layers. Reinforcement types: • Strip reinforcement - metal (usually) or plastic strips. • Grid reinforcement - tensile resistant elements (metal or plastic) arranged in rectangular grids. Steel bar mats. Welded wire mesh. • Geotextiles. Woven or nonwoven. Facing types: • Shotcrete, cast-in-place concrete panels, precast concrete panels, welded-wire mesh, gabions, wrapped geotextile.
Installation	• Built up from base in lifts of 1 to 2.5 feet. • Reinforcing elements placed in horizontal planes between lifts. • Usually granular backfill, although it depends on system requirements.
Design	External checks (conventional wall and embankment checks) • Overturning, sliding, bearing, slope stability Internal checks • Strength of reinforcement. • Strength of connections. • Durability of reinforcement.
Use	• Reduces lateral spreading and differential settlement, improves embankment stability. • Cheaper alternative to displacement improvements or deep foundations. • Geotextile can be used at base of embankment as working platform. • Uniform granular soils may cause damage to grid during construction [Koerner and Wilson 1992]. • Foundation subgrade and base improvements by confinement and separation: Confining soils brings particles closer together and prohibits infiltration of weaker soils [Welsh 1987]
Projects	• MSE wall saves about 25-50% over conventional retaining walls: • Reinforcing material = 10-20% of total cost, backfill = 30-40%, facing = 40-50% [Christopher et al. 1990]. • Tweepad Mine [Blight and Dane 1989 , Smith 1989] - massive deterioration. • Haliburton [Haliburton et al. 1980] - embankment. • Road subbases and drainage [Hoffman and Turgeon 1983, Bonaparte et al. 1988, Dawson and Lee 1988].

Type	**Fiber Reinforcement**
Description	• Tensile resistant strands mixed in with soil. • Natural, synthetic, or metallic strands.
Installation	• Fibers must be uniformly distributed throughout soil, random orientation.
Use	• Experimental. higher bearing capacity, self-healing slopes when eroded. not fully developed [Mitchell and Villet 1987] • 1% yarn added to soil can produce a high friction-high cohesion soil. • Greater stability, reduced erodibility, lower permeability. • Texol (French method) [CalTrans 1991]
Projects	Lab tests [CalTrans 1991, Fletcher and Humphries 1991]

Type	**Dynamic Compaction**
Description	Repeated dropping of heavy weight. Good for most soils.
Installation	• 10 drops per spot is maximum economic limit. • 20 tons dropped from 100′ is maximum. • No conclusive evidence that shape of weight matters [Welsh 1987].
Design	Effective depth of densification $(0.3 \text{ } to \text{ } 0.7)\sqrt{Wh}$ where W is weight in tons, and h is height of drop. Densification usually to 40 feet.
Use	• Suitability based on grain size, saturation, permeability, and drainage. • Best for pervious, larger grain sizes. • Reduces settlement, may cause problems with nearby structures.
Projects	$7-15/sq.m of surface, $35000 for mobilization [Welsh 1992, Leonards et al. 1980]

Type	**Drains**
Description	Types: • Sand, uncompacted sand columns. • Prefabricated (wick) drains with plastic core.
Installation	• Hole augured and backfilled with sand for sand drains. • Prefabricated drains are pushed into the soil with a mandrel.
Design	In-situ samples required for design.
Use	• Shorten pore water flow paths, reducing consolidation time. • Often used in conjunction with surcharge.
Projects	Sarkar and Castelli 1988, Kyfor et al. 1988, Saye et la. 1988, Lamb 1980

Type	Vibro-Compaction
Description	Rearranges loose cohesionless soils into denser soil with vibratory probe.
Installation	• Hollow steel tube with eccentric weight causes horizontal vibration [Welsh 1987]. • Vibrator 10-15′ long, ~7 tons, can influence up to 14′ diameter. • Spaced on triangular patterns @ 6′-12′ on center. • Extra sand brought in or the site can be dropped in elevation.
Use	• Improves relative density up to 85%. • Improves to depths of 115′. • Increases bearing capacity, reduces foundation settlement, increases resistance to liquefaction and shear. • Efficiency decreases as cohesion increases. • Soils should be <12% silts and clay w/ <3% clay. • One of the most economical and effective methods of densifying deep deposits of granular soils (<20% fines w/ <3% active clays). • Often used in coastal plain sediments, alluvial soils, glacial deposits, or controlled hydraulic fills.
Projects	$25/m of depth, $10-20000 for mobilization [Welsh 1992].

Type	Lime-Columns/Lime Stabilization
Description	• Lime added to soil to cement fine particles together. • Hydrated lime - most popular. • Quicklime - increased usage (about 25% of all lime usage).
Installation	• In-place or off-site mixing. • Hole drilled and quicklime mixed in with soil to form columns. • Pressure injected lime slurry 7′ to 10′ deep at 5′ spacing [Chou 1987].
Use	• Used offshore in Japan, some with diameters of 3.5m and 70m deep. • Good for organic soils. • Better cementing and results with less organic materials, more minerals, high pH, >20% clay, PI>10, higher curing temperatures (min 20-25C). • Permeability of soil may or may not increase [Juran et al. 1989]. • Light structures may be constructed on lime columns [Welsh 1987]. • Decreases liquid limit and plastic limit. • Decreases compressibility and potential expansiveness [Basma and Tuncer 1991, Tuncer and Basma 1991, Chou 1987].

CONSTRUCTION, DESIGN AND MAINTENANCE

Soil improvement is stressed in the context of rigorous quality control and quality assurance for embankments [Kramer and Sajer 1991]. The quality of backfill materials is important [Wahls 1990, Ardani 1987, Hopkins 1985]. Granular, strong materials are needed. Lightweight backfill materials may be used to reduce consolidation of foundation soils [James 1991, Wahls 1990. Lightweight fills such as low density concrete, polystyrene, and clamshells have all been used as embankment fills. In addition, organic material such as peat, bark, and sawdust might be used if they remain below the water table. Good compaction is an obvious requirement for improving backfills. Thinner lifts can be used to increase the effectiveness of compaction. Compaction requirements vary according to soil types, but strict compaction criteria such as that used in California [Stewart 1985] and in Kentucky [Hopkins 1985] can be effective in reducing settlement problems within the embankment.

Erosion must be prevented. Kramer and Sajer [1985] recommend gentler slopes for the side slopes of the embankment. Wahls [1990] and Kramer and Sajer [1985] recommend slope protection. Colorado has in the past extended abutment wings along the sides of the approach to provide more protection and lateral support of the approach embankments [Ardani 1987]. James [et al. 1991] believes that expansion joints at the bridge-approach interface must be sealed watertight so that runoff can not infiltrate the embankment fill. Adequate drainage within the fill and prevention of runoff water is suggested by many authors. Wyoming, Idaho, Kansas, Alabama, and North Dakota all use some sort of drainage system behind the abutment backwall to prevent water from infiltrating the embankment backfill [*Study* 1989].

APPROACH SLABS

Approach slabs are often used in an attempt to mitigate pavement faults. Approach slabs are reinforced concrete slabs with one end bearing on the bridge abutment and the other end resting directly on the highway embankment or a sleeper slab that supports both the approach slab and the adjacent roadway pavement. Many transportation agencies design slabs to span voids. This leads to thick slabs. According to Kramer and Sajer [1991], approach slab lengths range from ten feet to as much as 120 feet, with an average length of 40 feet. According to data collected by the Colorado DOT [*Study* 1989], approach slabs are usually 6 to 18 inches thick.

Approach slabs do not always eliminate the bump at the abutment. Ardani [1978], Kramer and Sajer [1991], Stewart [1985] all report undesirable bumps at bridges with the use of approach slabs. Kentucky has discovered that approach slabs do not eliminate bumps. Instead, it may more difficult to repair bumps at bridges with approach slabs. James [et al. 1991] also reports that flexible approaches in Texas performed better than approach slabs. Stewart [1985] reports that asphalt approach pavements are repaired more often than concrete approach slabs. Maryland has eliminated the use of approach slabs from its new designs and now uses asphalt approaches exclusively.

Kramer and Sajer's [1991] concluded that the use of an approach slab should be determined on a site by site basis. They suggest that approach slabs should be used for most bridges, particularly when the settlement of embankment fills or foundation is expected. Approach slabs should not be used when little long-term settlement is expected or if no approach embankment exists.

REMEDIATION OF EXISTING FAULTS

Pavement faults are fixed by correcting the elevation of the approach pavement. Patching and filling can make faults more gentle, though durability of patches is often a concern. Many transportation agencies including the Indiana and Missouri DOTs [*Study* 1989] use slabjacking to return approach slabs to their original elevations. Slabjacking is done mechanically or with pressurized grouting. For mechanical jacking, threaded holes are prefabricated into the approach slab. When the slab settles too much, it can be jacked off the abutment of foundation to its original position. Slabjacking is also done with grouting. Called "mudjacking", the technique uses drilled holes in the approach slab to pump pressurized grout beneath the slab. The grout fills voids beneath the slab and, by proper hole placement, can correct any differential settlement that has occurred. Ardani [1987] states that mudjacking is only a temporary solution, and that it may actually aggravate the problem by cracking the slab, a phenomenon that other agencies have reported.

Kramer and Sajer [1991] report experimental use of an inflatable bladder to raise or lower approach slabs using air pressure. Replaceable precast slabs have also been proposed [Kramer and Sajer 1991]. Precast elements are removed and reset as necessary for continuing settlements of embankments.

Slabjacking can correct faults if the approach has settled more than the abutment. The abutment may settle more than the approach, if the abutment has a shallow foundation. DiMillio [1982] reports the use of jackable abutments in Washington. Jackable abutments work by installing hydraulic jacks under each bridge girder during construction. The jacks rest on jacking pads on the abutment. If settlement of the abutment occurs, a central hydraulic system can operate all the jacks and raise the deck to its original level.

Literature sources on mitigation or remediation of pavement faults are summarized in Table 3-3.

Table 3-3 **Summary of Improvement of Fills and Structures at Approaches**

USE KEY

routine	Method which has been proven and is a relatively standard practice. May be routine in another country if not in the US. Foreign use noted in parentheses.
experimental	Method which is not used often enough to be considered routine. May have been implemented as an experimental feature.
never used	Method which has not been put into service

PERFORMANCE KEY

good	At least a slight improvement over the regular method or untreated system.
poor	No improvement but no worse than the original.
bad	Mitigation which actually did more harm than good.
inconsistent	Some improvement but also some harm, sometimes at the same site.
unknown	No information on performance.

class	**proponent**	**name/description**	**use**	**performance**
fill foundation fill	Meade and Allen [1985]	Stone Columns: soil drilled or augered and filled with columns of stone (details).	routine	good
fill foundation fill	Holtz [1989]	Drains: artificial drainage put into the soil to increase rate of consolidation.	routine	good
fill foundation fill	Holtz [1989]	Lime Columns: lime mixed with soil in columns.	routine (Europe)	good
fill foundation fill	Holtz [1989]	Dynamic compaction.	routine	good
fill foundation	Holtz [1989]	Vibroreplacement (stone or sand columns).	routine	good
fill foundation	Juran [et al. 1989]	Stone Columns (details).	routine	good
fill foundation	Kyfor [et al. 1988]	Prefabricated Drains: geocomposite drains.	routine	good

class	proponent	name/description	use	performance
fill foundation	Barksdale [1987]	Sand Compaction Piles: large diameter columns of compacted sand.	routine (Japan)	good
fill foundation fill	Saye [et al. 1988]	Prefabricated drains.	routine	good
fill foundation	Das [1988]	Geotextile Interface -- layer between fill and fill foundation.	experimental	good
fill	Kramer and Sajer [1991]	Strict construction criteria.	routine	good
fill	Kramer and Sajer [1991]	Replacement: remove poor fill and replace with better fill.	routine	good
fill	James [et al. 1991]	Seal Joints: keeps water from infiltrating fill.	routine	good
fill	Hopkins [1985, 1986]	Compaction requirements (standards).	routine	unknown
fill	McNulty [1979]	Lightweight fill.	experimental	unknown
fill	Scholen [1992]	Aggregate Stabilizers: additives to improve performance of subbases.	experimental	poor
fill	Nunan and Humphrey [1990]	Aggregate Stabilizers.	experimental	poor
fill	Caltrans [1991]	Yarn Reinforced Soil: small amounts of yarn fibers added to soil to add strength.	never used	unknown
fill	Bruce [1992]	Grouting.	routine	inconsistent
fill	Holtz [1989]	Berms, Flattened Slopes: prevents erosion, increases stability of embankment.	routine	good
fill	Holtz [1989]	Lightweight Fill.	routine	unknown
fill	Holtz [1989]	Pile Supported Embankments.	routine (Europe)	unknown
fill	Holtz [1989]	Preconsolidation of embankment.	routine	good
fill	Holtz [1989]	Grouting.	routine	unknown
fill	Holtz [1989]	Blasting.	experimental	inconsistent
fill	Holtz [1989]	Thermal.	experimental	good
fill	Reckard	Geotextile separators to prevent differential settlement.	routine	poor

class	proponent	name/description	use	performance
	[1991]			
fill	Fowler [1989]	Geotextile reinforced embankment.	routine	unknown
fill	Huang [1990]	Bottom ash as fill.	experimental	good
fill	Juran [et al. 1989]	Lime -- mixed with soil to improve soil properties.	routine (Japan)	good
fill	Basma and Tuncer [1991]	Lime (mixed with clay).	experimental	good
fill	Tuncer and Basma [1991]	Lime (mixed with clay).	experimental	good
fill	Duncan [et al. 1987]	Reinforced earth embankments.	routine	good
fill	Bonaparte and Christopher [1987]	Reinforced earth embankments.	routine	good
fill	Leonards [et al. 1980]	Dynamic compaction.	routine	good
fill	Dawson and Lee [1988]	Grid reinforced clay.	experimental	poor
fill	Hoover [1987]	Drains.	routine	good
slab	Kramer and Sajer [1991]	Concrete approach slab	routine	inconsistent
slab	Stewart [1985]	Concrete approach slabs (standard design details).	routine	inconsistent
abutment	Timmerman [1992]	Mechanically stabilized earth walls as abutments.	routine	good
abutment	James [et al. 1991]	Relief joints: allows slab to grow without moving abutment.	routine	inconsistent
abutment	DiMillio [1982]	Jackable abutments: jacking pads under each girder to jack to original position.	routine	good

Section 4 OBSERVED TOTAL SETTLEMENTS

This section is a collection of data on settlements of bridges and embankments in service. Data are collected from studies of settlements of individual structures, and from surveys of settlements of many structures. Most of these data are from bridges and transportation projects, but some data from settlement of building foundations are included. This section presents data on settlements, and examines average settlements of embankments and bridge substructures. Data on settlements are summarized, and average settlements are computed. Average settlements computed here are compared to averages from other summaries of settlements of bridges.

Settlements of bridges in service are basic data. Any finding on causes of pavement faults must be consistent with the observed performance of bridges in service. For example, a global mechanism of pavement faulting is indicated if total settlements of embankments are greater than total settlement of abutments. This difference should be greater for bridges with abutments on deep foundations. Abutments on footings should show total settlements that are closer to settlements for embankments. Conversely, nearly equal settlements of embankments and abutments, and similar settlements among abutments without regard to type of foundations are not consistent with a global mechanism for pavement faults.

The numbers of data points for observed settlement of structural foundations and of embankments are shown in Table 4-1. Several categories of data are shown. First, data are separated as ACTUAL settlements and as APPROXIMATE settlements. ACTUAL settlements are measured values of settlements reported for individual substructures or embankments. APPROXIMATE settlements are summaries of settlements presented as histograms. Histograms are provided in reports of some surveys of settlements of bridges. Data are collected from a variety of studies. Some are surveys of the performance of structures or embankments. Others are long-term monitoring programs for one or a few structures. Still others are demonstrations of types of foundations.

Data content varies among studies. Some studies offer detailed information on soil conditions, embankment heights, and foundation types for substructures. Other studies offer settlement data with incomplete or missing information on embankments or substructures. Among the limited data available, this synthesis separates data based on type of foundation for substructure (deep foundation or shallow foundation), on embankment height, and on occurrence of settlement before or after construction of bridge decks.

The magnitudes of settlements are listed in Table 4-2, and histograms of settlements for categories of foundations and embankments are shown in Figure 4-1 to Figure 4-18. In Table 4-2, both the mean and median values of total settlement are presented along with standard deviations of total settlement. Table 4-3 lists the settlement data for bridges only, and offers somewhat greater detail in these data. Median values of settlement are consistently less than mean values. For most categories, mean settlements exceed median settlements by a factor of two (Table 4-2, Table 4-3). Total settlements are characterized by many occurrences at low values of settlement, few occurrences at higher values, and some occurrences at very high values.

For embankments, total settlements after completion of fills are presented along with settlements after construction of the bridge deck (called post-construction settlements). Post construction settlements are the appropriate values to use for examining pavement faults. Figures for settlements of embankments show both total settlements (Figure 4-19, Figure 4-20, Figure 4-21) and post construction settlements (Figure 4-22, Figure 4-23, Figure 4-24), and also distinguish among embankments greater than 30 feet in height (Figure 4-24) and embankments less than 30 feet in height (Figure 4-23). Post-construction settlements of embankments are larger on average than settlements of bridge foundations. For embankments less than 30 feet high the difference in median settlements between embankments and abutments is 0.4 inches.

Foundation Type	Bridges	Others	Total
Footings on Cohesionless Soil	198	266	464
Footings on Cohesive Soil	5	86	91
Single Piles and Pile Groups	162	34	196
Bridges	Actual	Approximate	
All Abutments	299	507	
Abutments on Piles	97	124	
Abutments on Footings	202	383	
All Piers	185	261	
Piers on Piles	65	91	
Piers on Footings	120	170	
Embankments (total)	Actual	Approximate	
All Embankments	66	66	
Highway & Approach Embankments	44	44	
Approach Fills less than 30 feet high	15	15	
Approach Fills greater than 30' high	10	10	
Embankments (post-construction)	Actual	Approximate	
All Embankments	25	25	
Highway & Approach Embankments	25	25	
Approach Fills less than 30 feet high	13	13	
Approach Fills greater than 30' high	10	10	
Duration	Bridges	Others	Total
Construction Monitoring	41	158	199
In-Service Measurement	9	133	142
Long-Term Monitoring	180	0	180
Unknown	258	210	468

Table 4-1 Number of Data Points on Settlement

The character of distributions of settlements is evident in the figures. The first of these (Figure 4-1) shows all of the data for structural foundations. Most settlements are 1 inch or less. Only a few larger settlements are observed. This type of distribution of total settlements is characteristic of all subsets of settlement data. In following figures, the distribution of settlements for all footings (Figure 4-2), for footings on cohesionless soils (Figure 4-3), for footings on cohesive soils (Figure 4-4), for pile foundations (Figure 4-5), for bridge substructures (Figure 4-6), and for a set of sub-categories of bridge substructures including type of substructure, type of foundation and type of soil are shown. In all subsets, the same characteristic distribution of settlements is observed.

Foundation Type	Mean Settlement μ in	Median Settlement m, in	Standard Deviation σ, in
Footings on Cohesionless Soil	1.5	0.6	2.2
Footings on Cohesive Soil	12.2	2.3	13.2
Single Piles and Pile Groups	3.2	1.0	6.2
Bridges	μ	m	σ
All Abutments	2.1	1.2	2.5
Abutments on Piles	2.0	0.8	2.6
Abutments on Footings	2.1	1.3	2.5
All Piers	3.2	1.2	3.3
Piers on Piles	2.7	1.2	2.9
Piers on Footings	3.5	1.7	3.4
Embankments (total)	μ	m	σ
All Embankments	23.9	15.4	29.0
Highway and Approach Embankments	23.3	7.1	31.8
Approach Fills less than 30 feet high	4.4	2.5	6.3
Approach Fills greater than 30 feet high	14.2	16.3	8.9
Embankments (post-construction)	μ	m	σ
All Embankments	3.4	2.5	5.1
Highway and Approach Embankments	3.4	2.5	5.1
Approach Fills less than 30 feet high	3.4	1.6	6.5
Approach Fills greater than 30 feet high	4.0	3.6	3.2

Table 4-2 Summary of Total Settlements

Category	Points	Mean, in	σ, in	Median, in	Min, in	Max, in
Substructures	484	2.5	2.9	1.2	0.0	48.0
On shallow foundations	322	2.6	2.9	1.4	0.0	36.0
On deep foundations	162	2.3	2.7	1.0	0.1	48.0
In sand	213	2.7	2.8	1.5	0.0	10.2
On shallow foundations in	194	2.4	2.7	1.2	0.0	10.2
On deep foundations in sand	19	5.7	2.2	6.6	0.2	9.0
Abutments	299	2.1	2.5	1.2	0.0	48.0
On shallow foundations	202	2.1	2.5	1.3	0.0	36.0
On deep foundations	97	2.0	2.6	0.8	0.1	48.0
On shallow foundations in	123	1.2	0.9	1.1	0.0	7.5
Piers	181	3.2	3.3	1.2	0.0	20.4
On shallow foundations	116	3.5	3.4	1.7	0.0	20.4
On deep foundations	65	2.7	2.9	1.2	0.0	16.8
In sand	86	4.9	3.3	6.0	0.0	10.2
On shallow foundations in	69	4.5	3.5	6.0	0.0	10.2
On deep foundations in sand	17	6.3	1.2	6.6	4.8	9.0

Table 4-3 Total Settlement Data - Bridges Only

	Settlement		
Category	**% < 1 in**	**% < 2 in**	**% > 4 in**
Abutments and piers	43	60	23
Abutments and piers on shallow foundations	41	61	22
Abutments and piers on deep foundations	47	57	23
Abutments	42	65	17
Abutments on shallow foundations	39	68	15
Abutments on deep foundations	48	59	23
Piers	45	52	30
Piers on shallow foundations	46	51	35
Piers on deep foundations	45	54	23

Table 4-4 Frequency of Total Settlements

PREVIOUS SUMMARIES ON BRIDGE SETTLEMENT

Surveys of bridge settlements involving several hundred bridges have been conducted by Moulton [et al. 1985], and by the Transportation Research Board (several authors, see TRR 678). These same sources are used in this synthesis, and other data from smaller, individual studies of bridges, and from studies of settlements of building foundations are added to these summaries. At the same time, there is only limited new data in the tables on settlements presented here. Therefore, the values of mean settlement computed in this synthesis should match the values of settlement reported by other authors.

Summaries of Moulton's [et al. 1985] field data on settlements are shown in Table 4-5 and Table 4-6. Moulton's average values of settlement from field data are compared to the average values of settlement computed in this synthesis in Table 4-7. Two comparisons are made. First, the fractions of the population of settlement data less than 2 inches, and greater than 4 inches are compared. These are a measure of the form of the distributions of settlement data. The percentages for Moulton and for this synthesis are similar in all categories. The mean values of settlements are compared as well, and it is found that the mean values from Moulton are higher than mean value in this synthesis. It is apparent that decision to exclude settlements greater than 4 inches from the computation of mean values has a noticeable affect on the mean value. If extreme settlements are included in the computation of the mean values, then the last column of mean values is obtained. By including the extreme values, the data analysis in this synthesis agrees with the presentation in Moulton.

Settlement	Data Points	Percent	Abutments	Percent	Piers	Percent
0 to 2 in.	336	56	184	49	152	69
2 to 4 in.	120	20	98	26	22	10
4 to 6 in.	46	8	31	8	15	7
6 to 8 in.	43	7	20	5	23	10
8 to 10 in.	12	2	11	3	1	0
greater than 10 in.	43	7	35	9	8	4
total	**600**		**379**		**221**	

Table 4-5 Distribution of Settlements of Abutments and Pier From Moulton [et al. 1985]

Category	Data Points[1]	Mean Settlement in	Minimum Settlement in	Maximum Settlement in
abutments and piers	605	3.3	0.0	50.4
on spread footings	388	3.0	0.1	42.0
on piles	214	3.8	0.0	50.4
abutments	376	3.7	0.0	50.4
on spread footings	254	3.7	0.1	35.0
on piles	122	3.9	0.0	50.4
piers	226	2.5	0.0	42.0
on spread footings	134	1.8	0.1	42.0
on piles	92	3.6	0.0	18.0

Table 4-6 Settlement Data From Moulton [et al. 1985]

Category	% < 2″ (Synthesis)	% < 2″ (Moulton)	% > 4″ (Synthesis)	% > 4″ (Moulton)
Abutments and piers	60	56	28	24
On spread footings	61	—	27	—
On piles	57	—	30	—
Abutments	65	49	21	25
On spread footings	68	—	19	—
On piles	59	—	26	—
Piers	52	69	39	21
On spread footings	51	—	40	—
On piles	54	—	35	—

Table 4-7 Distribution of Settlement Data in Synthesis and in Moulton [et al. 1985]

Category	Mean Settlement in	Mean (Moulton) in	Mean w/extremes in
Abutments and piers	2.5	3.3	3.5
On spread footings	2.6	3.0	3.4
On piles	2.3	3.8	3.7
Abutments	2.1	3.7	3.5
On spread footings	2.1	3.7	3.2
On piles	2.0	3.9	4.0
Piers	3.2	2.5	3.5
On spread footings	3.5	1.8	3.6
On piles	2.7	3.6	3.3

Table 4-8 Comparison of Subset Data From This Synthesis and From Moulton [et al. 1985]

[1] The total number of points for all substructures, abutments, and piers do not agree between Table 3.5.1 and Table 3.5.2. This same discrepancy exists in the original Moulton data. No changes were made here, and hence this discrepancy was carried to this study.

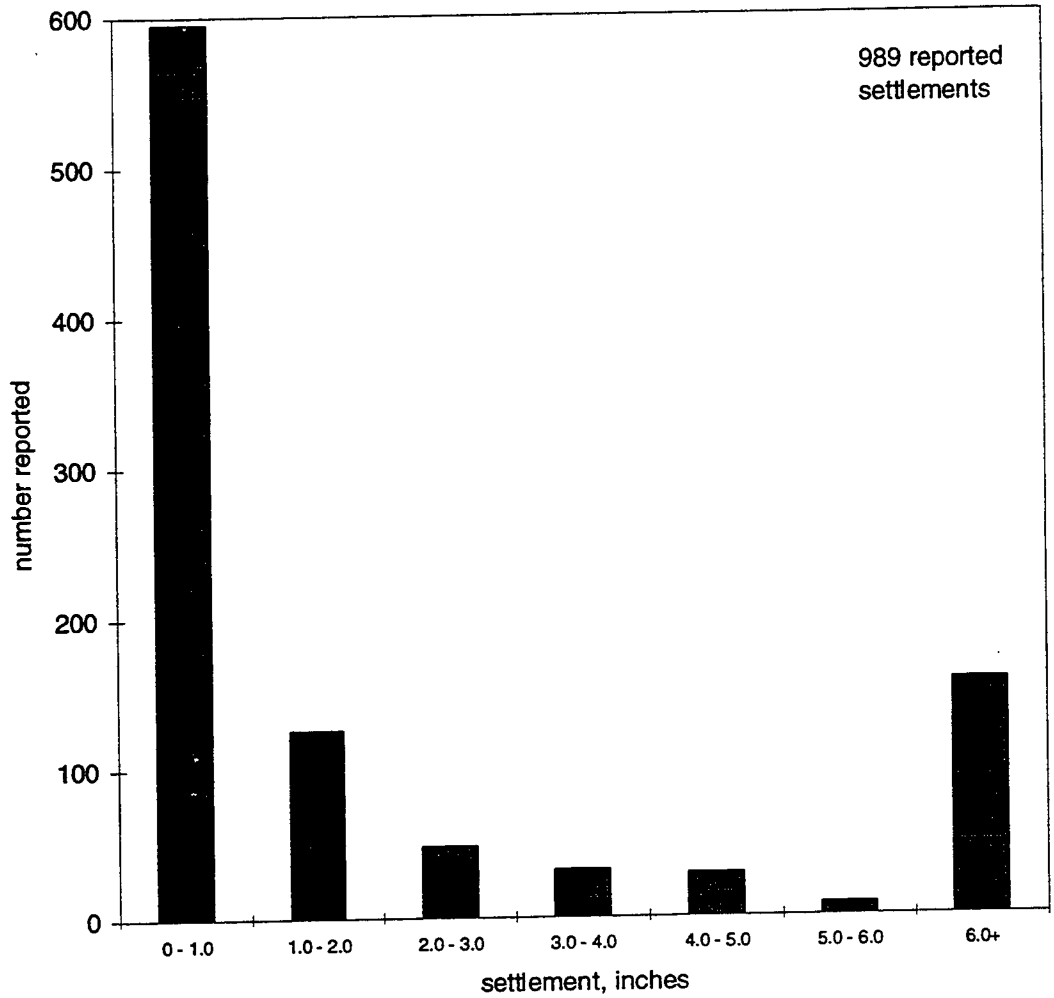


Figure 4-1 Total Settlements of Foundations

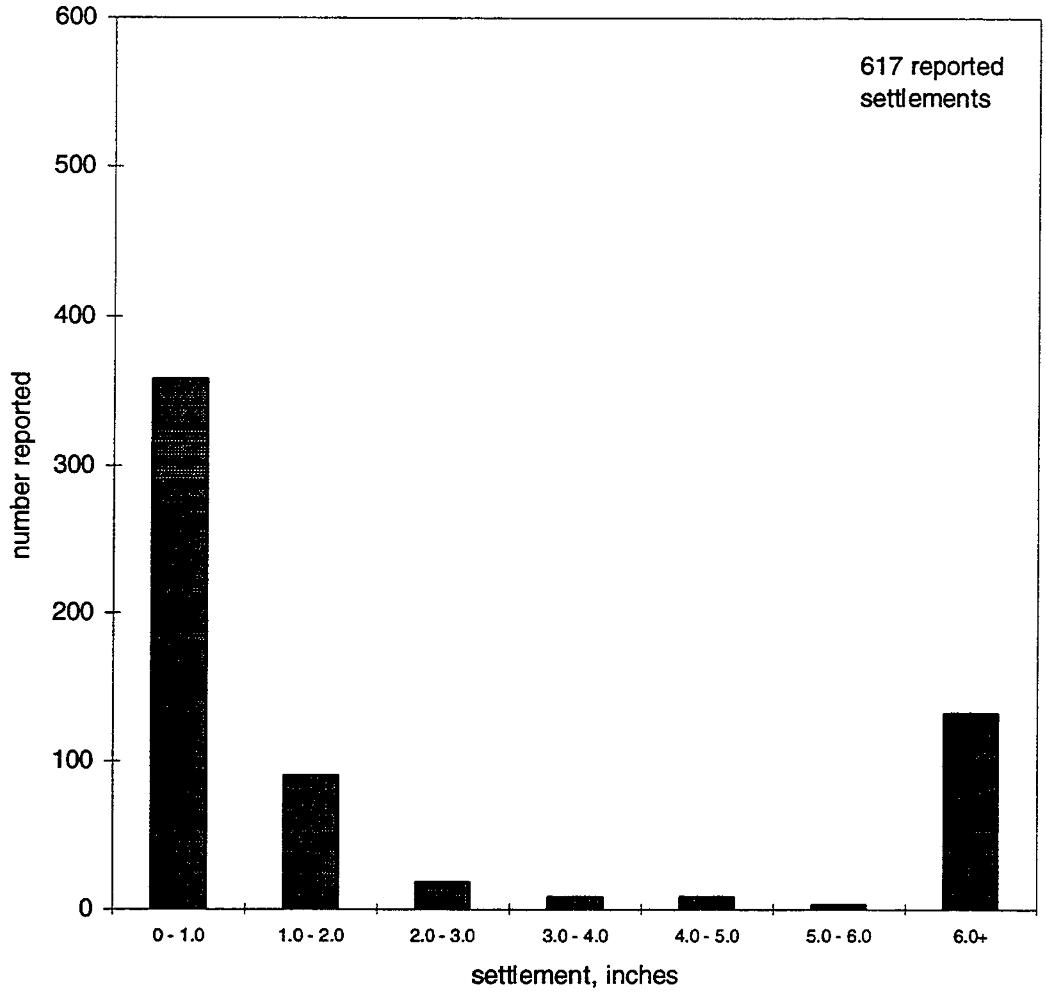


Figure 4-2 Total Settlements of Footings

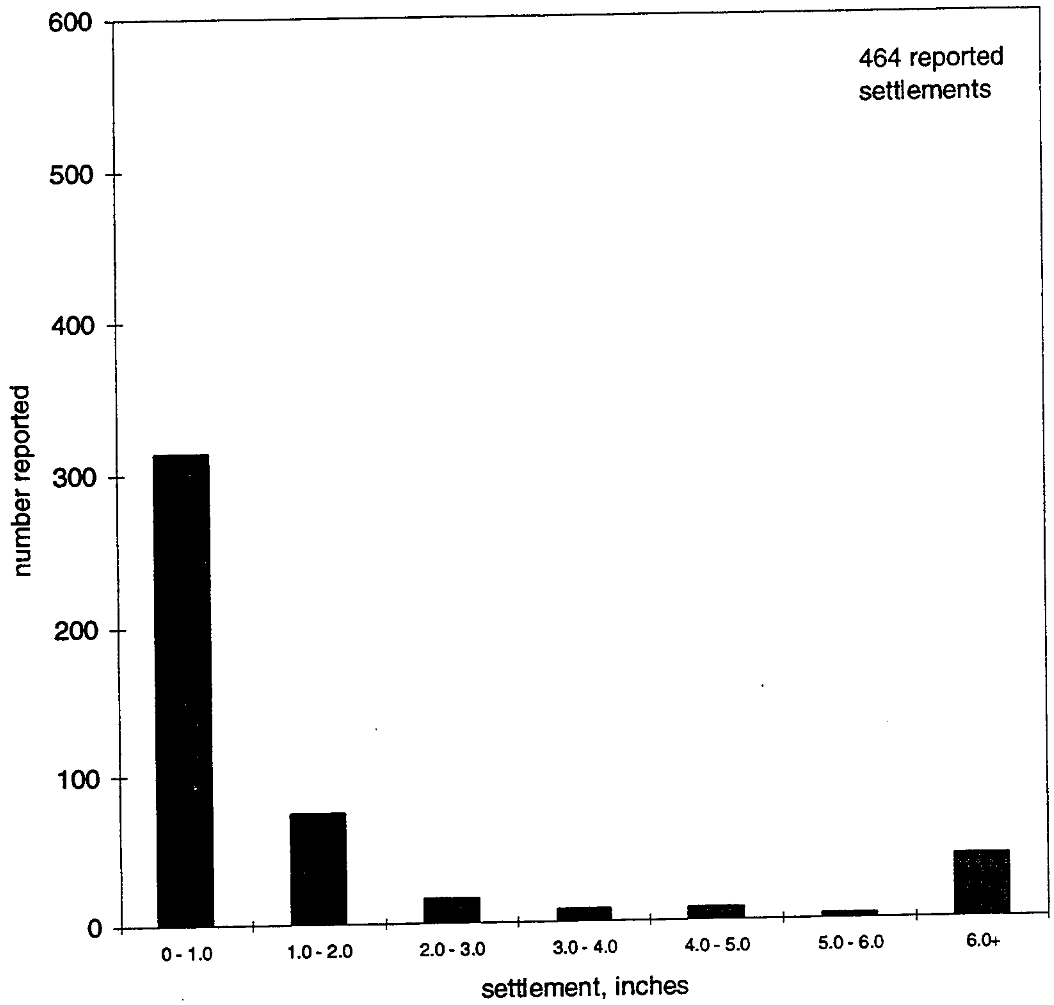


Figure 4-3 Total Settlements of Footings on Cohesionless Soil

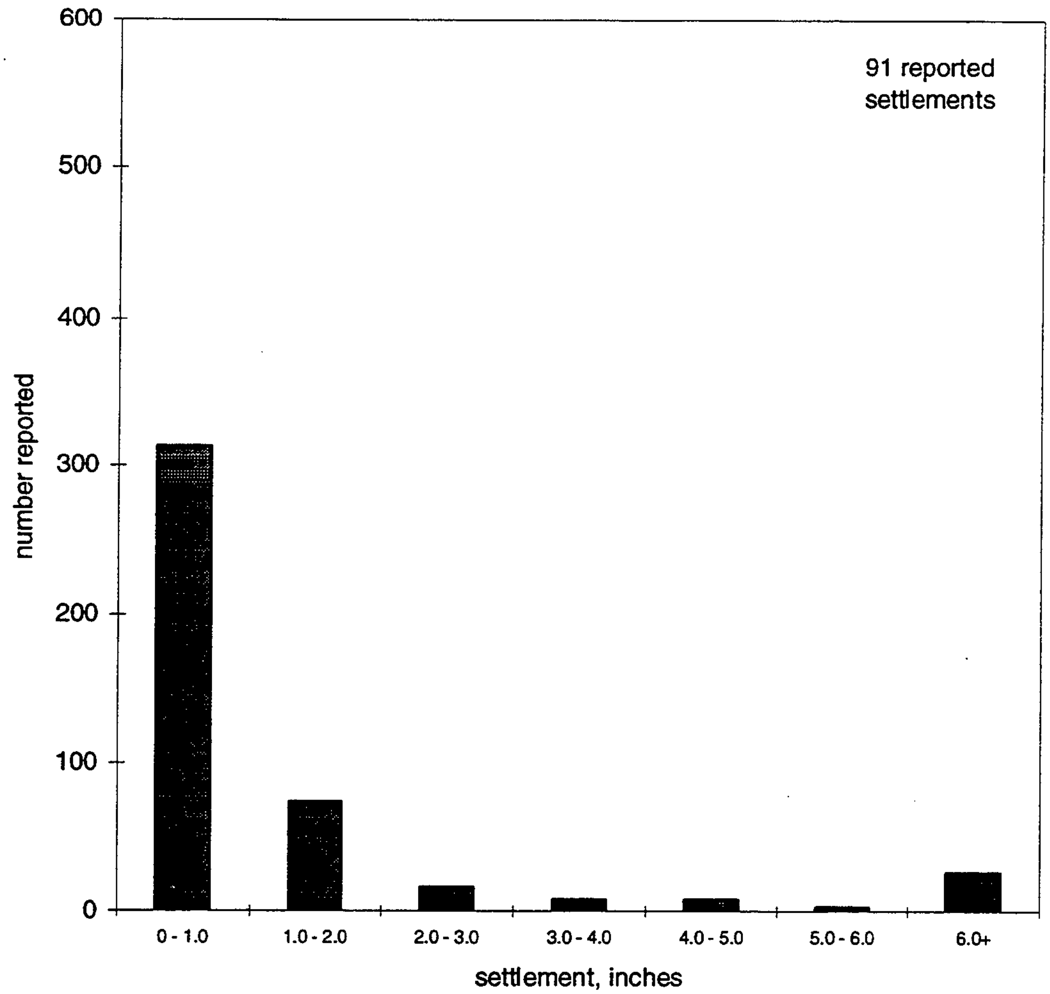


Figure 4-4 Total Settlement of Footings on Cohesive Soils

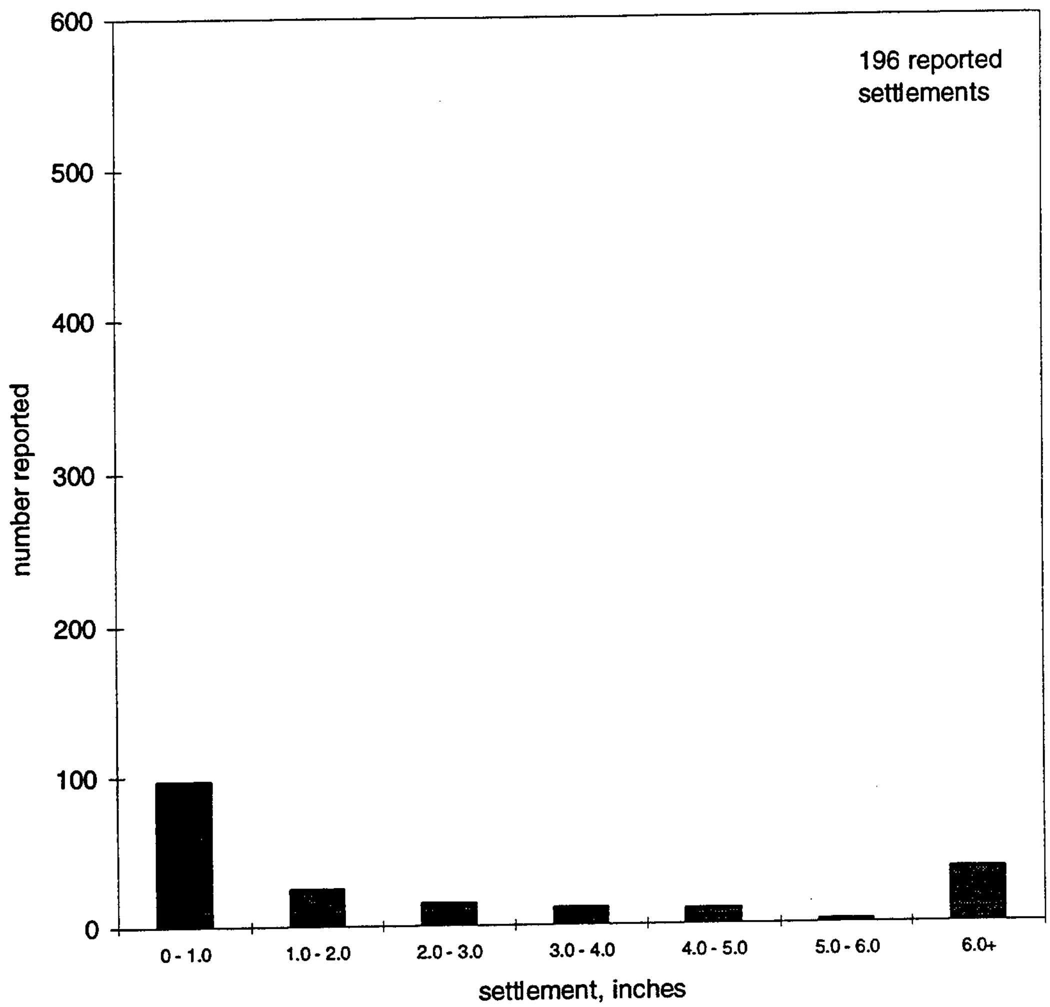


Figure 4-5 Total Settlements of Piles

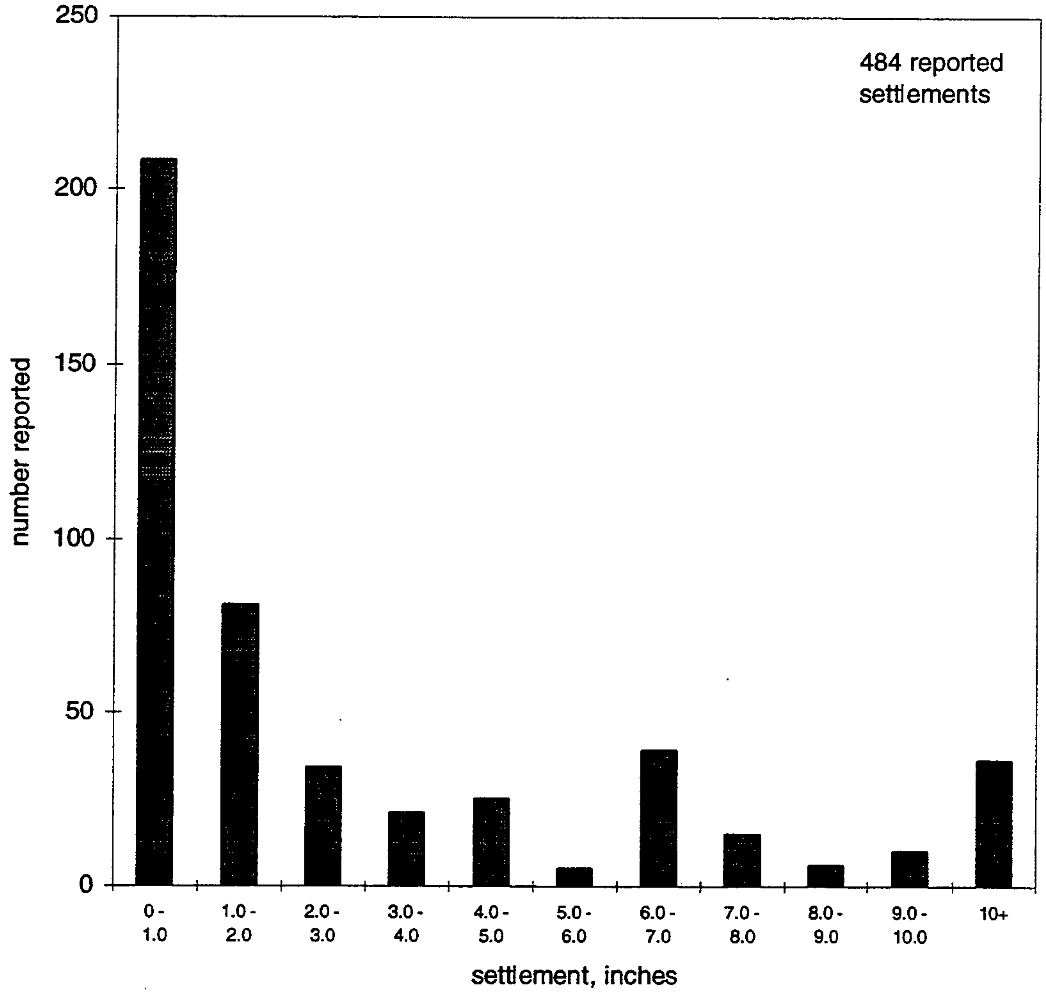


Figure 4-6 Total Settlement of Bridge Abutments and Piers

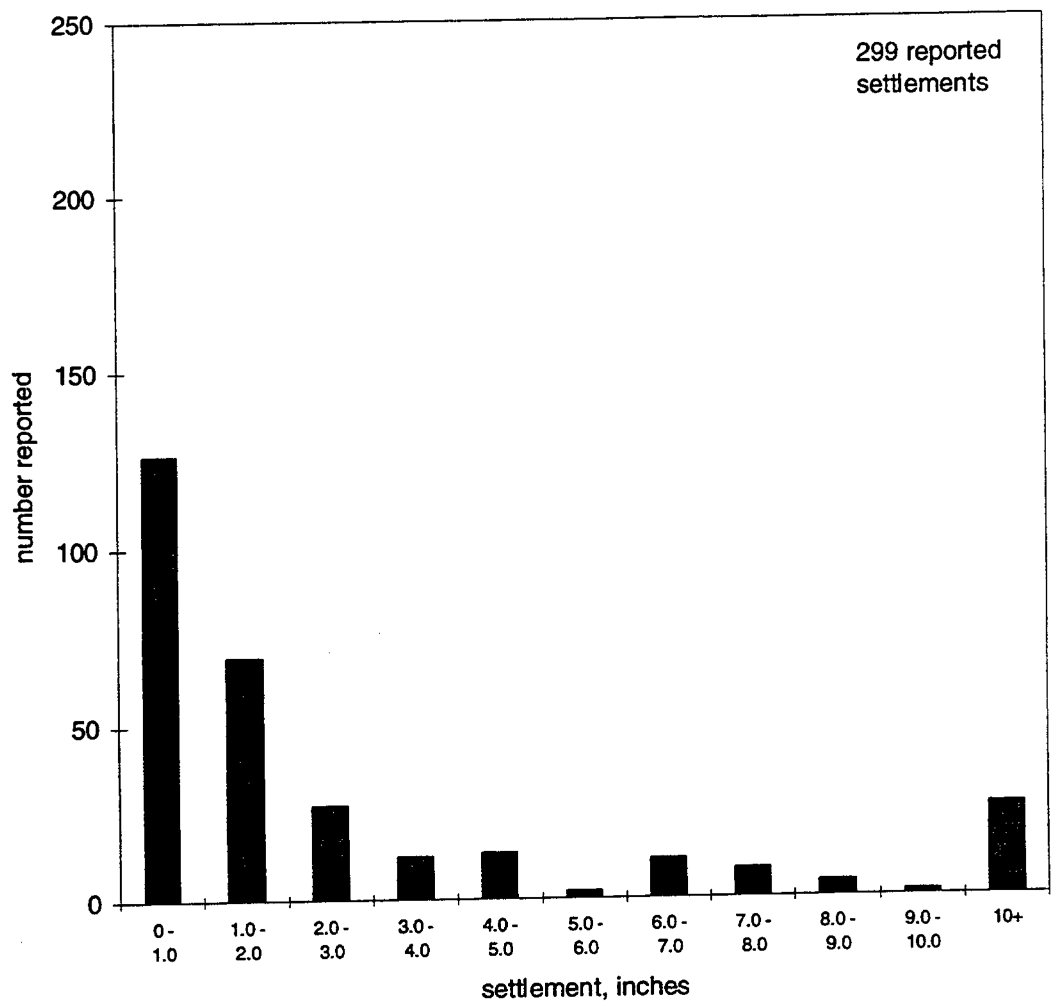


Figure 4-7 Total Settlements of Abutments

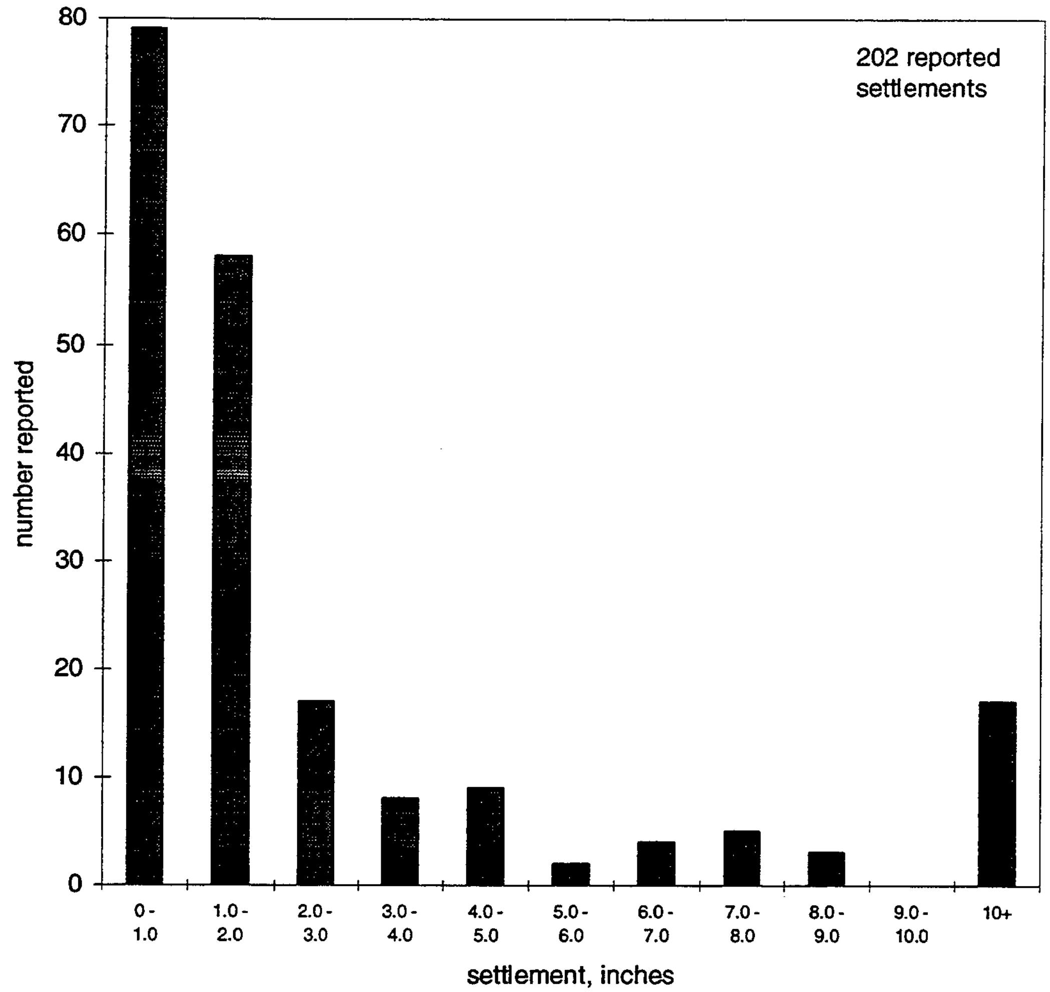


Figure 4-8 Total Settlements of Abutments on Footings

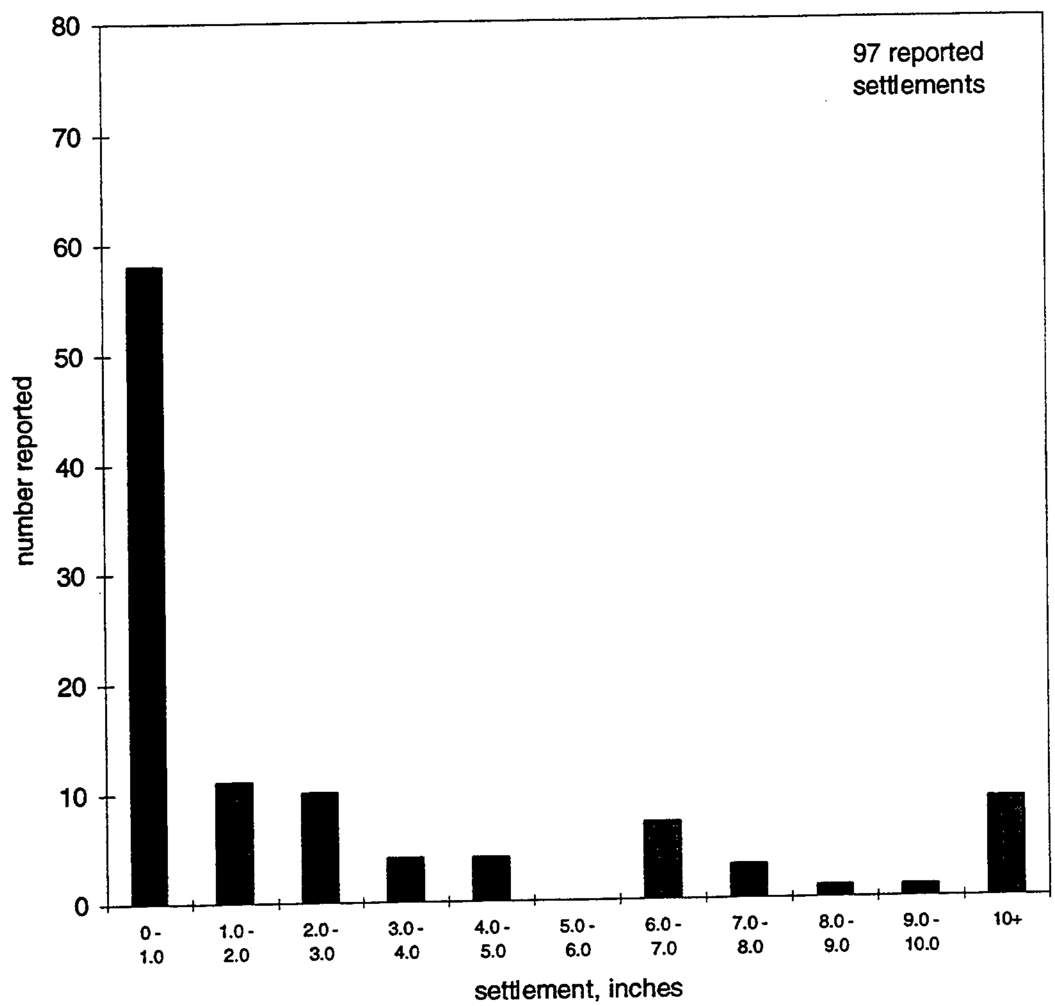


Figure 4-9 Total Settlements of Abutments on Piles

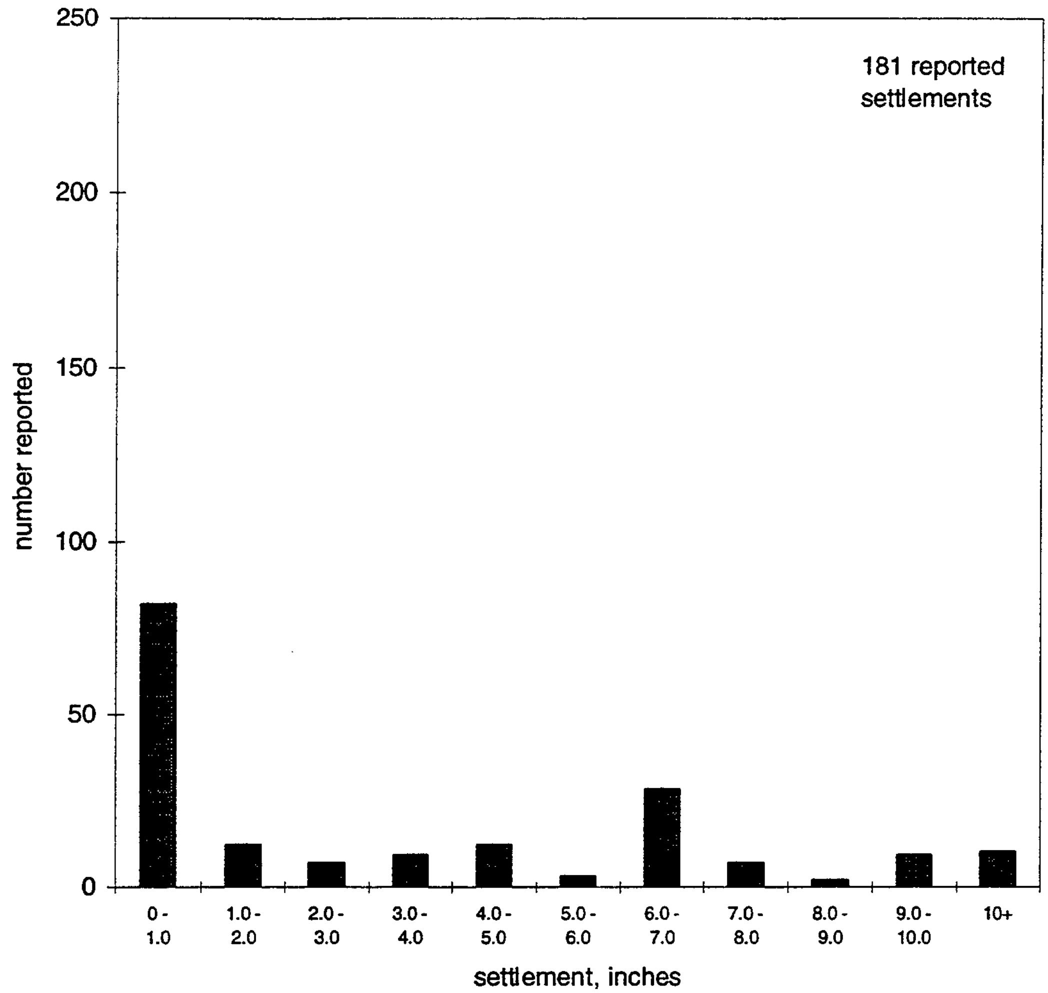


Figure 4-10 Total Settlements of Piers

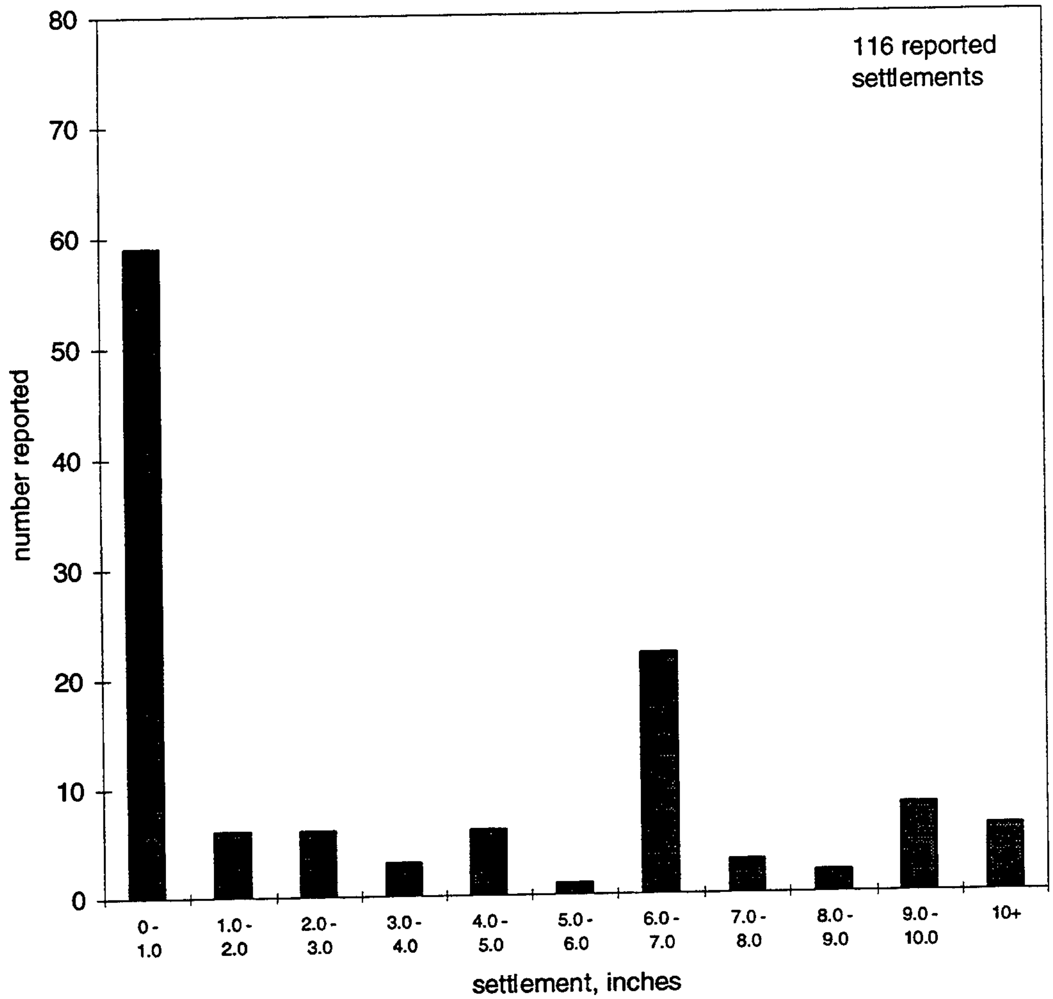


Figure 4-11 Total Settlement of Piers on Footings

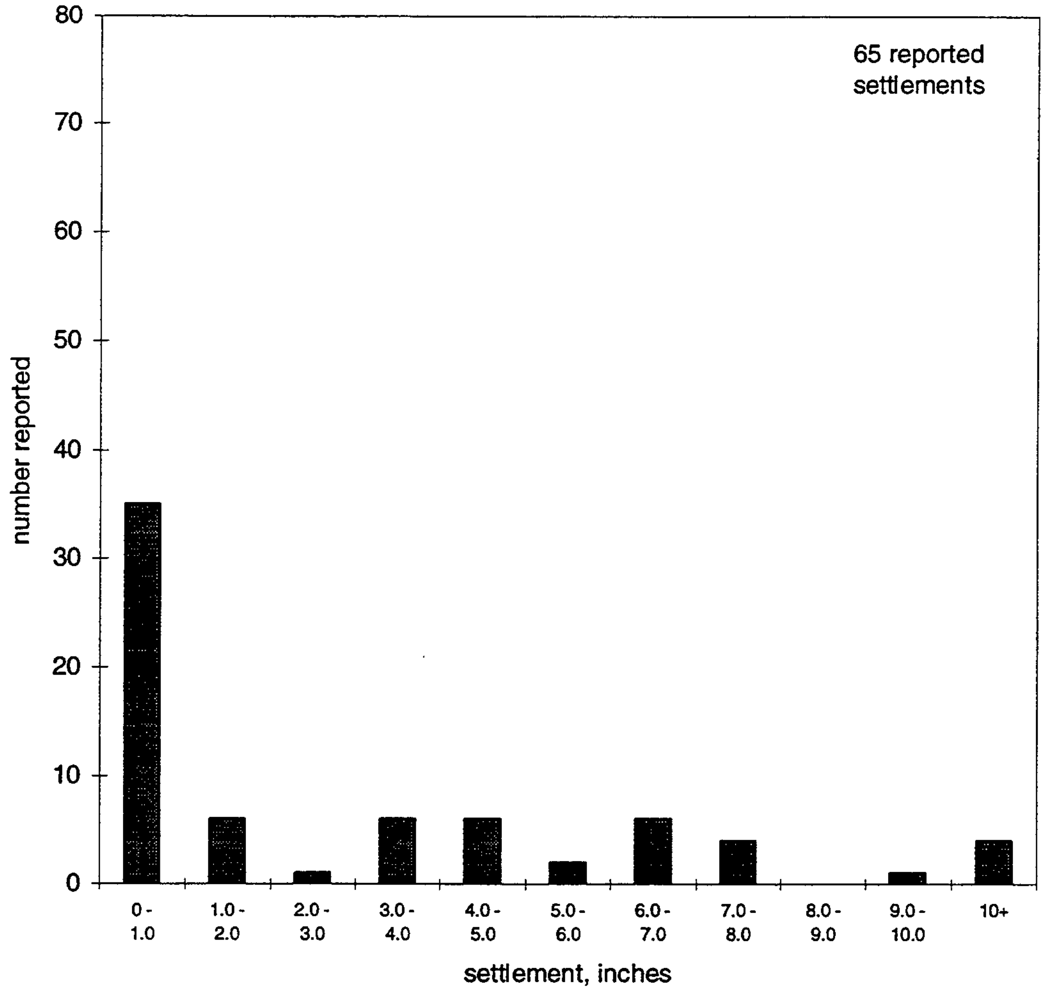


Figure 4-12 Total Settlements of Piers on Piles

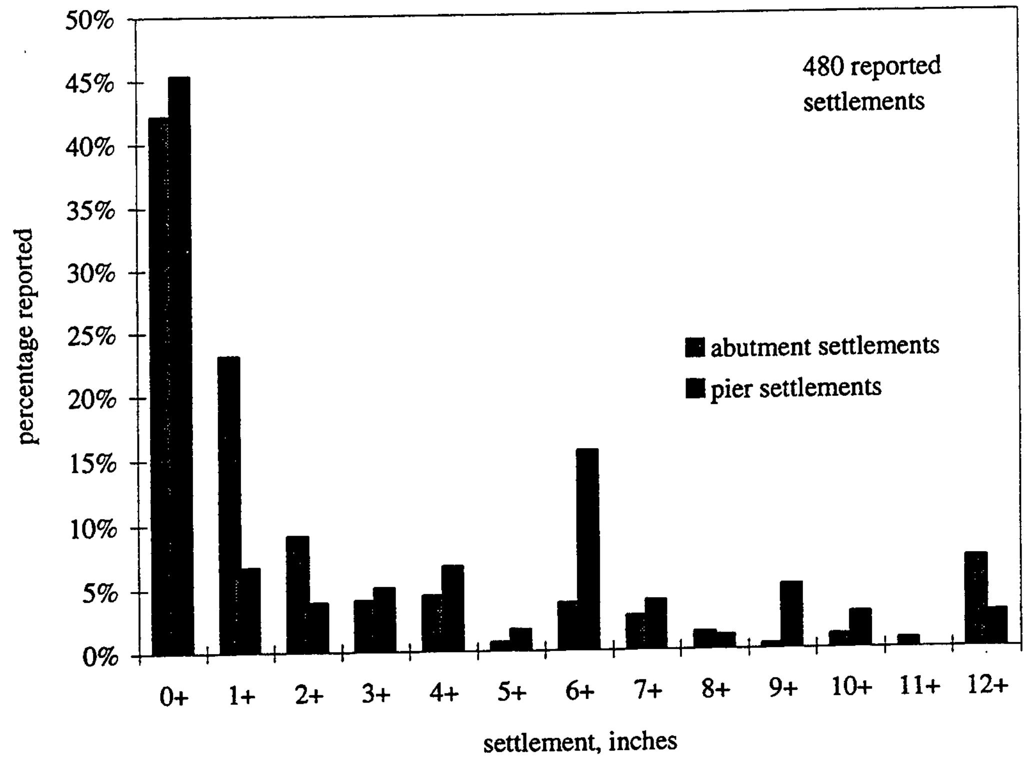


Figure 4-13 Comparison of Settlements of Abutments and Piers

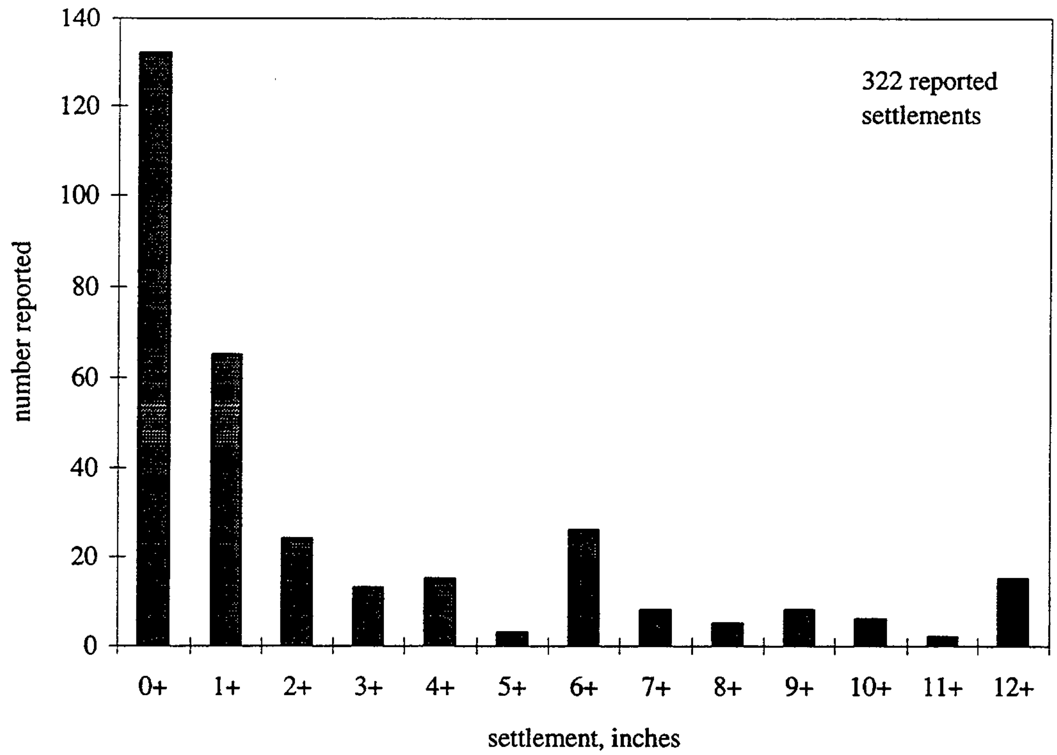


Figure 4-14 Settlements of Bridges on Shallow Foundations

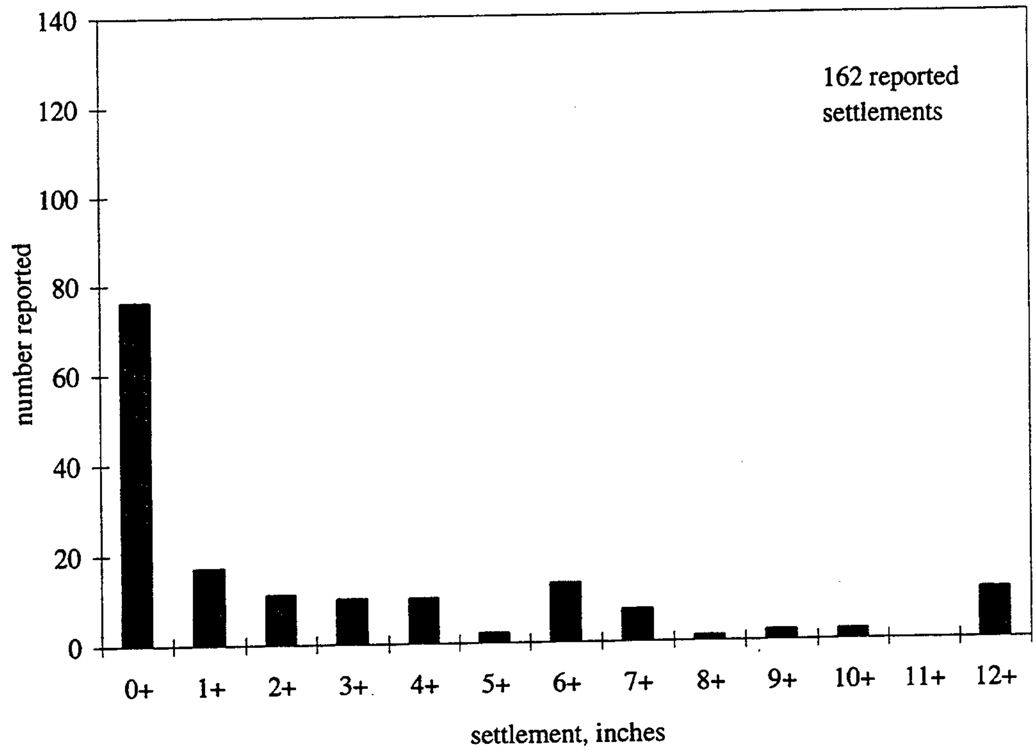


Figure 4-15 Settlements of Bridges on Deep Foundations

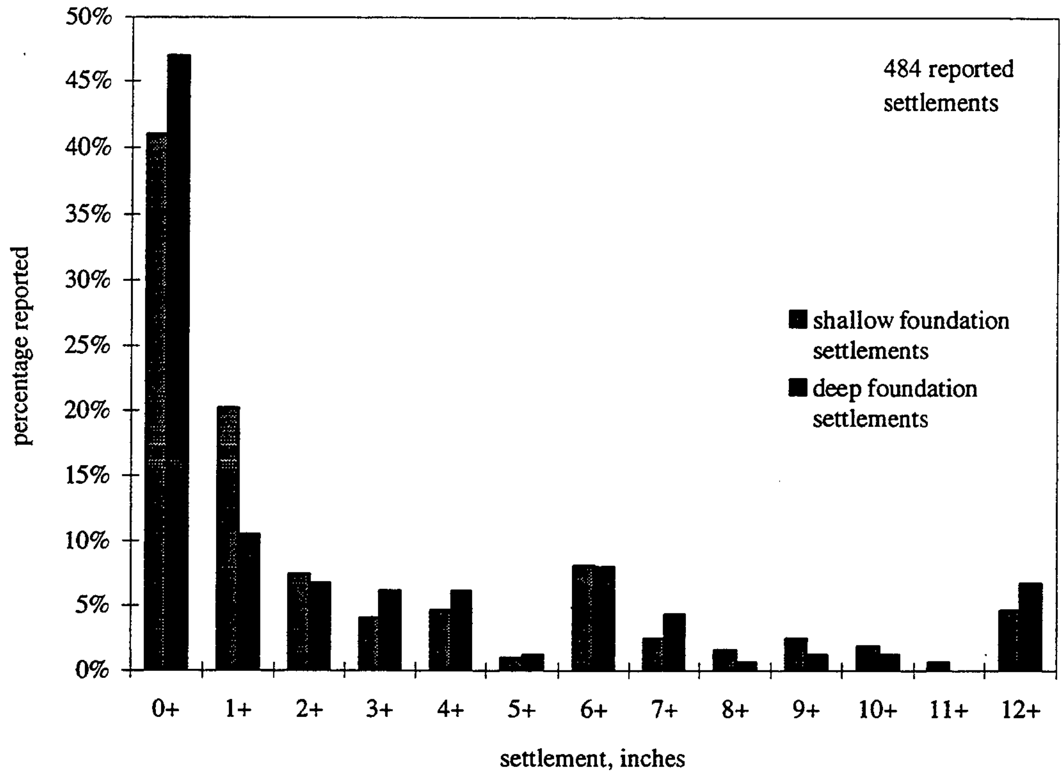


Figure 4-16 Comparison of Bridge Settlements on Shallow and on Deep Foundations

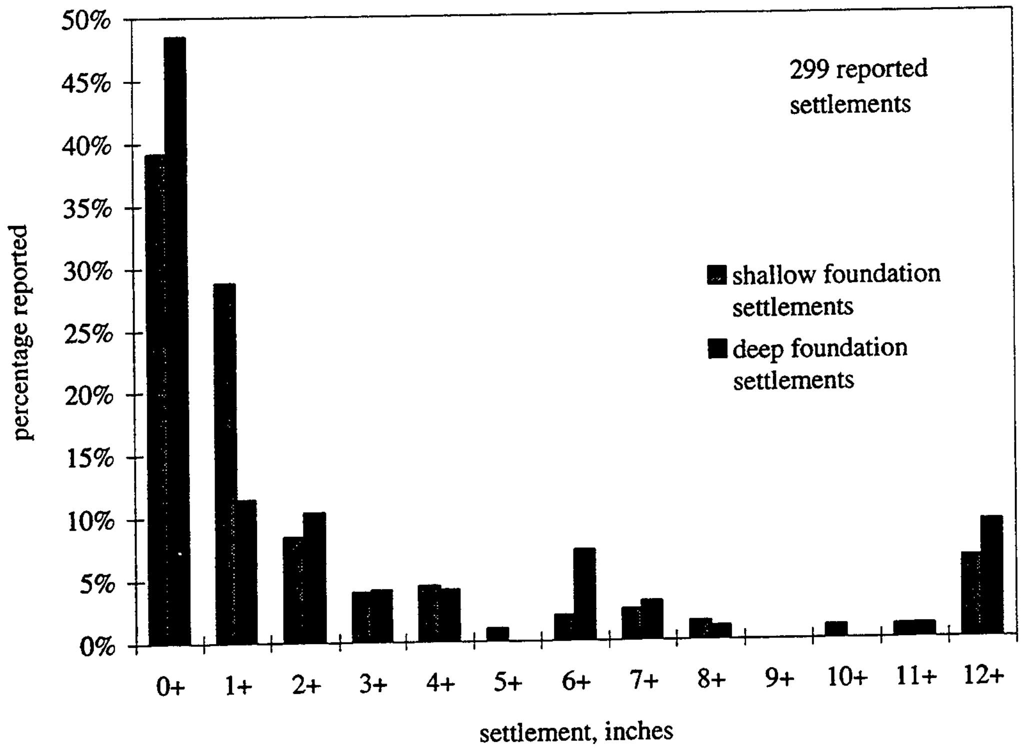


Figure 4-17 Comparison of Settlements of Abutments on Shallow and on Deep Foundations

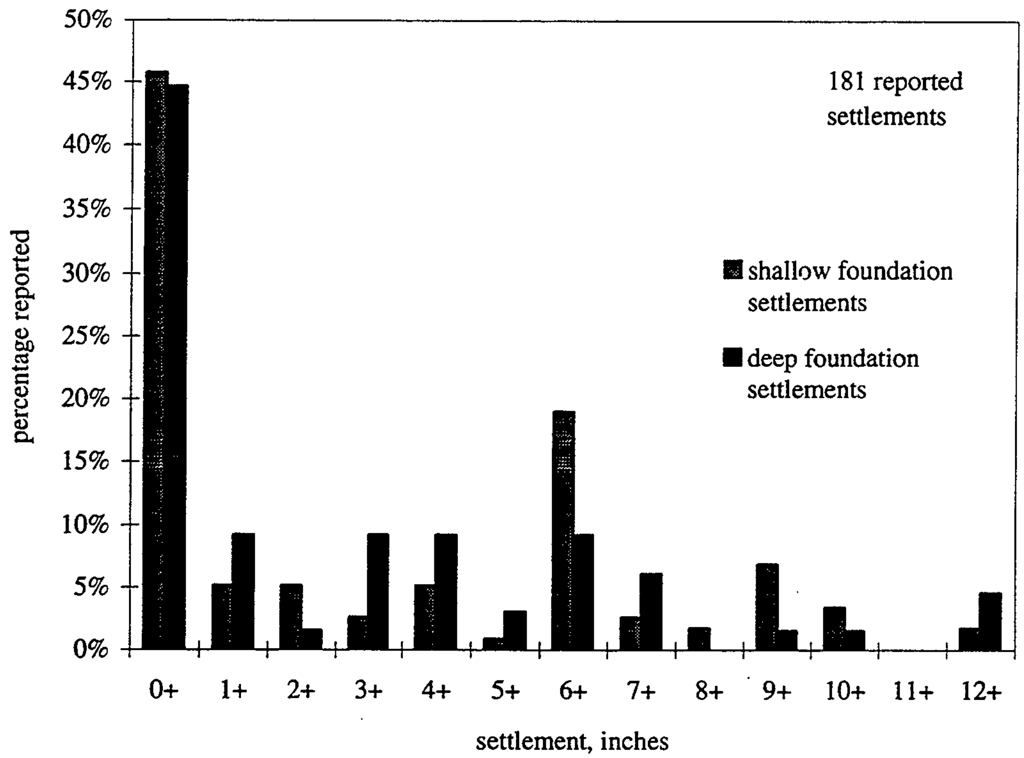


Figure 4-18 Comparison of Settlements of Piers on Shallow and on Deep Foundations

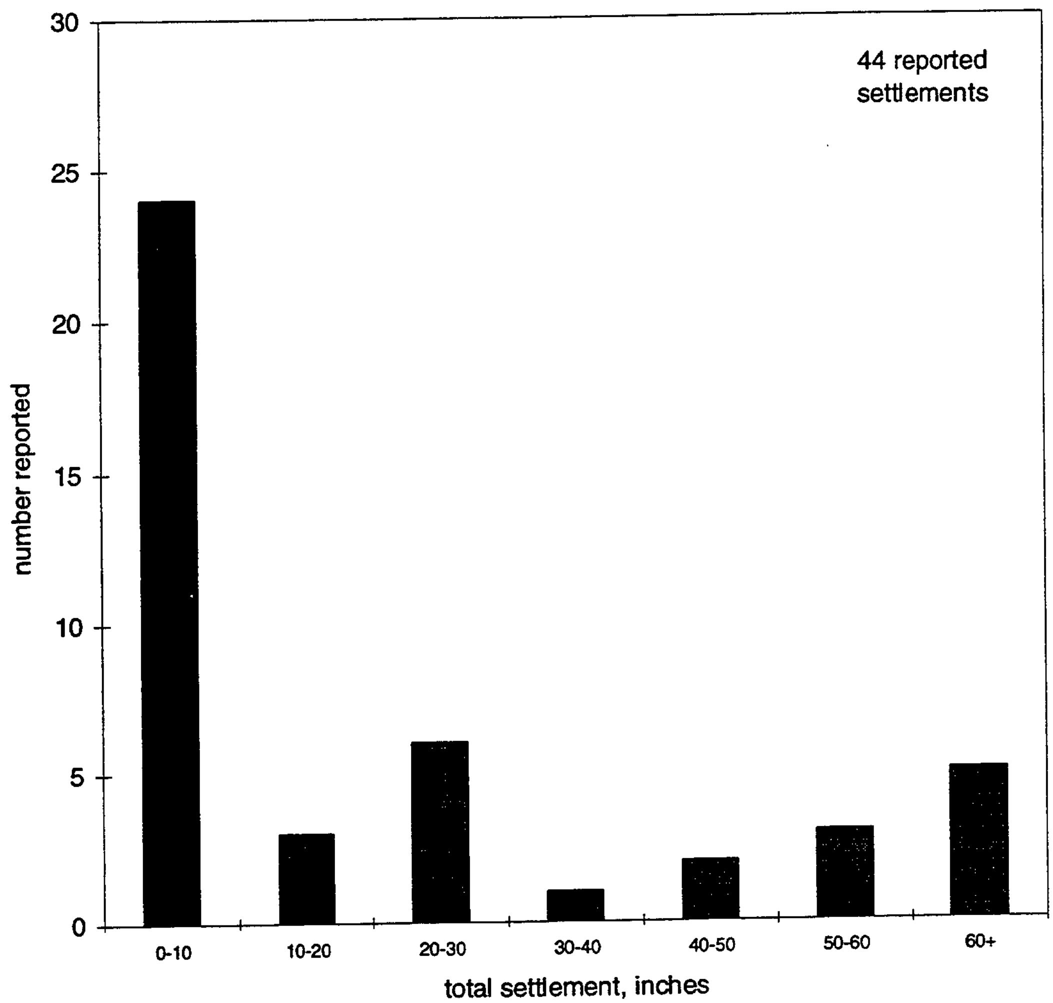


Figure 4-19 Total Settlements of Highway and Approach Embankments

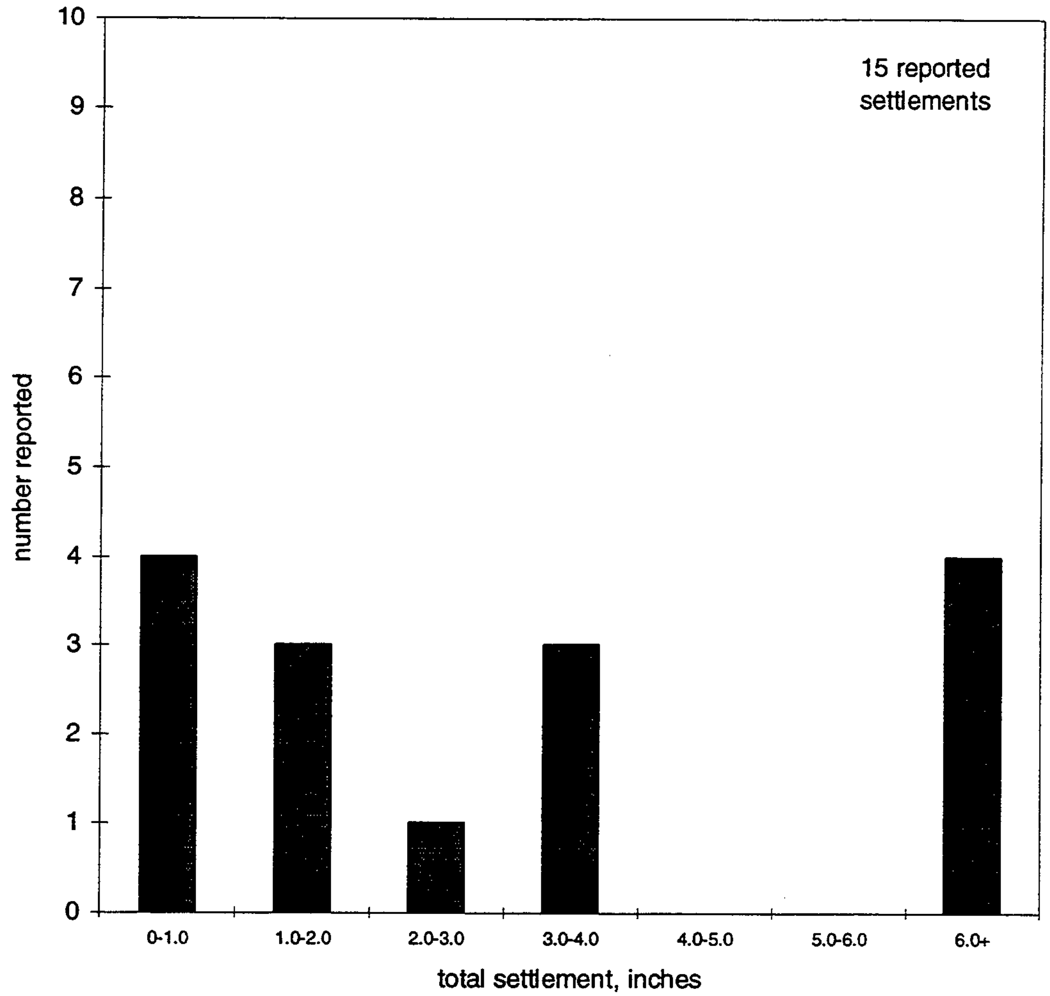


Figure 4-20 Total Settlements of Highway and Approach Embankments Less than 30 Feet Tall

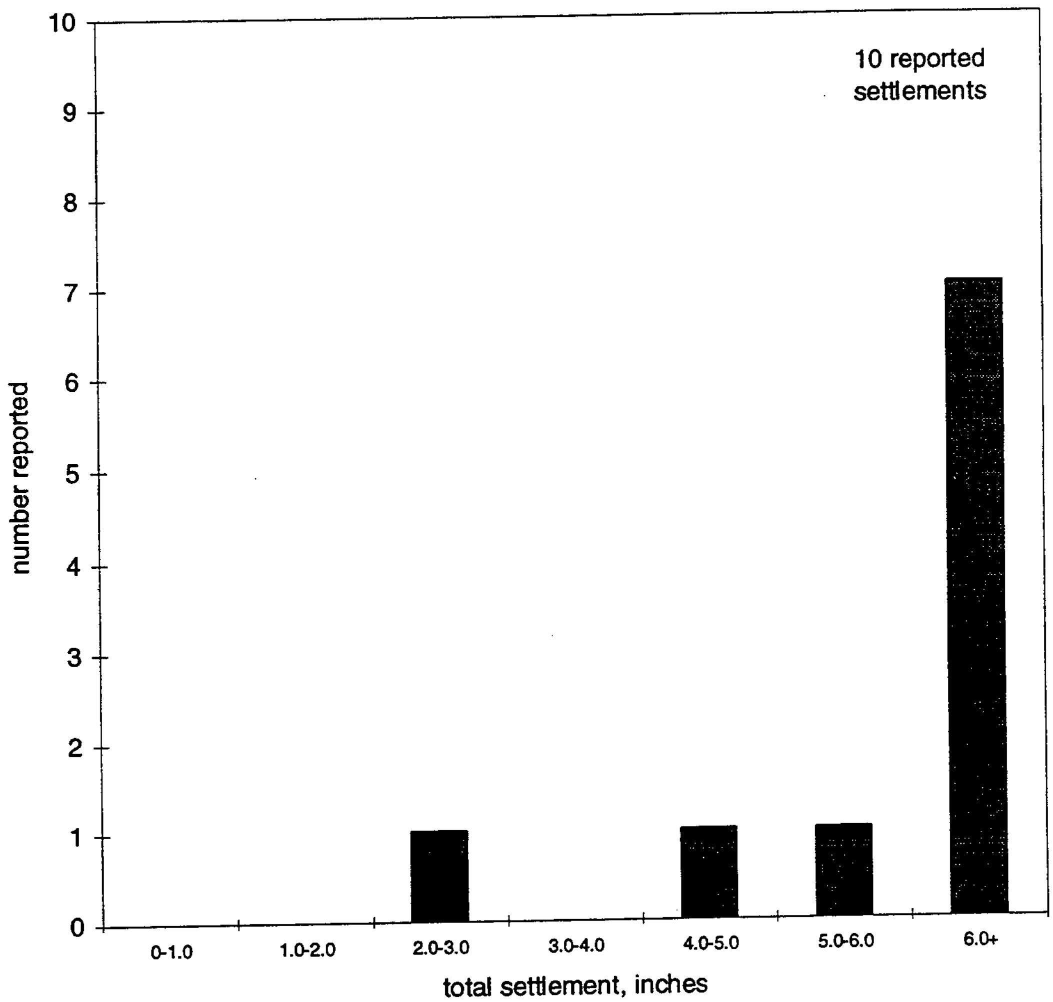


Figure 4-21 Total Settlement of Highway and Approach Embankments Taller than 30 Feet

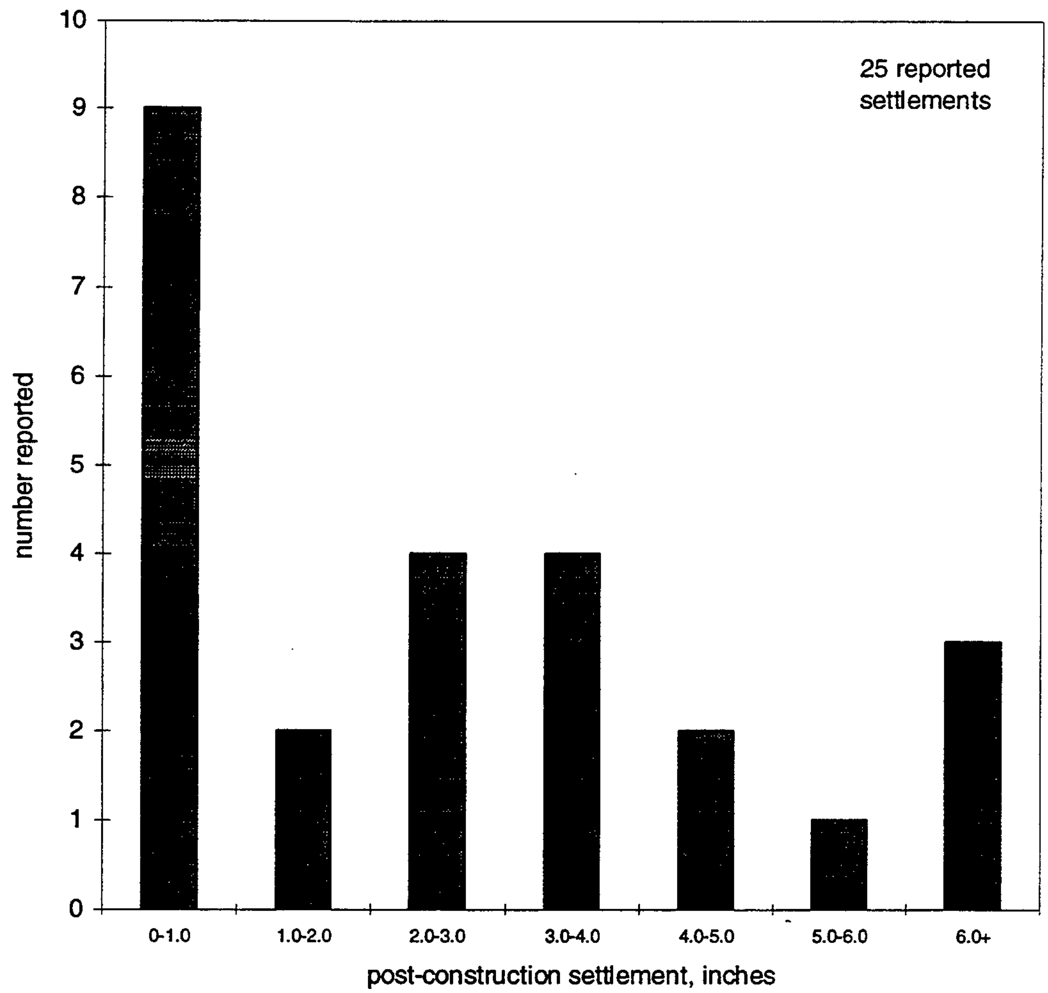


Figure 4-22 Post Construction Settlement of Highway and Approach Embankments

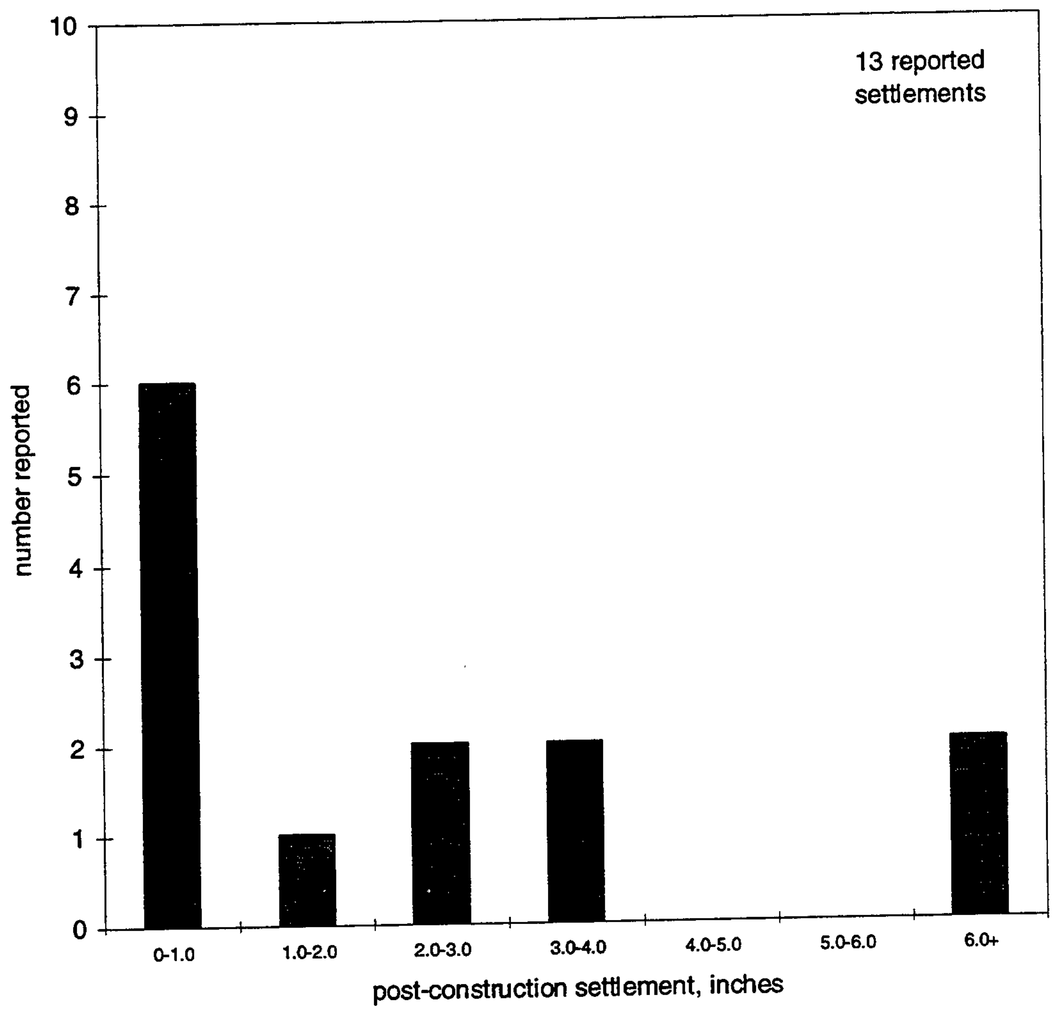


Figure 4-23 Post Construction Settlements of Highway and Approach Embankments Less than 30 Feet Tall

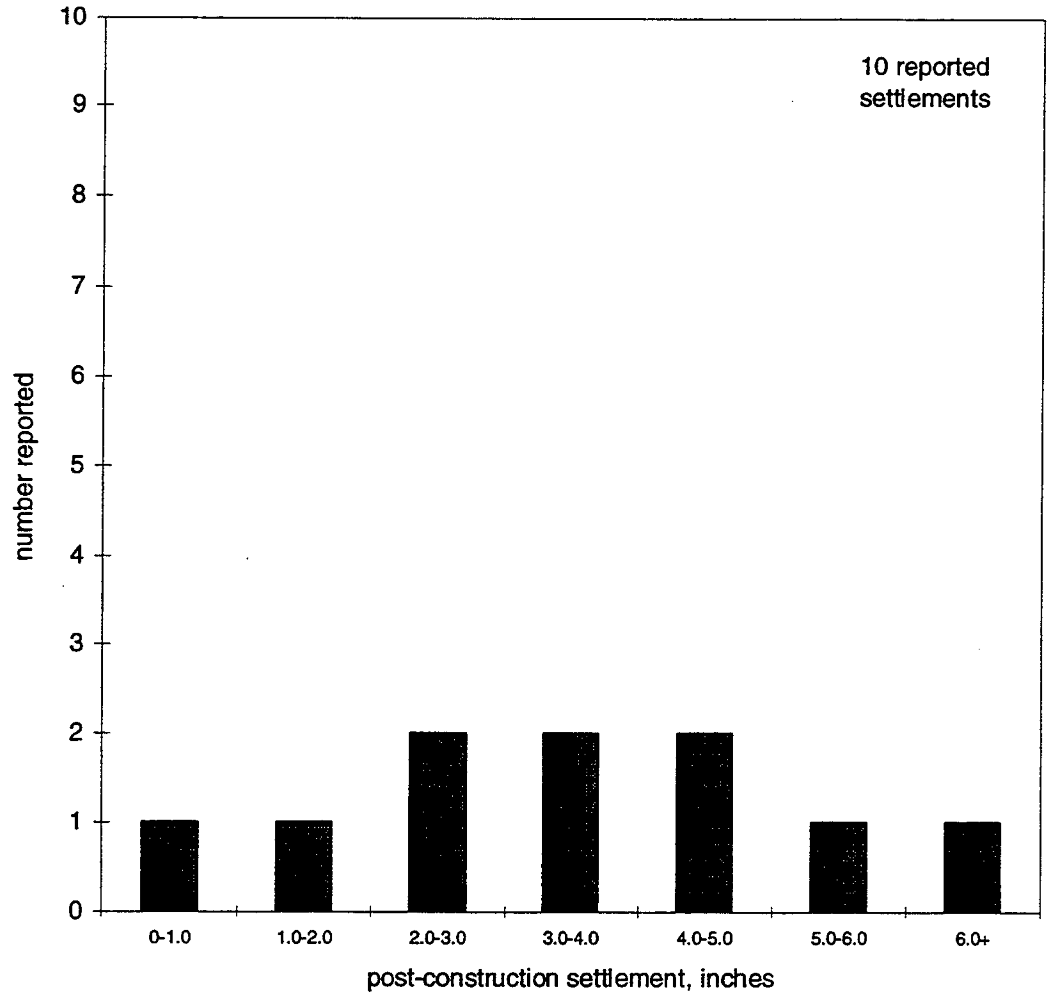


Figure 4-24 Post Construction Settlement of Embankments Taller than 30 Feet

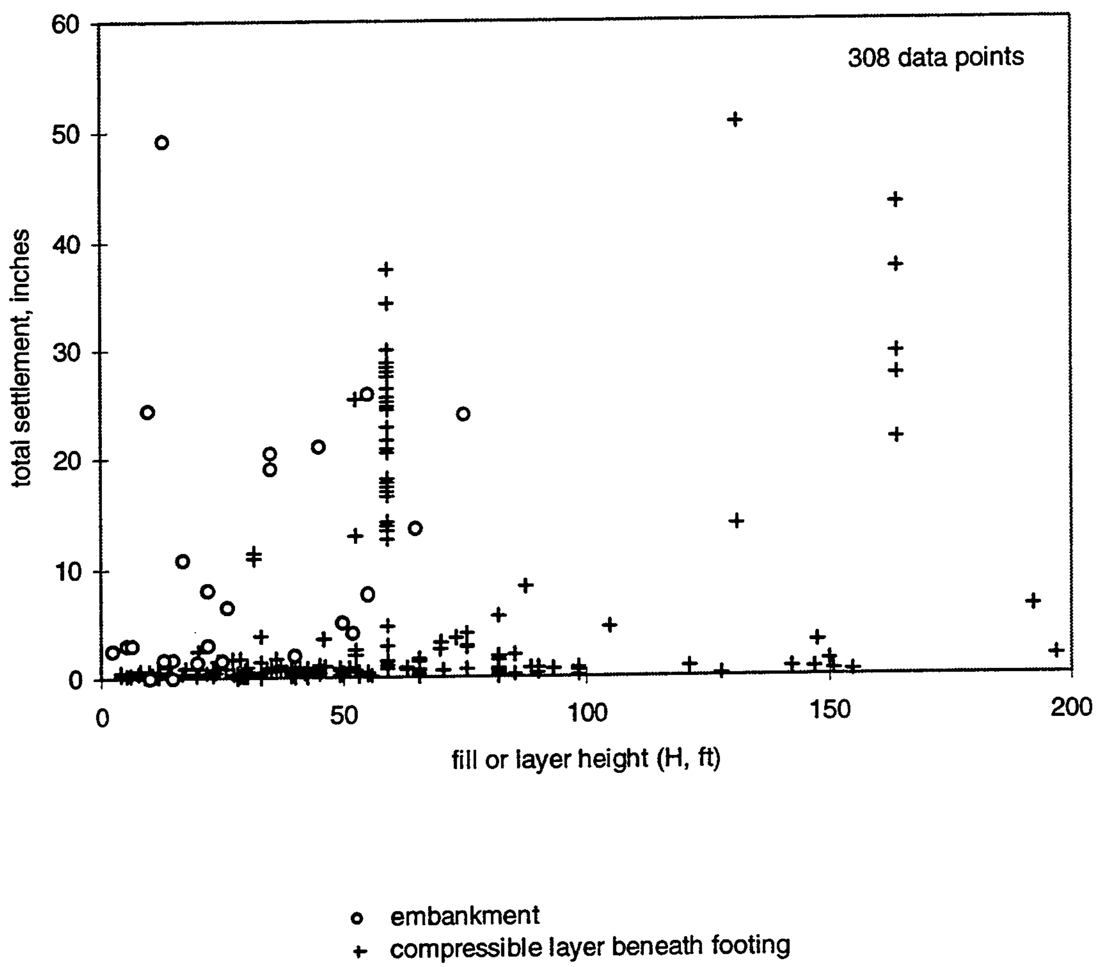


Figure 4-25 Total Settlement versus Height (for Embankments) or Layer Thickness (for Footings)

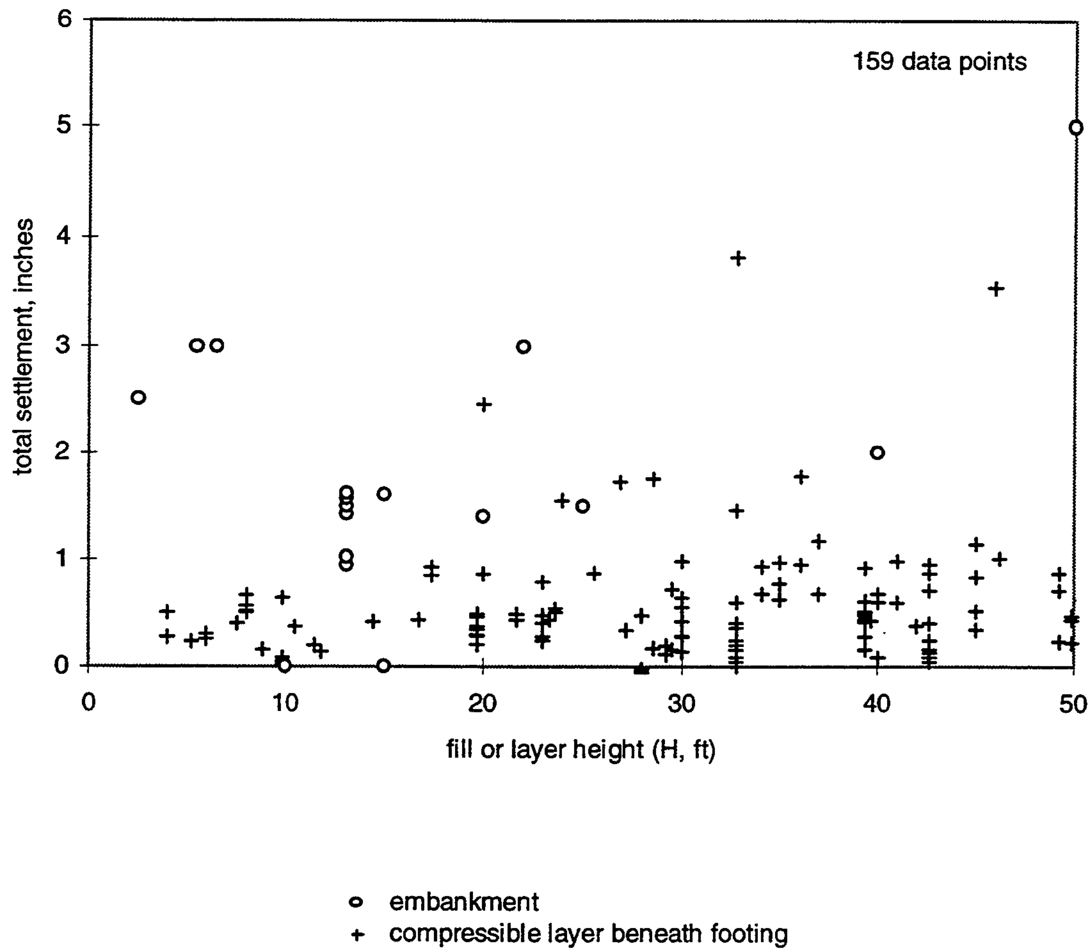
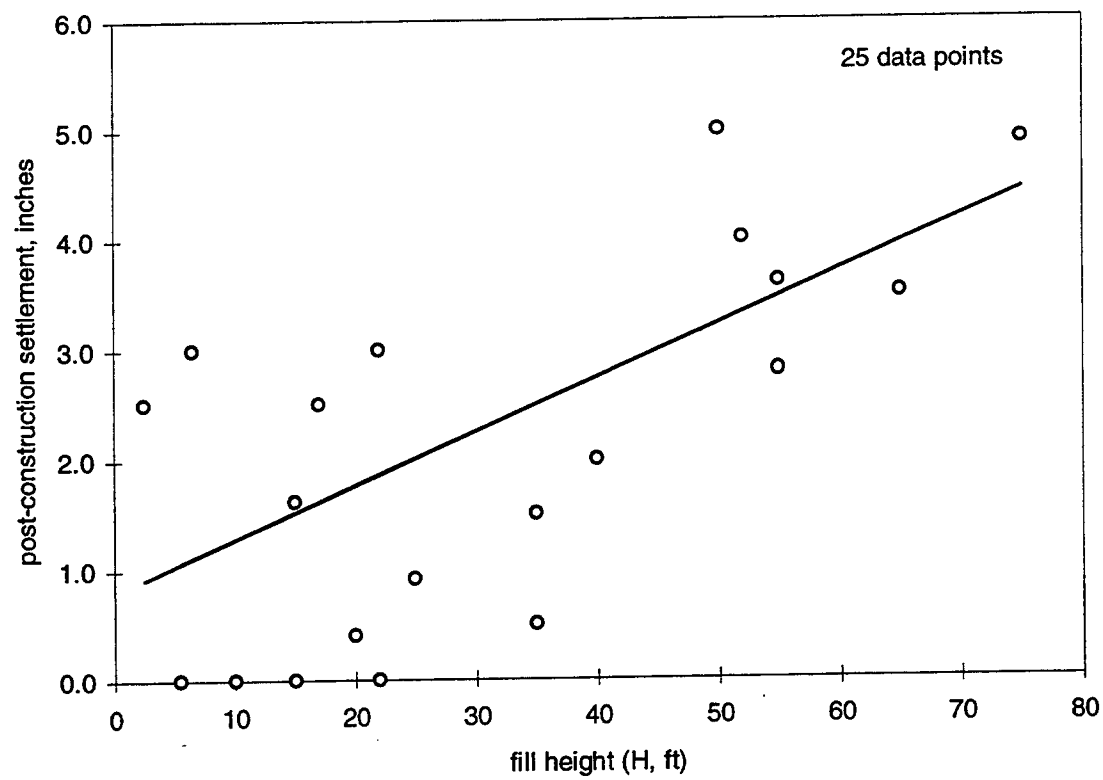
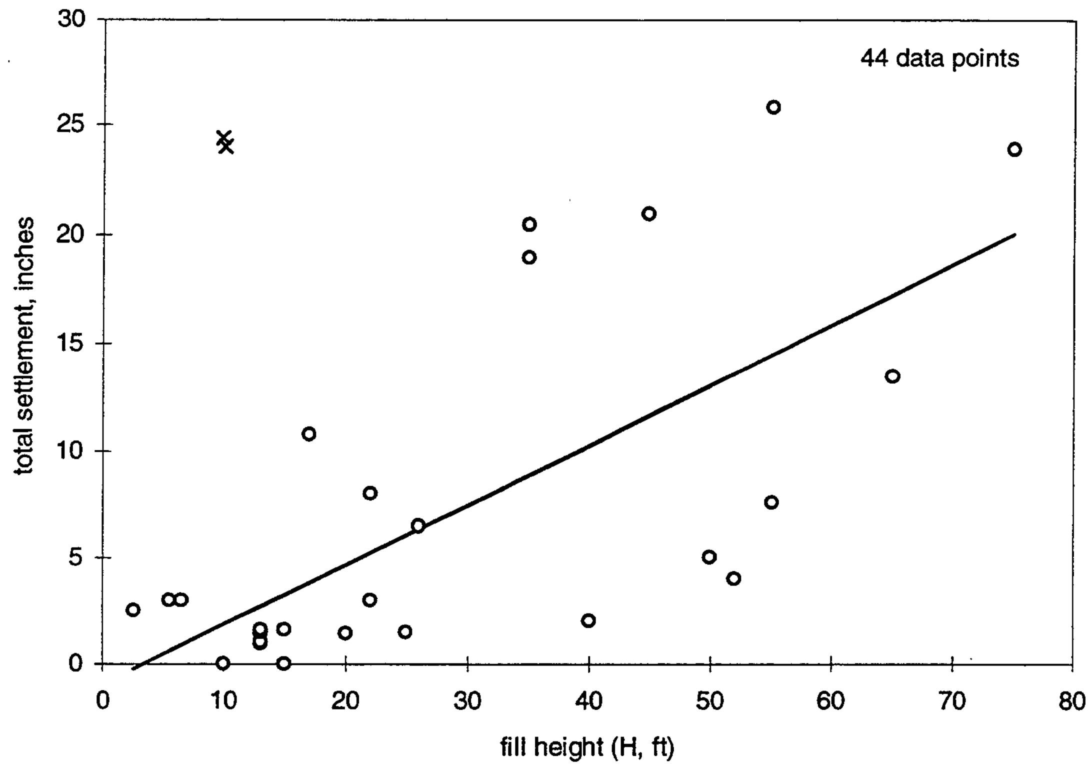


Figure 4-26 Total Settlement versus Height (for Embankments) or Layer Thickness (for Footings)



$$s_p = 0.80 + 0.05\ H$$
$$r = 0.54$$
$$\sigma(E_{rr}) = 1.56$$

Figure 4-27 Post Construction Settlement versus Fill Height



$s_T = -0.95 + 0.28\ H$

$r = 0.70$

$\sigma(E_{rr}) = 5.66$

Figure 4-28 Post Construction Settlement versus Fill Height

Table 4-9 Summary of Studies

rec	Authors	Subject	Source	Year	Type	Content	Copied	Summary
1	Ardani	Bridge approach settlement	CO DOT	1989	inspection measurement	visual descriptions		10 bridges inspected and likely causes of approach fill settlement were determined from present conditions of the approaches.
2	Yokel, F.	Proposed design criteria for shallow bridge foundations	US DOT	1990	code	analysis specifications		Reports information from other studies and presents LRFD specifications for shallow foundations.
3	Meade, B.W. Allen, D.L.	soil-bridge abutment interaction	KY DOT	1988	case literature	settlement data		Summary of other Kentucky DOT reports dealing with settlement; case studies of 6 bridges with approach settlement.
4	Kramer, S.L.	Bridge approach slab effectiveness	WA DOT	1991	literature case	slab data settlements		Summary of literature on approach slabs; site by site causes of settlement data for 9 bridges with approach slabs.
5	Laguros	Settlement of pavements behind bridge abutments	OK DOT	1989	questionnaire	responses equation		Summary of information obtained from questionnaire on 758 bridges in Oklahoma. Empirical equation for determining approach settlement from bridge age, embankment height, etc.
6	Timmerman	Mechanically stabilized embankments as bridge abutments	OH DOT	1992	questionnaire case	responses corrosion data	responses	Summary of information obtained from questionnaire regarding effectiveness of MSE walls for abutments. 3 studies of corrosion due to salt runoff.
7	James, R.W., et al	Approach roughness	TX DOT	1991	inspection measurement	bump data; pavement growth		Bump measurement at 165 bridges (0.3% of all bridges in Texas). Observational inspection. Analysis of longitudinal pavement growth.
8	Nataraja	Bridge foundations	FHWA	1982	literature	foundation data	foundation data for states	Data on bridge foundation types from Bridge Inventory statistics and other sources. Projections of future bridge construction.
9	Prellwitz	Foundation engineering handbook	Forest Service	1981		examples		Guidelines for field investigation, subsurface investigation, reduction of field data, and footing and piling analysis.
12	Barker, et al	Bridge foundations	NCHRP	1991	analysis literature	examples specifications		Design of shallow and deep foundations, retaining walls. Criteria for tolerable settlements of bridges.
15	DiMillio, A.F.	Spread footings on compacted fill	FHWA	1982	measurement	data		Assessment of safety and performance of 148 bridges on at least one spread footings in compacted approach fills. Measurement of 28.
17	Gifford, D.G. Wheeler, J.R. et al	spread footings for bridges	FHWA	1987	literature analysis monitor	data equations	bridge data settlement data method summaries	Used data from literature and field measurements to determine accuracy of six settlement techniques.

rec	Authors	Subject	Source	Year	Type	Content	Copied	Summary
20	Keene, P.	Tolerable movements of bridge foundations	TRB	1978	questionnaire	data		Discussion of 42 bridges from TRB survey of 1976. Proposed tolerable limits for bridge foundation settlement.
23	Moulton, L.K et al	Tolerable movement criteria for bridge foundations	FHWA	1986	questionnaire analysis	data	data tables	Presentation of data from 300+ bridges that moved. Example analyses for effect of settlement on concrete and steel bridges.
26	Wahls, H.	Bridge approaches	NCHRP	1990	literature	cases data	slab details	Synthesis of highway practice for bridge approaches. Summary of available literature.
28	Walkinshaw, J.L.	Tolerable settlements for bridges	TRB	1978	questionnaire	data		35 bridges from TRB survey of 1976 reviewed.
29	Hopkins, T.C.	Long-term movements of bridge approaches	KY DOT	1985	case monitor	data		Six elaborate case studies of bridge approaches monitored for up to 20 years. Empirical consolidation formula developed.
30	Allen, D.L.	Movements and forces of a bridge approach	KY DOT	1986	case	data		Detailed case study of a bridge approach. Much of the instrumentation for measuring was destroyed during construction.
31	Hopkins, T.C.	Embankments on clay foundations	KY DOT	1985	literature	specifications	conclusions	Explanation and specifications for Kentucky embankments. 90+ documented slope failures; so specifics for bridges.
32	Stewart, C.F.	Highway structure approaches	Caltrans	1985	inspection measurement	data	standard slab specs	820 California bridge approaches inspected with bump-measuring vehicle. Repair correlated with various characteristics of the approach.
33	Holtz, R.D.	Preloading by vacuum	ASCE	1970	literature	procedures		Description and explanation of preloading soil with vacuum action.
34	Johnson	Precompression	ASCE	1970	literature	procedures		Necessary information for preconsolidation of soil.
35	Mitchell	In-place treatment of foundation soils	ASCE	1970	literature	procedures		Compilation of various methods for improving soils.
37	Emmanuel	Abutment movement	TRB	1978	literature			Short discussion of the reasons for abutment movement and the problems of expansion joints.
38	Meade Allen	Stone columns for embankment foundations	KY DOT	1985	case monitored	design data	specifications and notes for design; settlement plots	Detailed case study of a stone column reinforced foundation for an embankment. Includes alternatives, design, installation and performance.
39	Stewart, C.F.	Bridge deck overlays	Caltrans	1989	inspection	data		Cracking of reinforced concrete deck overlays correlated with construction method, traffic, original deck, and weather.
40	Wolde-Tinsae, A.M. Klinger, J.E.	Integral abutments	MD DOT U of MD	1987	questionnaire	data		Compiled data on integral abutment use throughout the US and the world. Problems related to expansion joints.
41	McNulty, E.G.	Unstable bridge approach	KY DOT	1979	case	data		Documented replacement of failed embankment supporting bridge approach using lightweight fill and berms.

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42	Scholen, D.E.	Non-standard stabilizers	Forest Service	1992	case measurement	data		Performance of stabilizers for soils and aggregates in 160 miles of unpaved roads; performance and lab data
43	Christopher, B.R., et al	Reinforced soil structures	FHWA	1990	literature	procedures examples	details for designs	Guidelines and design evaluation for reinforced structures from reviews of lab tests, centrifuge tests, FEM, and full scale field tests.
44	Elias, V. Juran, I.	Soil nailing for highway slopes and excavations	FHWA	1990	literature	procedures details	design charts soil friction charts	Summarization of state-of-the-art in soil nailing. Details on materials and installation procedures.
45	Nunan, T.A. Humphrey, D.H.	Gravel stabilization	U of ME FHWA	1990	tests	data		Small-scale tests of aggregate stabilizers in unpaved roads.
46	Berg, R.R.	MSE slopes	FHWA	1993	literature guidelines			Guidelines on design and specifications of geosynthetic mechanically stabilized earth slopes on firm foundations.
47	-	Yarn reinforced soil	Caltrans	1991	lab	testing data		Triaxial testing of 6" x 14" samples with 0.2% to 1% yarn added. More time spent on preparation of sample than coherent results.
48	Welsh, J.P.	In-situ ground modification	FHWA	1992	literature	examples costs	references	Descriptions and notable ground modification projects. Vibroflotation, stone columns, dynamic compaction, grouting.
49	Bruce, D.	Ground treatment and in-situ reinforcement	FHWA	1992	literature case	procedures		Descriptions and procedures for grouting and soil cementing; case study of pinpiles.
50	Christopher, B.R. Holtz, R.D.	Current research on geosynthetics	FHWA	1992	literature	list		List of present studies in geosynthetics, includes notable studies.
51	Koerner, R.M. Wilson Fahmy, R.	Polymeric geogrid reinforcement of embankments over weak soils	Geotextile Research Inst	1992	literature	examples details		Descriptions of types of geogrids; standard testing procedures, installation; design; examples [part 3 of 4 on geogrids]
52	Holtz, R.D.	Treatment of problem foundations for highway embankments	NCHRP	1989	synthesis	descriptions examples		Design methods for nearly all types of foundation. Includes data from survey of 42 states.
53	Manning, D.G.	Effect of traffic induced vibrations on bridge deck repairs	NCHRP		synthesis			Not really applicable to our interests. Vibration effects on newly placed bridge overlays.
54	Reckard, M.	Cost effectiveness of geotextiles	AK DOT	1991	inspection measurement	procedure		Alaskan road embankments reinforced with geotextiles (4 million sq. yds.) inspected for effectiveness in preventing cracking of pavement and differential settlement of pavement.
55	Fowler, J.	Geotextile reinforced embankments on soft foundations	Army Corps	1989	case	procedures examples		Army Corps of Engineer's techniques for fabric-reinforced embankments (dikes). Case studies of test sections and design methods.

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56	Huang, W.H.	Bottom ash in embankments, subgrades, and subbases	Purdue FHWA	1990	lab	testing procedures		Testing of 11 bottom ashes for use in highway projects. Seems more as a way of disposal rather than improvement.
57	Koerner, R.M. Wayne, M.H.	Geotextile specifications	Drexel FHWA	1989	questionnaire	summary glossary	glossary	Reviews specifications on geotextiles from 46 states. Interesting and poor specifications pointed out; sample model specs; design details
58	Juran, I. et al	Soil improvement/reinforcement techniques for highway embankments	LSU FHWA	1989	cases literature	design methods	name-brand geotextiles; references	Evaluation of short- and long-term performance of improvement techniques; advantages and disadvantages; details and best situations for use of techniques
59	Elias, V.	Durability/corrosion of soil reinforced structures	FHWA	1990	literature	summary equipment		Testing procedures and equipment for determining corrosion rates in soils; characteristics of soils which determine corrosion
60	various (split into individual refs)		ASCE	1989				
61	Holtz, R.D.	Reinforcement of earth slopes and embankments	NCHRP	1991	literature	descriptions		Large compilation of reinforcement techniques for earth slopes and embankments; internal and external stability checks for walls
62		Voids beneath approach slabs	CO DOT	1989	questionnaire	details specifications	details plans	Summarization of returned information dealing with voids beneath approach slabs; includes slab and reinforcement details.
63	Basma, A.A. Tuncer, E.R.	Lime treatment for expansive clays	TRB	1991	lab	testing results	compression plots	Expansive clays from Jordan treated with lime and subjected to various lab tests for compressibility, permeability, grain size, etc.
64	Tuncer, E.R. Basma, A.A.	Lime treatment for a cohesive soil	TRB	1991	lab	testing results		Unconfined compression tests and others performed on soil treated with different amounts of lime (3, 6, and 9%).
65	Fletcher, C.S. Humphries, W.K.	Polypropylene fiber reinforced soil	TRB	1991	literature lab	lit summaries test results		Summary of various literature dealing with fiber reinforced soils; California Bearing Ratio tests on fiber-reinforced soils.
66	Blight, G.E. Dane, M.S.W.	Deterioration of Reinforced Earth wall complex	Geotechnique	1989	case	data details		Details of soil and construction characteristics which led to a complete deterioration of a Reinforced Earth wall complex.
67	Smith, A.C.	Deterioration of Reinforced Earth wall complex	Geotechnique	1989	case	data		Rebuttal to rec66.
68	Hoffman, G.L. Turgeon, R.	Long-term in-situ properties of geotextiles	PennDOT	1983	inspection lab	data		Comparison of 12 geotextiles exhumed from road subbases after six years with their original properties.
69	Chou, L. (ed.)	Lime stabilization	TRB	1987	literature	details		Summary of improvements from lime addition to soil; construction procedures.

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70	Frondistou-Yannas, S.	Corrosion of reinforced soil retaining structures	FHWA	1985	literature cases	data		Characteristics of soils which cause corrosion of metal reinforcements; field studies of corrosion of four reinforced walls.
71	Barksdale, R.D.	Sand compaction piles	Army Corps	1987	literature	design theory details		Design theory of sand compaction piles which are popular in Japan for reclamation of land from the sea; installation procedures.
72	Christopher, B.R. Holtz, R.D.	Geotextile engineering manual	FHWA	1985	manual	test methods specifications		Manual meant for FHWA training course in geotextiles used to evaluate each application; cost and performance reviewed.
73	Vokas, C.A. Stoll, R.D.	Reinforced elastic layer systems	TRB	1987	analysis	FEM		FEM continuum model for layer reinforcement when nonlinear and inelastic effects are small.
74	Duncan, J.M., et al	Reinforced embankment	TRB	1987	case analysis	details FEM	settlement plot	Design and performance of a failed embankment later improved with steel mesh to construct it to its full height; FEM design.
75	Bonaparte, R. Christopher, B.R.	Reinforced embankments over weak foundations	TRB	1987	analysis	FEM	movement plots references	FEM used to determine failure mechanisms (sliding, bearing, slip surface), strain, settlement; includes successful design and construction methods.
76	Sakti, J.P. Das, B.M.	Clay reinforced with geotextile layers	TRB	1987	lab	model tests data	settlement plots lab sequence	Model tests of a strip foundation on saturated soft clay with geotextiles; depth, number, and spacing of layers varied.
77	Sarkar, S.K. Castelli, R.J.	Wick drains in clay	TRB	1988	case monitored	data procedures	settlement plots	Performance of Alidrain and Geodrain in soft clay at wharf site; settlement monitored over 1 year, compared to estimated consolidation.
78	Kyfor, Z. et al	Prefabricated vertical drains beneath an embankment	TRB	1988	case monitored analysis	data	consolidation formula; settlement plot	Construction of highway embankment with vertical drains to increase rate of consolidation; settlement monitored for different drain spacings and compared to estimated.
79	Saye, S.R., et al	Wick drains	TRB	1988	case monitored	data	embankment settlement plots	Installation of wick drains in 2 mile long embankment; monitored with settlement gages and piezometers for 3 years.
80	Murray, R.P.	Two-dimensional pile system	TRB	1980	case	description		Piles battered in opposite direction to form slope-supporting grid; high cost and limited applications.
81	Lamb, S.E.	Horizontal drains	TRB	1980	case	description data		Water table dropped and stability of embankment increased with horizontal drains.
82	Leonards, G.A., et al	Dynamic compaction of granular soils	TRB	1980	case	procedure data		Granular material from nearby gravel pit densified with 5.9 metric ton weight dropped repeatedly from 12m.
83	Haliburton, T.A., et al	Fabric-reinforced embankment	TRB	1980	case measurement	construction data		Test section of 5000 ft embankment (800 ft) constructed on very soft foundation soils with geotextile interface; measured after 1 year.
84	Shields, D.H., et al	Bridge abutments in approach fills	TRB	1980	case	instrumentation		Midpass structure built in 1978 and instrumented to measure movement of abutments in approach fills; no results.

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85	Bonaparte, R., et al	Survivability and durability of a nonwoven geotextile	ASCE	1988	case lab	testing data		Strength testing of two polypropylene, continuous filament, thermally bonded, nonwoven geotextiles uncovered from road subbases.
86	Das, B.M.	Shallow foundation on sand underlain by soft clay with geotextile interface	ASCE	1988	lab	data		Model tests of geotextile layer of various lengths at various depths at interface of clay and sand fill.
87	Dawson, A. Lee, R.	Full-scale foundation trials on grid reinforced clay	ASCE	1988	case	details data		4 full-scale strip footing tests of alluvial clay site with different compaction and geotextile configurations monitored for 4+ years.
88	Hoover, T.P.	Wick drains		1989	case	data		Monitored performance of two types of wick drains in the approach embankments of a bridge.
89	Hussin, J.D. Ali, S.	Soil improvement of one site by three methods		1989	case monitor	data		Compared the effectiveness of vibroreplacement stone columns, dynamic deep compaction, and compaction grouting for improvement of one site.
90	Peggs, I.D., et al	Durability of polypropylene geotextile in an unpaved road	TRB		case lab	data		Two samples of geotextile exhumed from base of unpaved roads and tested for remaining strength [see 85].
91	Bandimere, S.W.	Slabjacking	TRB	1986	case	data		Description of slabjacking methods used in Wyoming.
92	Blacklock, J.R. Wright, P.J.	Injection stabilization for embankments	TRB U of Ark	1986	case	data		3 sites where injection of lime grout was used to attempt to stabilize failed embankments.
93	Briaud, J.L. et al	Pressuremeter for predicting settlements of shallow foundations	TRB		measurement analysis	data equations		Method for predicting settlements of shallow foundations based on Menard's method using pressuremeter; results of 17 footings on stiff clays.
94	Byrne, P.F. Lacey, H.S.	Replacement of Brighton Ave Bridge	TRB		case	data		Old bridge with failing secondary members replaced; fill grouted.
95	Mitchell, J.K. Huber, T.R.	Stone columns			monitor analysis	data references		Monitored project with vibroreplacement stone columns; settlement analysis by different methods compared to actual after 6 years.
96	Bernadi, R. et al	Settlement of shallow foundations	Geot Engng Congress	1991	literature	references equations		Outline for selection of parameters for elastic settlement analysis.
97	Tan, C.K. Duncan, J.M.	Settlement of footings on sand	Geot Engng Congress	1991	literature analysis	data references		Comparison of 12 settlement prediction methods for spread footings on sand with 76 footings based on SPT data.
98	Ameen, S.F.	Settlement of inclined loaded footings on sand	Asian CSMFE	1991	model analysis	data		Small-scale model test results compared to predicted settlements from various methods.
99	Okamoto, T.	Settlements of large mat foundations	Asian CSMFE	1991	literature analysis	data equations		Expressions developed to determine maximum and differential settlements from 35 sets of data from literature.
100	Troung, H.V.P.	Movement of abutments due to cyclic loading	Asian CSMFE	1991	model	data		Small-scale load tests on model abutments; incomplete without other Troung references.

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101	Sohn, J. et al	Geogrid reinforced soil	Asian CSMFE	1991	model FEM	data		Model test of geogrid reinforced soil to reduce differential settlements beneath pipe; lab compared to FEM results.
102	George, E.A.J. Abam, T.K.S.	Settlement prediction	Asian CSMFE	1991	analysis	data equations		Settlement predictions made for a tank on sand; no comparison to actual because it hadn't been built.
104	Marangos, Ch.	Settlement prediction from pressuremeter and oedometer	ICSMFE	1989	model analysis	data equations		outline for obtaining two types of deformation moduli from pressuremeter; no comparison with actual settlement.
105	Vargas, M.	Long-term settlements of tall buildings on sand	ICSMFE	1989	analysis	data		Explanation of secondary compression of sand which occurred after publication of an earlier paper which described the elastic settlement.
106	Bartolomey, A.A., et al	Prediction of pile settlement	ICSMFE	1989	analysis	nomographs		Empirical and theoretical settlement prediction methods presented with no comparisons.
107	Bolle, A., et al	Probabilistic approach to settlement of bridge	ICSMFE	1989	analysis	data		Outline of procedure to predict maximum settlement as well as tilt and rotation of a footing probabilistically.
108	Yamashita, K., et al	Settlement of large diameter bored pile groups	ICSMFE	1989	analysis	equations	all	Correction factor for Mindlin's method which accounts for non-linearity of soil; two comparisons between predicted and observed.
109	Tsuchida, T.	Differential settlements	ICSMFE	1989	case monitor	data	autocorrelation plot settlement plots	Total and differential settlements monitored at Tokyo International Airport; settlement predicted form consolidation; autocorrelation of settlements.
110	Bergado, D.T., et al	Inverse analysis of settlement	ICSMFE	1989	case monitor	data equations	equations settlement plot	Five methods of settlement prediction compared to measured settlement along Bangna-Bangpakong Highway (soft clay).
111	El Ghamarawy, M.K.	Observed and predicted settlements	ICSMFE	1989	case monitored FEM			Settlement of footing on 6m silty clay monitored during and after construction; compared to various predictions including FEM.
112	El-Sohby, M.A. Ellebuddy, A.M.	Grouting to control settlement of bored piles	Asian CSMFE	1991	case monitored	data		52 bored piles were regrouted after initial grouting to reduce settlement.
113	Kaniraj, S.R.	Settlement ratio of pile foundations in sand	Soil & Fnds	1993	analysis	equations	equations predicted/measured plot	Development of semi-empirical equation for determining settlement ratio of pile group to pile in sand; comparisons with observed.
114	Papadopoulos, B.P.	Settlement of shallow foundations on cohesionless soils	ASCE	1992	analysis literature	data equations	equations settlement data	Settlement prediction based on elasticity with a soil modulus determined from lab tests; compared with other methods for 61 sets of data.
115	Monley, G.J. Wu, J.T.H.	Geogrid for approach fills	ASCE	1993	analysis FEM		references	
116	Burland, J.B. Burbridge, M.C.	Settlement of foundations on sand and gravel	Instn Civil Engrs	1985	analysis literature	equation	compressibility plots settlement data	Over 200 sets of footing data used to empirically determine a relationship between a soil compressibility coefficient and SPT data; used in settlement equation.

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117	Vanmarcke, E.H. Fuleihan, N.F.	Probabilistic prediction of settlements	Stats & Prob Conf	1975	analysis case	equations	soil data plots settlement distribution	Variability in soils properties used to show variability in 1-D consolidation settlement prediction method.
118	Bergado, D.T., et al	Inverse analysis of geotechnical parameters of soft clay	ASCE	1992	case analysis	data	equations soils data settlement plots	Comparison between predicted and observed settlements, 3 methods used; two sections of improved clay one with drains and one with stone columns.
119	Degroot, J.B. Baecher, G.B.	Estimating autocovariance of in-situ soil properties	ASCE	1993	analysis	equations	references	Presents method and equations for determining autocovariance of soil properties from a discrete number of samples.
120	Lopes, F.R., et al	Settlement of raft foundation on sand	Instn Civil Engrs	1994	case monitor	equations data		Monitored settlement of raft foundation on sand and compared settlement predictions by three methods.
121	Krizek, R.J.	Probabilistic analysis of predicted and measured settlement	Can Geot J	1977	analysis case	equations data	case study	Method of determining settlements due to consolidation expanded as a probabilistic determination and compared to actual settlements.
122	Leonards, G.A. Frost, J.D.	Settlement of shallow foundations on granular soils	ASCE	1988	analysis	equations		Outline use of dilatometer to determine settlements.
123	Bowles, J.E.	Elastic foundation settlements on sand	ASCE	1987	analysis	equations data	settlement data	Modified elastic method for determining settlements; comparison between predicted and observed for cases from literature.
124	Tanahashi, H.	Probability-based prediction of differential settlements	Soils & Fnds	1994	analysis	equations	all	Use of Timoshenko beam on Pasternak model to determine probabilistic settlements.
125	Georgiadis, M.	Settlement and rotation of footings on sand	Soils & Fnds	1993	model	model results	load vs. settlement plot	Model tests to determine effect of load, load eccentricity, and depth of embedment on settlement of shallow footings; predictions made with method from earlier reference.
126	Indrratna, B., et al	Performance of embankment stabilized with vertical drains	ASCE	1994	case FEM	data		FEM used to predict settlement of embankment on soft clay with vertical drains.
127	Barneich, J.A.	Vehicle-induced ground motion	ASCE	1985	experiment	data	all	Discusses the vibrations from vehicles on various road surfaces.
128	Edil, T.B. Mochtar, N.E.	Prediction of peat settlement	ASCE	1984	analysis	data	all	Presentation and discussion of prediction method for settlement of peats which mainly occurs as secondary compression.
129	Holzlohner, U.	Sand properties governing foundation settlement	ICSMFE	1985	analysis	equation		
130	Tavares, A.X.	Settlement of foundation on clay	ICSMFE	1985	experiment	equation		
131	Bergdahl, V., et al	Calculation of settlement of footings in sands	ICSMFE	1985	experiment	equations data	ratio plots	Six methods of settlement prediction compared with actual results from slabs of various sizes.

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132	Van Impe, W. DeBeer, E.	Improvement of settlement of soft layers by stone columns	ICSMFE		analysis	equations	equations	Equations for determining settlement of soil reinforced with stone columns; uses geometry, stone characteristics, and oedometer.
133	Adachi, K.	Settlement of raft foundation	Asian CSMFE	1987	case analysis	data equations	settlement data	Settlement predictions calculated from various methods and compared to actual for 47 story building.
134	Fukida, N., et al	Foundation improvement by polymer grid reinforcement	Asian CSMFE	1987	model FEM	data	settlement plot	Comparison of lab analysis and FEM analysis of settlements of loading plate on grid reinforced sand.
135	Goughnor, R.R.	Settlement of stone columns in soft ground	ECSMFE	1983	analysis	equations plots	equations plots	Equations and curves to be used in determining the settlement of stone reinforced ground; provided by Vibroflotation Foundation Co.
136	Oteo, C.S.	Prediction of settlements after deep vibration in granular fills	ECSMFE	1983	lab	data		Two volcanic soils and one sand tested for the effect of vibration.
137	Laumans, Q.	Settlement behavior of soil improved with vertical drains	ECSMFE	1983	case monitor	data	settlement plots	Settlements of sands with and without drains were monitored for settlement under diverse magnitudes of loads.
138	Bourdeau, P.L. Harr, M.E.	Stochastic theory of settlement of loose cohesionless soils	Geotechnique	1989	analysis model			Soil model based on intergranular forces and voids used to predict settlement; compared with model tests of strip footing.
139	Nova, R. Montrasio, L.	Settlements of shallow footings on sand	Geotechnique	1991	analysis	data		Method to evaluate settlement and rotation of rigid shallow footings on sand under combined inclined and eccentric loads.
140	Goughnor, R.D. DiMaggio, J.A.	Soil reinforcement methods on highway projects	ASCE	1978	literature cases	data	all	Discussion of successful highway projects utilizing reinforced earth, stone columns, cast-in-place piles, and permanent ground anchors.
141	Schmertmann, J.H.	Static cone to compute settlement over sand	ASCE	1970	analysis literature	data equations	settlement data	Techniques which uses cone penetration test to predict settlement; compared with 3 other methods for 35 settlement measurements.
142	Meyerhof, G.G.	Settlement prediction for spread footings on sand	ASCE	1965	analysis	equations data	settlement data	SPT used to determine settlement of shallow footings on sand; predictions compared with actual measurements from Vargas.
143	DeBeer,E. Martens, A.	Settlement prediction for bridge substructures	ICSMFE	1957	analysis	equations data		Discussion of differential settlement of bridges and method for determining it from CPT data [see 144].
144	DeBeer, E.	Settlement prediction for bridges on sand	ICSMFE	1948	analysis monitor	data	settlement data bridge dimensions	Presents detailed records of settlement for many bridge substructures, details of the bridges, and detailed CPT plots.
145	Meyerhof, G.G.	Bearing capacity and settlement of pile foundations	ASCE	1976	analysis	data equations	equations ratio plots	Two methods for determining pile settlement in sand: one from SPT data and one from CPT data; compared to actual settlements.
146	Skemptom, A.W. Bjerrum, L.	Settlement analysis of foundations on clay	Geotechnique	1952	analysis	equations data	data settlement plots	Settlement prediction method using oedometer results for clays; comparison between predicted and observed.

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147	Schultze, E. Sherif, G.	Prediction of settlements from evaluated settlement observations	ICSMFE	1973	analysis literature	data equations	settlement data	Settlement prediction method developed by statistically determining soil modulus from 48 settlement measurements.
148	Taylor, B.B. Matyas, E.L.	Influence factors for settlement estimates of footings	Can Geot J	1983	literature analysis	equations data		Comparison of Steinbrenner approximation of influence factors for footing settlement with exact Giroud solution.
149	Bazaraa, A.R. Kurkur, M.M.	SPT to predict pile settlements	ASCE	1986	analysis	equations nomographs	equations nomographs settlement plots	Modifications to Poulos-davis elasticity solution for pile settlement depending on construction and loading; comparison between predicted and observed.
150	Berdahl, U. Ottoson, E.	Calculation of settlements on sands from field tests	Swed Geot Inst	1982	analysis	equations data	settlement data	Determination of settlement for two bridges by CPT, WST (weight sounding test), HfA (dynamic probing), and pressuremeter; 6 methods compared.
151	Schmertmann, J. H.	Dilatometer to compute settlement	ASCE	1986	analysis	equations example	settlement data	Empirical settlement method for footings on many soil types which uses dilatometer measurements.
152	Briaud, J.-L. Gibbens, R. M.	Predicted and measured behavior of spread footings on sand	ASCE	1994	analysis	equations data		compilation of many different predictions for the behavior of 5 spread footings on sand compared to actual behavior.
153	Garga, V.K. Quin, J.T.	Settlements of plates on sand	Britain	1974	field	data	settlement data	Settlement of 16 plates on sand compared to predictions made with SPT and CPT data.
154	Levy, J.F. Morton, K.	Settlements of spread footings for building on sand	Britain	1974	field	data	settlement data	Settlement data for multiple similar footings at a few sites.
155	Cornell, C.A.	Probabilistic analysis for soil foundations or embankments	Stats & Prob I		analysis	equations example		Probabilistic estimation of soil properties from measured data at same site.
156	Holtz, R.D. Krizek, R.J.	Statistical evaluation of soils data	Stats & Prob I		analysis	method data		Multiple linear regression analysis to correlate soil properties for 20 sets of data; correlation of unconfined compressive strength.
157	Vanmarcke, E. H.	Probabilistic modeling of soil profiles	ASCE	19	analysis	equations		Equations for determining autocorrelation; contains autocorrelation plots for blow count.
158	D'Appolonia, D. J. et al	Settlement of spread footings on sand	ASCE	1968	case analysis	data	all	Discussion of previous settlement methods; case study of steel mill with 340 footings, monitored for 4 years.
159	Holtz, W. G. Gibbs, H.J.	Settlement of spread footings on sand	ASCE	1969	discussion		all	Discusses correction for overburden pressure included in [158].
160	Peck, R.B. Bazaraa	Settlement of spread footings on sand	ASCE	1969	analysis	equation	all	Correction for relative density of sand; reduction of Terzaghi and Peck's method for settlement by two-thirds, correction for water table.
161	Kovacs, W.D.	Variation in SPT values	ASCE	1994	analysis	equations		Discussion of variation in blow counts from SPT from liners, hammer type, operation, etc.

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162	Skiles, D.L. Townsend, F.C.	Shallow foundation settlement from dilatometer	ASCE	1994	analysis tests	equations		Use of Schmertmann's prediction of settlement from dilatometer [151] for precast square R/C footings.
163	Wahls, H. E. Gupta	Settlement of shallow foundations on sand	ASCE	1994	analysis tests	equations data		Elastic prediction modified to account for strain level and confining pressure; settlement of 31 footings calculated and compared to other predictions and actual settlement.
164	Paikowsky, S. G., et al	Pile settlement based on dynamic measurements	ASCE	1994	tests	data		83 measurements on 57 piles, 89% of predictions were greater than actual; equations not given.
165	Bauer, G. E.	Control of settlements using geogrids	ASCE	1994	field	data		Geogrids used to reduce rutting in unpaved roads and airfields (20%-25% reduction); also used to reduce settlement of soil around conduit.
166	Stewart, D. Fahey, D.	Stone columns in soft clay	ASCE	1994	lab tests	data		Centrifuge test of stone column reinforced soil and 1:10 scale model of single column; stress concentration and settlement reduction measured.
167	Brignoli, E., et al	Stone columns to reduce settlements	ASCE	1994	case	data		Stone columns reduced settlement in storage area by 50%.
168	Paice, G. M., et al	Influence of spatially random soil stiffness on settlement	ASCE	1994	analysis	data		Monte Carlo simulation used for linear elastic soil with spatially random Young's modulus.
169	Berardi, R. Lancelotta, R.	Settlements of footings on sand	ASCE	1994	analysis	data		125 cases from [116] used to compare "new" elasticity method too four other methods.
170	Poulos, H. G.	Pile settlement prediction	ASCE	1994	literature	outline		Presents various methods by which pile settlement can be predicted.
171	Alpan, I.	Settlement of footings on sand	Pub. Works	1964	analysis	equation		Use of plate loading tests and SPT values to predict settlement (1 example).
172	D'Appolonia, D. J., et al	Settlement of footings on sand	ASCE	1970	analysis	equation		Closure to [158]; includes elastic settlement prediction method.
173	Yamashita, K., et al	Immediate settlement of piles	Soils & Fnds.	1987	analysis	equation data		26 measured settlements of single piles predicted by modified Mindlin method [108], predicted generally 1.5 times observed; no pile groups.
174	Asaoka, A.	Observational procedure of settlement prediction	Soils & Fnds.	1978	analysis	equations method	graphical method coefficient plots	Procedure for predicting future settlements from observed settlements, referenced in [110].
175	D'Appolonia, D. J., et al	Initial settlement of structures on clay	ASCE	1971	analysis	equations	settlement ratio plots	Technique for predicting initial clay settlement from elastic settlement.
176	Corotis, R. B., et al	Probabilistic approach to predicting consolidation settlement	TRB	1975	analysis	equations data		Equations developed for distribution of settlement based on Buissman-DeBeer method [143] using probabilistic techniques and over 700 consolidation tests.

rec	Authors	Subject	Source	Year	Type	Content	Copied	Summary
177	Parry, R. H. G.	Bearing capacity from SPT values	ASCE	1977	analysis	equation		Settlement prediction for footings on sand which accounts for overburden pressure measured by SPT included.
178	Parry, R. H. G.	Estimating settlement from plate bearing tests	Geotechnique	1978	analysis	equation		Settlement prediction based on plate bearing tests and SPT.
179	Menard, L.	Pressuremeter test results	Sols	1975	analysis	equations	influence factors	Empirical settlement techniques for piles and footings on any soil from pressuremeter measurements (used by Briaud [93]).
181	Christian, J. T. Carrier, W. D. III	Janbu's influence factors	Can Geotech. J	1978	literature	plots	improved charts	Investigates the origins of Janbu's influence factors for elastic settlements and points out shortcomings.
182	Pells, P. J. N. Turner, R. M.	Janbu's influence factors	Can Geotech J	1978	FEM	data		Limited study that refutes data in [181].
183	Burland, J. B.	Janbu's influence factors	Can Geotech J	1978	discussion			Points out incorrect comparisons in [182].
184	Christian, J. T. Carrier, W. D., III	Janbu's influence factors	Can Geotech J	1978	discussion			Points out limited scope of [182].
185	Picornell, M. Del Monte, E.	Settlement of shallow foundations	ASCE	198?	case	equations data		12 settlement plates on sand monitored for 10 days and compared to predictions by three methods.
186	Cressie, N.	Spatial prediction and site selection	EPA	1987	analysis	equations		Trend surface models and random field models for estimating soil properties from other observations.
187	Anderson, L. R., et al	Probabilistic modeling of embankments	Utah State U	1982	analysis measurement	equations data	autocorrelation equations & plots	Equations developed for autocorrelation of shear strength and failure planes; 23 CPT soundings.
188	Randolph, M. F. Wroth, C. P.	Vertical deformation of pile groups	Geotechnique	1974	analysis	equations		Equations for shaft and base settlement of piles and interaction between piles; comparison to experimental piles (within 10 to 20%).
189	Koerner, R. M. Partos, A.	Settlement of building on pile foundation in sand	ASCE	1974	analysis	equations data		Pile group and single pile loading tests compared with several prediction techniques.
190	Poulos, H. G. Davis, E. H.	Settlement of single axially loaded piles and piers	Geotechnique	1965	analysis	equation		Prediction equation for settlement of piles, includes table of influence factors, based on elasticity.
191	Berardi, R. Lancellota, R.	Stiffness of granular soils	Geotechnique	1988	analysis	equation		Reexamination of [116] to determine stiffness relation to N for elasticity equation for settlement on sand.
192	Skempton, A. W.	Effects of overburden, relative density, etc. on SPT	Geotechnique	1983	analysis field tests	equations		Development of empirical relationships between N and relative density; contains effects of delivered energy and liners on results - see [161].
193	Giroud, J. P.	Influence factors	ASCE	1972	analysis	data	influence factors	Influence factors for empirical settlement predictions (commonly used).
194	Schmertmann, J. M.	Statics of SPT	ASCE	1979	analysis			Delivered energy for SPT, similar to [161] and [192].

Section 5 PREDICTION OF TOTAL SETTLEMENTS

This section summarizes studies of the accuracy of predictions of settlements of structures. Predictions of settlement are important to the design and evaluation of methods intended to mitigate pavement faults. The ability to design for compatible settlements of embankments and structural foundations relies on predictions of settlement. An evaluation of the magnitude of pavement faults that may occur at a site relies on predictions of settlements.

Settlement predictions for footings and for piles are presented and, where possible, sources in the literature are grouped by similar method of prediction. Studies of the accuracy of predictions are reviewed. Mean error in predictions and variance of error are computed from studies found in the literature.

This review contains seven categories of methods of prediction of settlement, includes work from 36 sources in the literature, and compares predicted and observed settlements for more than 1500 data points.

A ratio of predicted settlement to observed settlement is computed. It is found that predictions of settlement are sometimes inaccurate. More important to the question of pavement faults, it is found that there is a significant variability in the relation of predicted to observed settlements. Predictions of settlement from generally accurate methods fail to estimate settlements of some foundations because of the inherently random nature of settlements.

Randomness sets a limit on the reliability of predictions of settlements and makes it less likely that some differences in settlements, possibly causing pavement faults, can be identified at design time.

METHODS FOR PREDICTION OF SETTLEMENTS

Methods of prediction of settlements are outlined. These are methods that have been used by other authors in studies of accuracy of predictions of settlements. Their work is compiled here. This synthesis goes on to an independent assessment or the accuracy of predictions of settlements. Notations differ among literature sources. In this synthesis a common notation is adopted, and equations from individual papers are transcribe to this common notation.

s	Settlement
N	Blow count from standard penetration test (SPT), blows/ft
N'	SPT value corrected for overburden pressure and presence of water table
B	Width of rectangular or square footing or diameter of circular footing
L	Length of rectangular footing ($L \geq B$)
q	Applied pressure
q_b	Overburden pressure
q_c	Cone penetration resistance
E	Soil modulus
ν	Poisson's ratio of soil
H	Thickness of layer
Δh	Incremental thickness of compressible soil layer
D	Depth of embedment of footing
I	Influence factor
R	Ratio of total settlement to initial elastic settlement

Table 5-1 Notation for Predictions of Settlement

PREDICTION OF SETTLEMENT OF FOOTINGS.

ELASTICITY

$$s = (qB/E)*I \qquad \text{Eq. 5-1}$$

Soil behavior is elastic for applied loads that are much less than ultimate loads. Settlement is linearly proportional to load. The soil modulus, E, is determined from empirical relations with in-situ strength tests or from laboratory testing. The elastic settlement is adjusted by influence factors represented as the single variable I in Eq. 5-1. Influence factors reflect characteristics of the soil and footing, including: Depth of embedment of footing, thickness of supporting soil layer, shape of footing (strip, rectangular, square, circular), aspect ratio of rectangular footing (length versus width), depth of water table, overburden pressure, Poisson's ratio for the soil, and creep of soil.

Variations of Eq. 5-1 for footings on sand include Berardi & Lancellota [1988], Bowles [1987], D'Appolonia [1968], and Papadopoulos [1992]. The variations depend primarily on the influence factors and the methods for determining the soil modulus. D'Appolonia [1971] and Skempton-Bjerrum [1952] use the elasticity approach to calculate the initial settlement on clay and then modify that by a factor or by adding additional terms (representing consolidation) to calculate the final settlement.

STRAIN FACTOR

$$s = \{q\Sigma(\Delta h/E)\}*I \qquad \text{Eq. 5-2}$$

The strain factor method calculates the settlement as the sum of the integration of strains of individual soil layers of height, Δh, with each soil layer having its own modulus, E. The total settlement is adjusted by strain influence factors, I, much like the elastic method. These individual layer settlements are summed over the entire compressible layer to a depth of at least two times the width of the footing (2B) below the footing. This method is similar to the popular Schmertmann method [1970] and the more recent method of Wahls-Gupta [1994].

COMPRESSIBILITY CONSTANT

$$s = \Sigma\{(1/C)\ln(q/q_b)\Delta h\} \qquad \text{Eq. 5-3}$$

The compressibility constant method uses a coefficient (the constant of compressibility), C, the applied pressure, q, and the overburden pressure, q_b, to calculate the settlement for a number of layers. As in the strain method, settlements of individual layers are summed over all layers to a depth of 2B below the footing. The compression index is calculated from cone penetration test data ($C = 1.5q_c/q_b$) as in the Buismann-DeBeer method [DeBeer and Martens, 1957] or from the initial void ratio and the compression index, c_c, determined from consolidation tests as in Hough [1959] ($C = (1+e_o)/c_c$).

EMPIRICAL

Empirical methods determine settlements from data on actual settlement data correlated with in-situ tests such as plate loading tests (Alpan [1964], Meyerhof [1965], Peck & Bazaraa [1969], Parry [1978], Terzaghi & Peck [1948]), standard penetration test data, N, (Alpan [1964], Burland

& Burbidge [1985], Meyerhof [1965], Oweis [1979], Peck & Bazaraa [1969], Parry [1977], Schultze & Sherif [1973], Terzaghi & Peck [1948]), pressuremeter data (Menard [1975]), and dilatometer data (Schmertmann [1986]). Cone penetration data, used in Schmertmann's method and the Buismann-DeBeer method, are often converted to standard penetration data using empirical correlations (best if done on a site by site basis).

Empirical relations generally include one or more of the following parameters:

- N', N -- corrected or uncorrected blow count from SPT.
- $(2B/B+1)^2$, B/B_o -- empirical relations from plate loading tests that compare footing width, B, to a reference plate width, B_0 (often 1).
- q -- applied pressure.
- I -- influence factors..
- α -- exponent for parameters (i.e. N^α, E^α, B^α).

Correlations between in-situ data and actual settlements are discovered by regression analysis.

OBSERVATIONAL PROCEDURE

$$s(t_j) = \beta_o + \Sigma\beta_s s_{j-s} \qquad \textbf{Eq. 5-4}$$

The settlements at any time, t_j, are determined from settlements that have already been observed. The coefficients are determined from consolidation theory. This procedure, developed by Asaoka [1978], cannot be used to predict the settlement of a structure that has not been constructed. However, this procedure may be useful in determining the potential magnitude of total settlement once an existing structure has begun to settle.

PILES

SINGLE PILES

Methods for prediction of settlement single piles (Bazaraa & Kurkur [1986], Poulos-Davis [1965], and Yamashita et al. [1989, 1987]) are based directly or indirectly on Mindlin's first solution; an elastic solution. The basic equation is generally the same as that for footings, although it varies for each author. Bazaraa & Kurkur utilize plots and charts to allow the use of more influence factors in determining the settlement.

PILE GROUPS

Settlements of pile groups are predicted with empirical relations. Equations proposed by Meyerhof [1976] treat pile groups as equivalent footings, using effective depth that is a fraction of pile lengths, and using in-situ tests (CPT or SPT) to determine settlements of groups.

An alternative method is to calculate the ratio of pile group settlement to a single pile settlement. A single pile settlement determined from a load test are multiplied by the ratio to determine the settlement of the group. The ratio equations depend on the length, diameter, and spacing of the piles within the group. Kaniraj [1993] reports on equations proposed by Meyerhof, Morgan & Poulos, and Skempton have all proposed ratio equations

.Methods for the predictions of settlements are summarized in Table 5-2

Table 5-2 Methods of Predictions of Settlement

Source	Notes
Alpan, 1963 $s = 4\alpha_o P/(B+1)^2 I_s$	Empirical using SPT and plate test relations Footings on sand Parameters α_o (in·ft^2/ton) = inverse of modulus of compressibility decreases with decreasing N range: 0.01 → 0.15 I_s = shape influence factor increases with increasing L/B ratio range 1 → 2.36 N corrected for very fine or silty sand of moderate density below groundwater table: $N' = 15 + 0.5(N - 15)$ for $N > 15$
Asaoka, 1978 $s_i = \beta_o + \Sigma\beta_k s_{i-k}$ $k = 1..n$	Observational Footings on clay Parameters $\beta_o,...,\beta_n$ determined from consolidation theory s_i = settlement at time t_i
Bazaraa & Kurkur, 1986	Elastic - iterative approach using plots and charts Single piles on any soil Parameters pile diameter pile length pile type (Prepakt, Bauer, etc.) loading soil installation method
Berardi & Lancellotta, 1994 $s = (q' - q_o')BI/E$	Elasticity Footings on sand Parameters I = influence factor related to foundation shape, Poisson's ratio, and layer thickness $E = K_E P_a[0.5(q' + q_o')/P_a]^{0.5}$ q_o', q' = initial and final value of vertical effective stress @ depth = 1/2 of the active zone depth, within which the majority of settlement occurs {K_E, P_a are unclear}
Bowles, 1987 $s = qB(1-v^2)I_1 I_2/E$	Elastic Footings on sand Parameters I_1 = influence factor = $F_1 + [(1 - 2v)/(1 - v)]*F_2$ F_1 = non-dimensional factor increases with increasing H/B range: 0.036 → 1.941 F_2 = non-dimensional factor decreases with increasing H/B increases with increasing L/B I_2 = influence factor [Fox, 1948] increases with increasing L/B decreases with increasing D/B increases with increasing v E = soil modulus range: $2.5q_c \rightarrow 3.5q_c$ (from CPT data) 10(N + 15), ksf (from SPT data)
Buisman-DeBeer, 1957 $s = \Sigma[(1/C)\ln(q/q_o)h]$	Compressibility coefficient Footings on sand Parameters $C = 1.5q_c/q_o$ (from CPT data)

Source	Notes
Burland & Burbidge, 1985 $s = 1.71\, I_s I_H I_t q' B^{0.7}/N^{1.4}$, mm normally consolidated sand $s = 1.71\, I_s I_H I_t (q' - 2/3\, q_o')B^{0.7}/N^{1.4}$, mm overconsolidated sand $s = (1/3)1.71\, I_s I_H I_t q' B^{0.7}/N^{1.4}$, mm overconsolidated, $q' < q_o'$	Empirical using SPT Footings on sand Parameters B = footing width, m q = footing pressure, kN/m^2 I_s = shape influence factor $=[(1.25\, L/B)/(L/B + 0.25)]^2$ depends on L/B range: $1 \rightarrow 1.56$ I_H = layer thickness influence factor = $(H/z)(2 - H/z)$ depends on layer thickness, H, and depth of influence, z range: $0 \rightarrow 1$ I_t = time factor = $1 + R_3 + R_t \log(t/3)$ R_3 = percentage of time-dependent settlement during first 3 years R_3 = percentage of time-dependent settlement during each log cycle of time after 3 years. Depends on soil type range: $0 \rightarrow 1$
D'Appolonia, 1970 $s = qBI/M$	Elastic Footings on sand Parameters M = soil modulus increases with increasing N (SPT) I = influence factor from Janbu, 1959 depends on L/B, D/B, H/B
D'Appolonia, 1971 $s_i = RqBI/E_u$	Elastic Footings on clay Parameters s_i = initial clay settlement R = ratio of initial settlement to elastic settlement decrease with increasing q/q_u increases with increasing f, initial shear stress ratio increase slightly with decreasing H/B range: $R > 1.0$ f = initial shear stress ratio = $(1-K_o)/(2S_u/q_o')$ S_u = undrained shear strength E = soil modulus range: $1000S_u \rightarrow 1500S_u$ for lean inorganic clays lower for highly plastic and organic clays q_o' = initial stress (before loading)
Giroud, 1972 $s = qB/E * I$	Elastic Footings on any soil Parameters I = influence factor (from tables). Depends on ν, L/B, B/H
Hough, 1959 $s = \Sigma[(1/C)\ln(q/q_o)h]$	Compressibility coefficient Footings on sand Parameters $C = (1 + e_o)/c_c$ e_o = initial void ratio c_c = virgin compression index

Source	Notes
Kaniraj, 1993 $R_g = N_p/R_g'$	Pile group ratio for sand Parameters $R_g' = 1.128[(m-1)(n-1)t^2/(1+2(L/D)\tan\theta)^2 + (m+n-2)t/(1+2(L/D)\tan\theta)+1]^{1/2}$ = ratio for equal stress N_p = number of piles m = (number of rows of piles) n = (number of columns of piles) L = embedded length of pile θ = loan dispersion angle $\cong 7°$
Menard, 1975 s, cm = $(2/9E_B)qB_o(I_{s1}(B/B_o))^{\alpha} + (\alpha/9E_A)qI_{s2}B$ B > 60cm s, cm = $(2/9E_B)qB_oI_{s1}^{\alpha} + (\alpha/9E_A)qI_{s2}B$ B < 60cm	Empirical using pressuremeter Footings on any soil Parameters α = factor related to soil type $0.25 \rightarrow 1.0$ B_o = reference width = 60cm I_{s1} = shape influence factor increases with increasing L/B range: $1 \rightarrow 2.65$ I_{s2} = shape influence factor increases with increasing L/B range: $1 \rightarrow 1.5$ $E_A = E_1$ (modulus of top layer within depth of influence) E_B = weighted series average of soil moduli for layers within depth of influence upper layers are weighted higher α (unnamed factor) depends on soil type range: $0.25 \rightarrow 1$
Menard, 1975 s, cm = $I_D(q'/E)B_o(I_{s2}(B/B_o))^{\alpha}$ 60cm < B < 200cm s, cm = $I_D(q'/E)I_{s2}^{\alpha}$ B < 30cm	Empirical using pressuremeter Single piles with diameter less than 2m on any soil Parameters I_D = embedment depth influence factor = $1/[\{1.6 \rightarrow 0.2\}D/B]$ q' = stress at butt of pile I_s = shape influence factor 1 (circular) 1.12 (square)
Meyerhof, 1965 $s = 8q/N$ B < 4ft $s = 12[B/(B+1)]^2/N$ B > 4ft	Empirical using SPT Footings on sand
Meyerhof, 1976 $s = 2qB^{1/2}I_D/N$ (multiply by 2 for silty sand)	Empirical using SPT Pile groups on sand Parameters q = net foundation pressure, Tsf B = width of pile group, ft N' = SPT blow count corrected within seat of settlement (D < B) I_D = embedment depth influence factor = $1 - D'/(8B) > 0.5$ D' = effective depth of pile group
Meyerhof, 1976 $s = qBI_D/(2q_c)$	Empirical using CPT Pile groups on sand Parameters I_D = embedment depth influence factor = $1 - D'/(8B) > 0.5$ D' = effective depth of pile group
Meyerhof, 1959 $R_g = t(5 - t/3)/(1 + 1/n)^2$	Empirical Pile group ratio Parameters t = ratio of spacing between adjacent piles to diameter of pile n = number of rows of pile for a square pile group

Source	Notes
NAVFAC, 1983 $s = (4qB/I)(B^2/(B+1)^2)$ $B < 20'$ $s = (4qB/I)(B^2/(B+1)^2)$ $B > 40'$ interpolate for values of B between 20′ and 40′	Empirical Footings on sand or clay Parameters q (tons/ft^2), B (ft) I (in·tons/ft^2) = influence factor depends on soil type and relative density (sand) or unconfined compressive strength (clay) increases with increasing sand density or clay stiffness range: 50 → 350
Oweis, 1979 $s = \Sigma[(qB/E)I]$	Empirical Footings on sand Parameters I = influence factor, determined for each layer depends on N-value, effective vertical stress in layer correction for blow count $N' = 4N/(1 + 2\sigma_v')$ $\sigma_v' < 1.5$ksf $N' = 4N/(3.25 + 0.5\sigma_v')$ $\sigma_v' > 1.5$ksf
Papadopolous, 1992 $s = qBI/E$	Elasticity Footings on sand Parameters I = influence factor depends on stress history, D/B, L/B, loading, E vs. q′ $E = E_o + \lambda q'$ $(0 < \lambda < E_o/q')$ E_o = initial modulus from stress-strain curve of consolidation test E can also be determined by in-situ tests ($\lambda = 0$)
Parry, 1977 $s = 300qB/N_m$	Empirical using SPT Footings on sand Parameters N_m = N corrected for overburden pressure = $N(\sigma_v')_1/(\sigma_v')_2$ $(\sigma_v')_1$ = effective overburden @ 0.75B below footing $(\sigma_v')_2$ = effective overburden @ depth N is measured
Parry, 1978 $s = s_1(B/B_1)[(N_m)_1/(N_m)_2]I_e$	Empirical using SPT Footings on sand Parameters s_1 = settlement of plate B_1 = width of plate $(N_m)_1$ = N-value (blow count) for plate test $(N_m)_2$ = N-value for footing N_m = weighted average = $(3N_1 + 2N_2 + N_3)/6$ N_1 = N @ 0 → 2/3 B N_2 = N @ 2/3 B → 4/3 B N_3 = N @ 4/3 B → 2B I_e = excavation correction for foundations in excavations not backfilled increases with increasing D_e/B (D_e = depth of excavation) range: 1 → 4

Source	Notes
Peck and Bazaraa, 1969 $s = I_w I_D(2q/N')(2B/(B+1)^2)$	Empirical using SPT Footings on sand Parameters I_w = water table influence factor range: $0 \rightarrow 1$ I_D = embedment influence factor = $1 - 0.4(\gamma D/q)^{1/2}$ q (tons/ft^2), B (ft) N′ = N corrected for overburden pressure γ = unit weight of soil (pcf) D = depth of footing embedment
Poulos-Davis, 1965 $s = PI/LE$	Elasticity Single piles Parameters I = influence factor depending of L/d, H/L L = length of pile d = diameter of pile
Randolph and Wroth, 1974 $s = P(1 - \nu)/(2dG)$	Single piles Parameters d = diameter of pile G = shear modulus of soil
Schmertmann, 1970 $s = I_o I_c(q - q_o)\Sigma(I_s/E)h$	Strain factor Footings on sand Parameters I_o = overburden influence factor = $1 - 0.5[q_o/(q - q_o) > 0.5$ I_c = creep influence factor = $1 + 0.2\log(t/0.1)$ t in years I_s = influence factor $(1 + \nu)[(1 - 2\nu)A + F]$ depend on A and F which vary with L/B and depth of layer h range: $0 \rightarrow 7$ $E = 2q_c$ (from CPT)
Schmertmann, 1986 $s = \Sigma(\Delta\sigma_v' \cdot h/M)$	Empirical using dilatometer Footings on any soil Parameters $\Delta\sigma_v'$ = increase in vertical effective stress in layer after load is applied M = modulus from dilatometer
Schultze & Sherif, 1973 $s = qI_H/[1.14N(B/B_1)^{0.5}B/(1 + 0.4D/B)$	Empirical Footings on sand Parameters I_H = thickness of layer influence factor depends on H/B < 2 B_1 = 1cm B = footing width, cm q = mean contact pressure, kg/cm^2
Skempton, 1953 $R_g = [(4B + 2.7)/(B + 3.6)]^2$	Empirical Pile group ratio for any soil Parameters B = pile group width, m

Source	Notes
Skempton-Bjerrum, 1952 $s = qB(1-\nu^2)I_s I_H + \mu s_{oed}$	Elastic Footings on clay Parameters s_{oed} = settlement from oedometer $\mu = A + \alpha(1 - A)$ increases with increasing A range: $0.3 \rightarrow 1.2$ A = pore water pressure coefficient increases with clay sensitivity (heavily overconsolidated to very sensitive) range: $0 \rightarrow 1.2$ α = coefficient increases with H/B range: 0 (H/B = 0) $\rightarrow$ 0.25 (H/B $\rightarrow \infty$)
Terzaghi and Peck $s = (3q/N)(2B/(B+1)^2)$	Empirical using SPT Footings on sand Parameters q (tons/ft^2), B (ft)
Vesic, 1967 $R_g = (B/D)^{1/2}$	Empirical Pile group ratio on any soil Parameters B = width between centers of two edge piles D = depth of piles
Wahls-Gupta, 1994 $s = q\Sigma[I \cdot h/(E((q_o' + q')/2)^{1/2})]$	Strain factor Footings on sand Parameters $E = 43.8(1 + \nu)K(p_{atm})^{1/2}$ K = coefficient p_{atm} = atmospheric pressure (reference pressure) I = influence factor = $(1 + \nu)[I_z - 3\nu I_m]$ I_z, I_m = influence factors from Bouissinesq
Yamashita, 1987 $s_{iKGL} = I_{iKGL}/E_{sij} d\, F_{jL} \eta(r_{KL})$	Elastic Single piles or pile groups on any soil Parameters s_{iKGL} = vertical displacement of soil adjacent to the Kth element of the ith pile due to F_{jL} I_{iKGL} = displacement influence factor with respect to the ith element of the Kth pile due to F_{jL} E_{sij} = equivalent elastic modulus of soil between the ith and jth layers d = pile diameter F_{jL} = interaction force acting on the jth element of the Lth pile r_{KL} = horizontal distance between the Kth pile and Lth pile $\eta(r_{KL})$ = correction function range: less than 1.0

COMPARISON OF PREDICTED AND OBSERVED SETTLEMENTS.

The performance of predictions of settlement is measured both as accuracy and as variability. Predictions of settlement should be accurate in the mean, and also accurate for individual foundations.. Because settlements themselves are random, accuracy in the mean is more easily achieved than accuracy in predictions for individual foundations.

Predictions are normalized against observed settlements yielding a settlement ratio R_P

$$R_P = \frac{S_{Pre}}{S_{Obs}} \qquad \text{Eq. 5-5}$$

For R_P, a value of 1.0 indicates that the prediction matches the observation. Values greater than 1.0 indicate that the prediction is larger than the observation. Settlement ratios are computed for comparisons of predicted and observed settlements that have been reported in the literature. Work under this synthesis does not attempt to compute settlement, but only to work with settlements computed and reported by others.

For each study of settlements, settlement ratios R_P are computed for all data together with mean μ_P and median m_P values, and standard deviations σ_P of settlement ratios. Settlement ratios are listed in Table 5-3. Predictions of settlements are generally conservative. In the mean, settlements are overestimated by 64%. The median overprediction is 20%. Figure 5-1 shows mean settlement ratios and a range of ± one standard deviation. Figure 5-2 shows all data points. In Figure 5-2 note the change in y-axis and the presence of outliers for settlement ratios. Predictions are scattered. Even for methods that have a mean settlement ratio near 1.0, fully accurate, settlements of individual foundations may be overestimated by a factor of 4 or more.

Figs	Category	Source / Method	Data Points	R_P Mean	R_P Median	R_P COV
2.1, 2.2	All sources	All	1522	1.64	1.20	0.83
2.3, 2.4	Overall method performance	Observational	23	0.91	0.85	0.45
		Strain factor	144	1.36	1.12	0.54
		Compressibility coefficient	152	1.65	1.42	0.75
		Elasticity	440	1.28	1.05	0.72
		Empirical†	478	1.24	1.09	0.58
		Empirical	764	1.92	1.36	0.84
2.5, 2.6	Methods for footings	Oweis, 1979	10	1.65	1.59	0.36
		D'Appolonia, 1971	11	1.20	1.03	0.35
		Parry, 1977	13	0.88	0.85	0.37
		Schmertmann, 1986	16	0.97	1.13	0.32
		Skempton-Bjerrum, 1952	17	1.18	1.10	0.22
		Hough, 1959	20	1.78	1.85	0.35
		Asaoka, 1978	23	0.91	0.85	0.44
		Peck & Bazaraa, 1969	30	0.73	0.64	0.56
		Menard, 1975	31	1.07	1.03	0.38
		Wahls-Gupta, 1994	31	1.13	0.98	0.53
		Bowles, 1987	48	2.25	2.28	0.44
		Schultze & Sherif, 1973	56	0.97	0.97	0.33
		Papadopoulos, 1992	61	1.08	0.99	0.32
		Meyerhof, 1965	74	1.52	1.50	0.32
		Schmertmann, 1970	113	1.43	1.18	0.53
		Alpan, 1964	118	3.44	3.02	0.44
		Berardi & Lancellotta, 1994	125	1.06	0.83	0.76
		Buismann-DeBeer, 1957	132	1.63	1.32	0.80
		D'Appolonia, 1970	135	1.28	1.05	0.87
		Burland & Burbidge, 1985	145	1.45	1.24	0.63
		Terzaghi & Peck, 1948	168	2.77	2.43	0.66
2.6, 2.7	Elasticity methods for footings	D'Appolonia, 1971	11	1.20	1.03	0.35
		Skempton-Bjerrum, 1952	17	1.18	1.10	0.22
		Bowles, 1987	48	2.25	2.28	0.44
		Papadopoulos, 1992	61	1.08	0.99	0.32
		Berardi & Lancellotta, 1994	125	1.06	0.83	0.76
		D'Appolonia, 1970	135	1.28	1.05	0.87
2.8, 2.9	Empirical methods for footings	Oweis, 1979	10	1.65	1.59	0.36
		Parry, 1977	13	0.88	0.85	0.37
		Schmertmann, 1986	16	0.97	1.13	0.32
		Peck & Bazaraa, 1969	30	0.73	0.64	0.56
		Menard, 1975	31	1.07	1.03	0.38
		Schultze & Sherif, 1973	56	0.97	0.97	0.33
		Meyerhof, 1965	74	1.52	1.50	0.59
		Alpan, 1964	118	3.44	3.02	0.44
		Burland & Burbidge, 1985	145	1.45	1.24	0.63
		Terzaghi & Peck, 1948	168	2.77	2.43	0.66
2.10 2.11	Methods for piles	Meyerhof, 1976[CPT]	15	1.51	1.58	0.32
		Meyerhof, 1976[SPT]	16	1.21	1.08	0.30
		Yamashita, 1987	22	1.37	1.29	0.36
		Bazaraa & Kurkur, 1986	72	1.01	1.00	0.34

Table 5-3 Summary of Settlement Ratios

Six types of settlement predictions are compared in Figure 5-3 and Figure 5-4. Observational methods that rely on an extrapolation of settlement that have already occurred appear to offer

† without Alpan and Terzaghi & Peck source data
CPT Uses CPT data
SPT Uses SPT data

the most accurate and least scattered predictions. Empirical methods relating blowcount to settlement may be the least accurate and the most scattered.

Predictions of settlements of footings are shown in Figure 5-5 and Figure 5-6. Predictions for settlements of footings by elasticity methods are shown in Figure 5-7 and Figure 5-8. Empirical methods for settlements of footings are shown in Figure 5-9 and Figure 5-10. Predictions for settlements of piles are shown in Figure 5-11 and Figure 5-12.

A similar, though smaller, review of predictions of settlements for footings is reported by Gifford [et al. 1987]. Data presented by Gifford are compared to data compiled in this synthesis (Table 5-4). Gifford found some predictions that underestimated settlements. The data compilation in this synthesis finds no similar underestimates. Small size is an issue (discussed below). Settlement predictions are found to be less accurate, and more conservative, in larger comparisons.

	Gifford [et al. 1987]		This Synthesis	
Method	**Footings**	**R_P mean**	**Footings**	**R_P mean**
Strain factor	30	1.35	144	1.36
Compressibility coefficient	20	1.78	152	1.65
Elasticity	30	0.97	440	1.28
Empirical	60	0.85	478	1.24

Table 5-4 Comparison with Gifford's Evaluation of Settlement Accuracy

Settlement ratios are not strongly affected by the magnitude of real settlements. In Figure 5-13 and Figure 5-14 mean settlement ratios are plotted against mean observed settlements. Figure 5-13 shows all data, include four studies of large settlements. Figure 5-14 shows the subset of data at observed settlements less than 2.0 inches. There is no apparent trend in settlement ratios as a function of settlement magnitude. The absence of large settlement ratios for large real settlements is not conclusive because there are so few points at large settlement.

Mean settlement ratio is weakly correlated with the number of points in a data set (Figure 5-15). There is a small tendency for larger mean settlement ratio for larger data sets because there is a greater probability of encountering some outliers. Median settlement ratio has no correlation with the number of data points.

The scatter in predictions is expressed as the coefficient of variation of R_P. COV values are shown in Figure 5-16 for all data and for footings and piles as separate categories. Many studies have a COV of about 0.4. COV yields a measure of how much a prediction of settlement may differ from the settlement for an individual foundation. The settlement ratio R_P is not correlated with COV of total settlement (Figure 5-17), and the COV of settlement ratio is not correlated with the mean magnitude of settlements (Figure 5-18 and Figure 5-19).

COV of settlement ratio R_P is strongly correlated with the number of data points (Figure 5-20). The accuracy of different methods for prediction of settlement then must be considered in the light of the size of studies of methods. Relatively good performance of some methods may be an artifact of a small population of settlements.

SIGNIFICANCE

The potential for the appearance of pavement faults at individual projects may be estimated by computation of the expected settlements of abutments and of embankments. Predictions must be accurate.

Present-day methods for prediction of settlement are conservative, rather than accurate. Many methods overestimate settlements by only 20% or 30%, but settlement predictions, in the aggregate overestimate real settlements by more than 60%. Overestimation of settlements by 60% is the mean performance of methods of prediction. Most methods will overestimate settlements by a factor of 2 or 3 for some foundations. Conservative estimates may be useful as limits in design of structures, but they are less useful in the prediction of pavement faults. The general finding here is that the ability to predict settlements in the course of normal design process may not be sufficiently accurate for the assessment and mitigation of pavement faults.

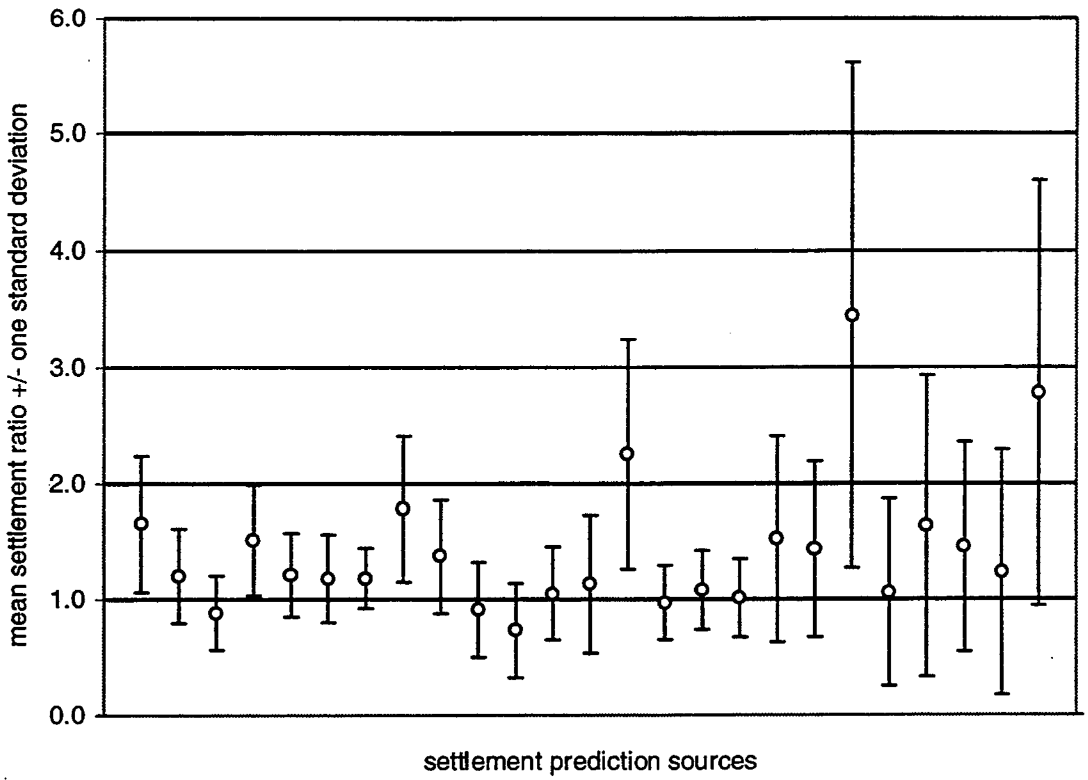


Figure 5-1 Performance of Settlement Prediction - All Methods

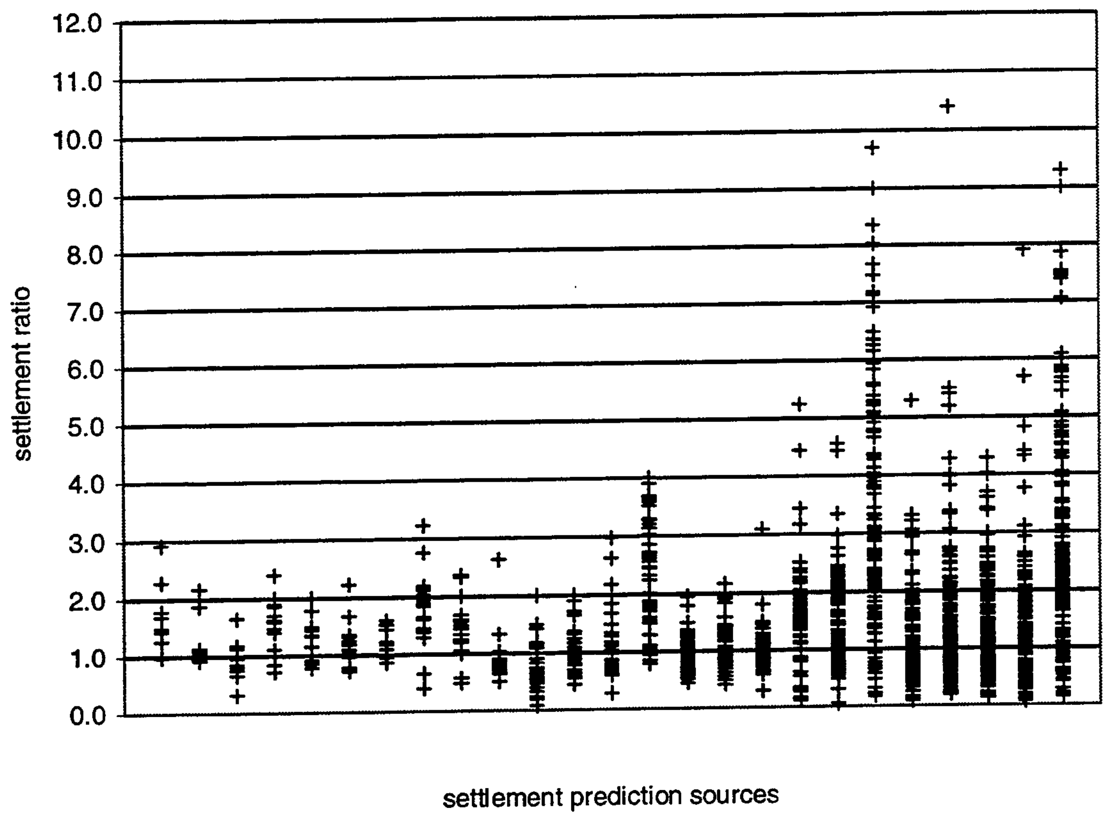
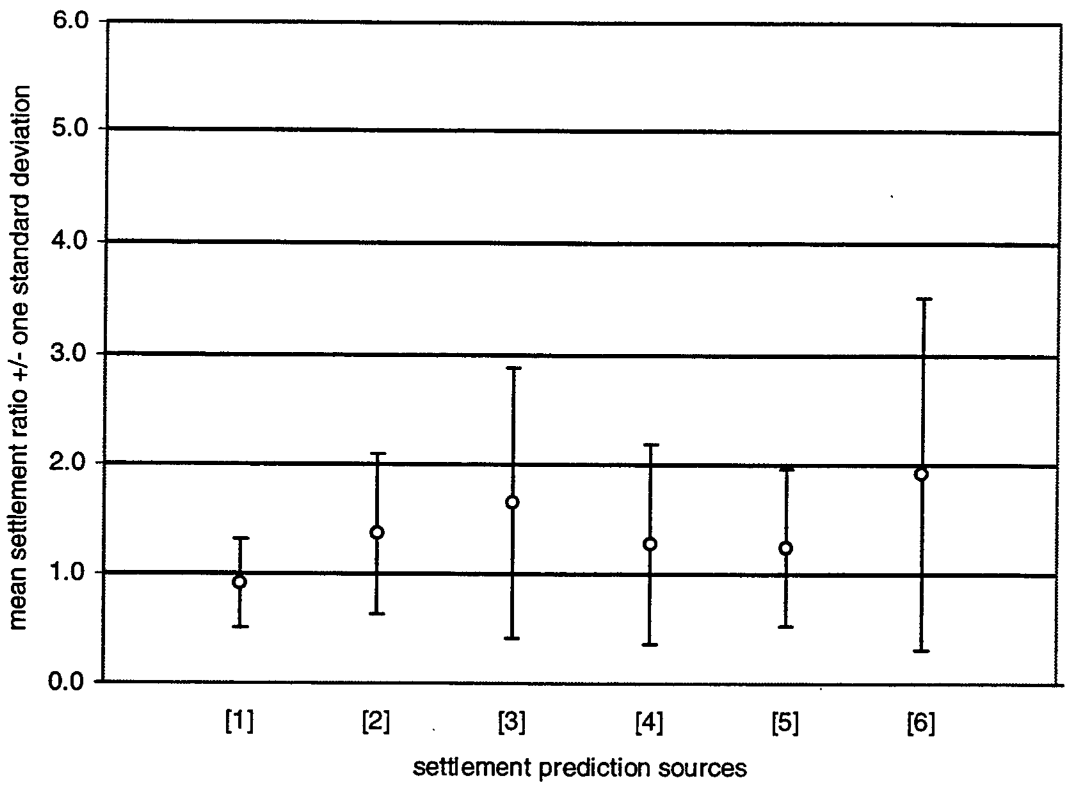


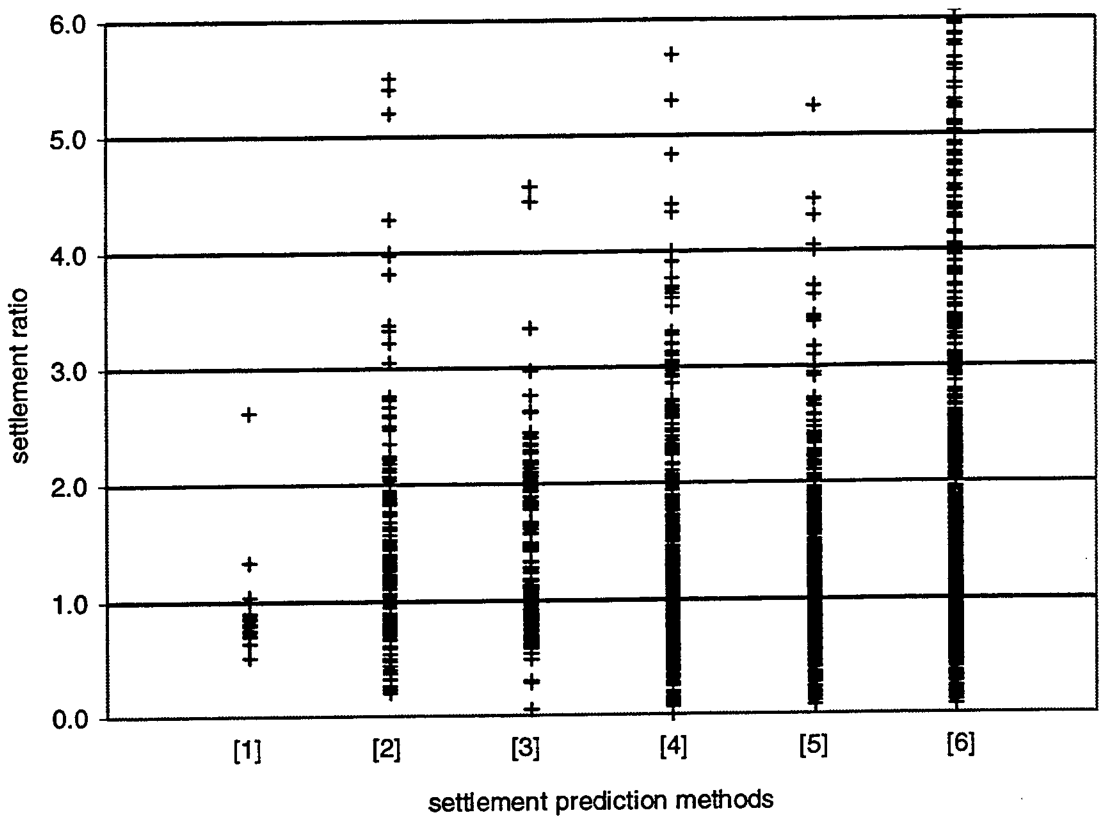
Figure 5-2 Performance of All Settlement Predictions



[1] Observational (23)
[2] Strain factor (144)
[3] Compressibility coefficient (152)
[4] Elasticity (440)
[5] Empirical without Alpan and Terzaghi & Peck (478)
[6] Empirical (764)

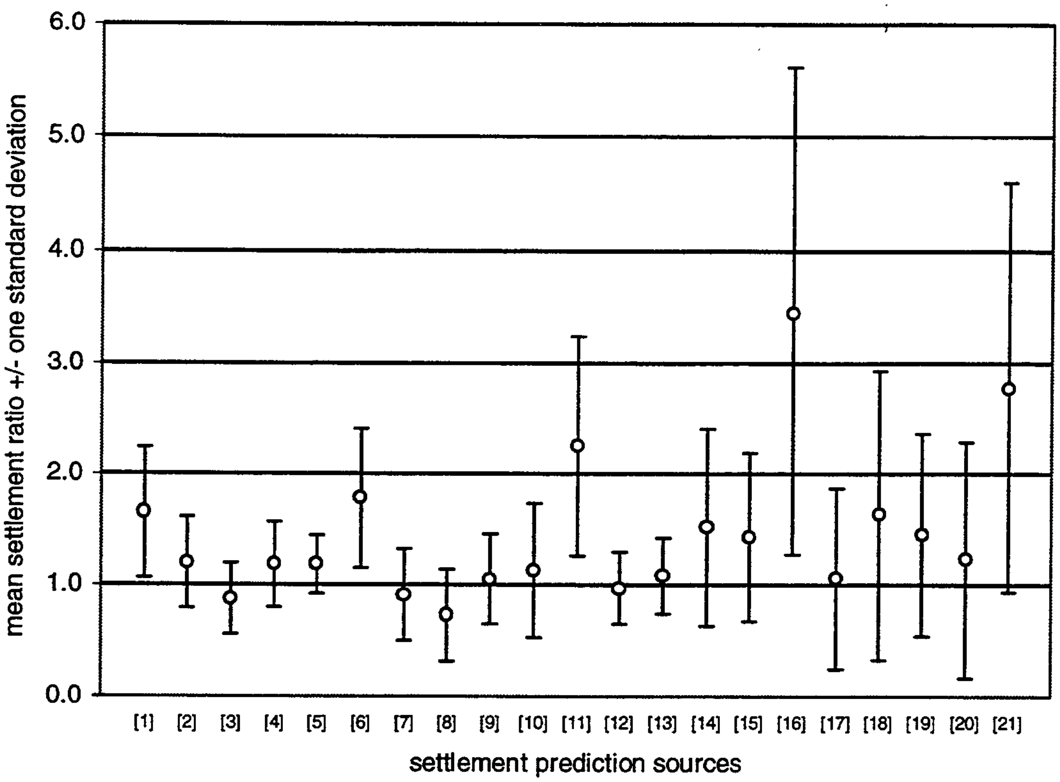
The number in parentheses behind each method is the number of points (i.e. calculated settlement ratios) used to determine the performance of each method.

Figure 5-3 Settlement Prediction - Average Performance of Methods



[1] Observational
[2] Strain factor
[3] Compressibiltiy coefficient
[4] Elasticity
[5] Empirical (without Alpan and Terzaghi & Peck)
[6] Empirical

Figure 5-4 Performance of Settlement Prediction Methods



[1] Oweis, 1979
[2] D'Appolonia, 1971
[3] Parry, 1977
[4] Schmertmann, 1986
[5] Skempton-Bjerrum, 1952
[6] Hough, 1959
[7] Asaoka, 1978
[8] Peck & Bazaraa, 1969
[9] Menard, 1975
[10] Wahls-Gupta, 1994
[11] Bowles, 1987
[12] Schultze & Sherif, 1973
[13] Papadopoulos, 1992
[14] Meyerhof, 1965
[15] Schmertmann, 1970
[16] Alpan, 1964
[17] Berardi & Lancellotta, 1994
[18] Buisman-DeBeer, 1957
[19] Burland & Burbidge, 1985
[20] D'Appolonia, 1970
[21] Terzaghi & Peck, 1948

Figure 5-5 Settlement Predictions for Footings

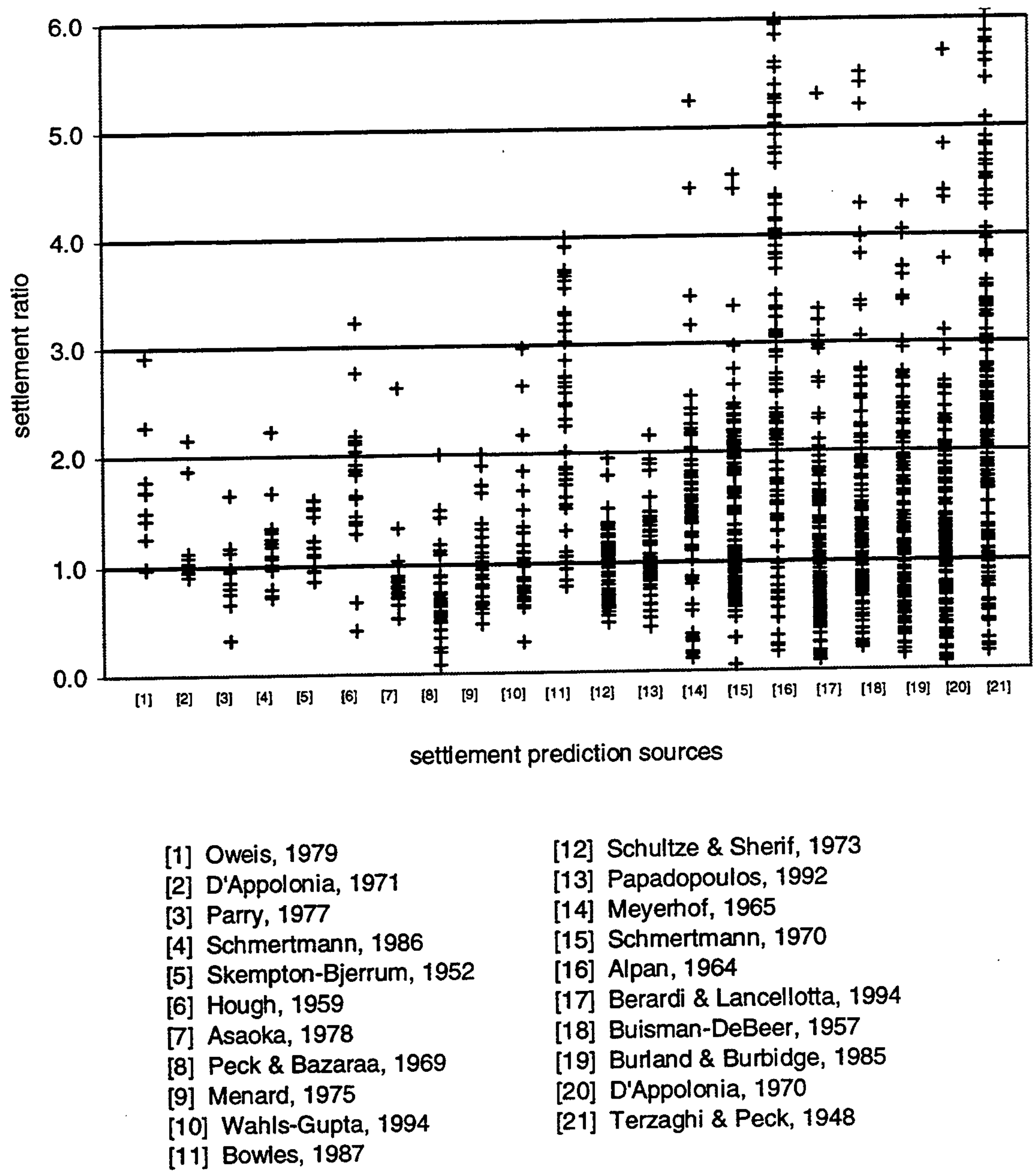
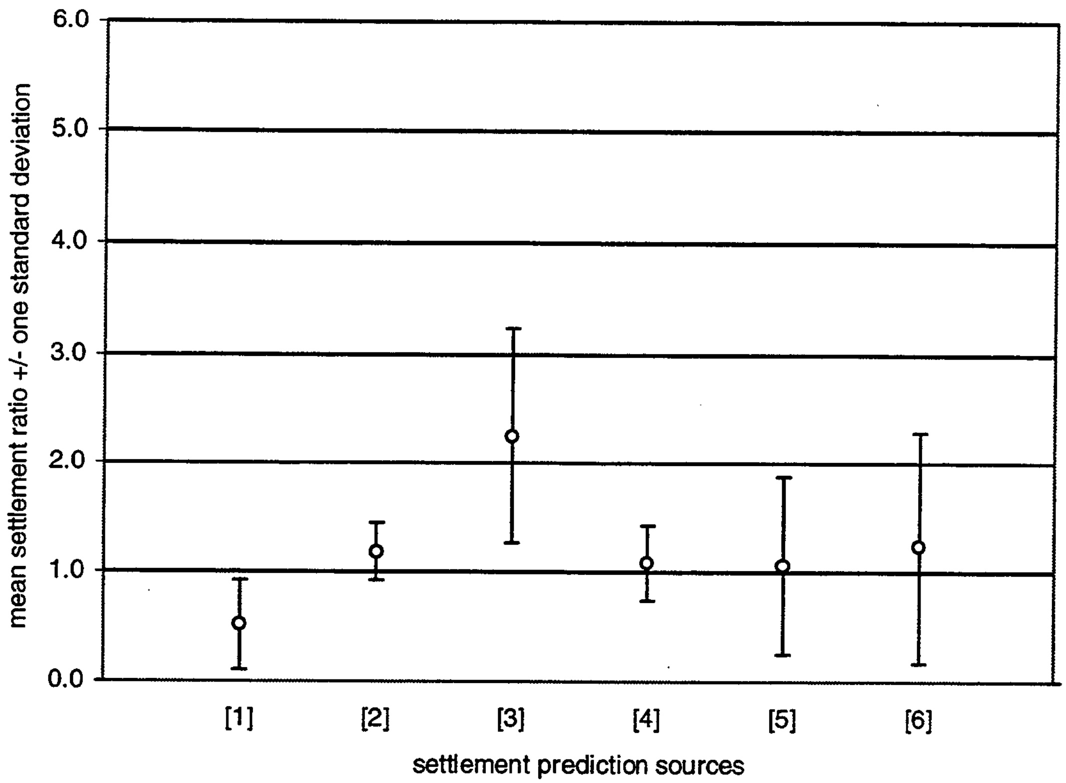


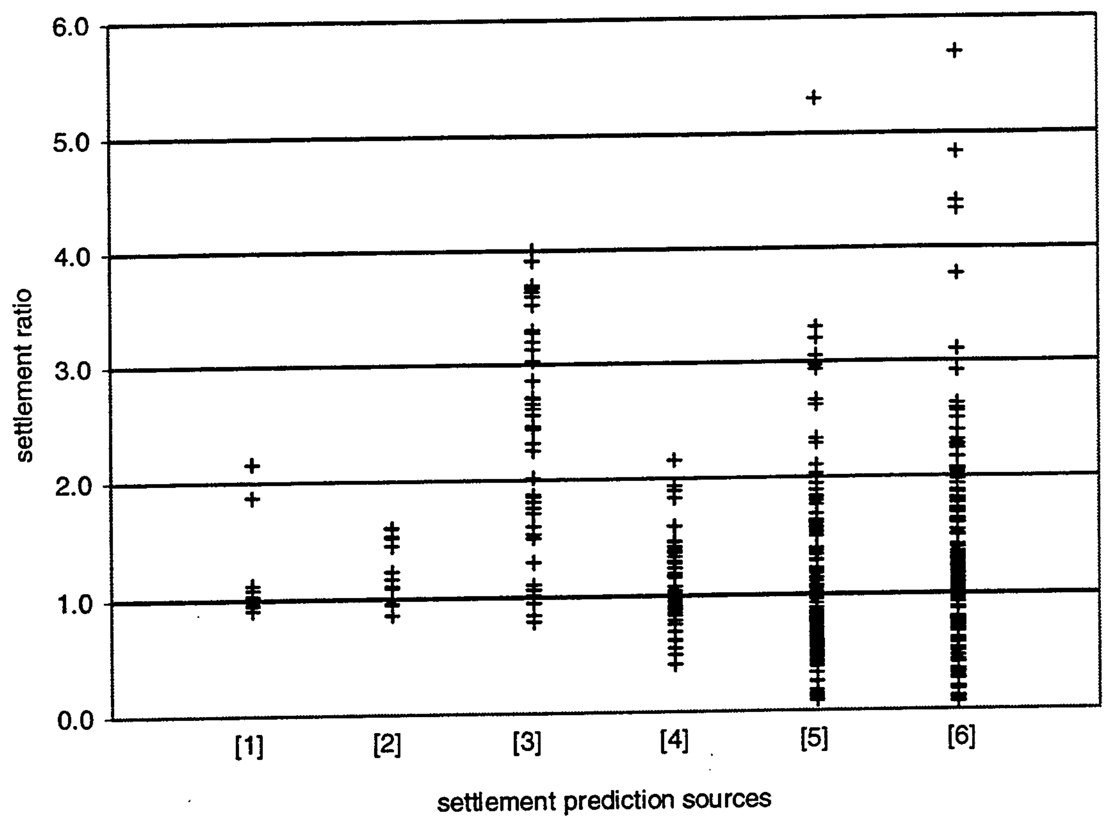
Figure 5-6 Performance of Settlement Predictions for Footings



[1] D'Appolonia, 1971
[2] Skempton-Bjerrum, 1952
[3] Bowles, 1987
[4] Papadopoulos, 1992
[5] Berardi & Lancellotta, 1994
[6] D'Appolonia, 1970

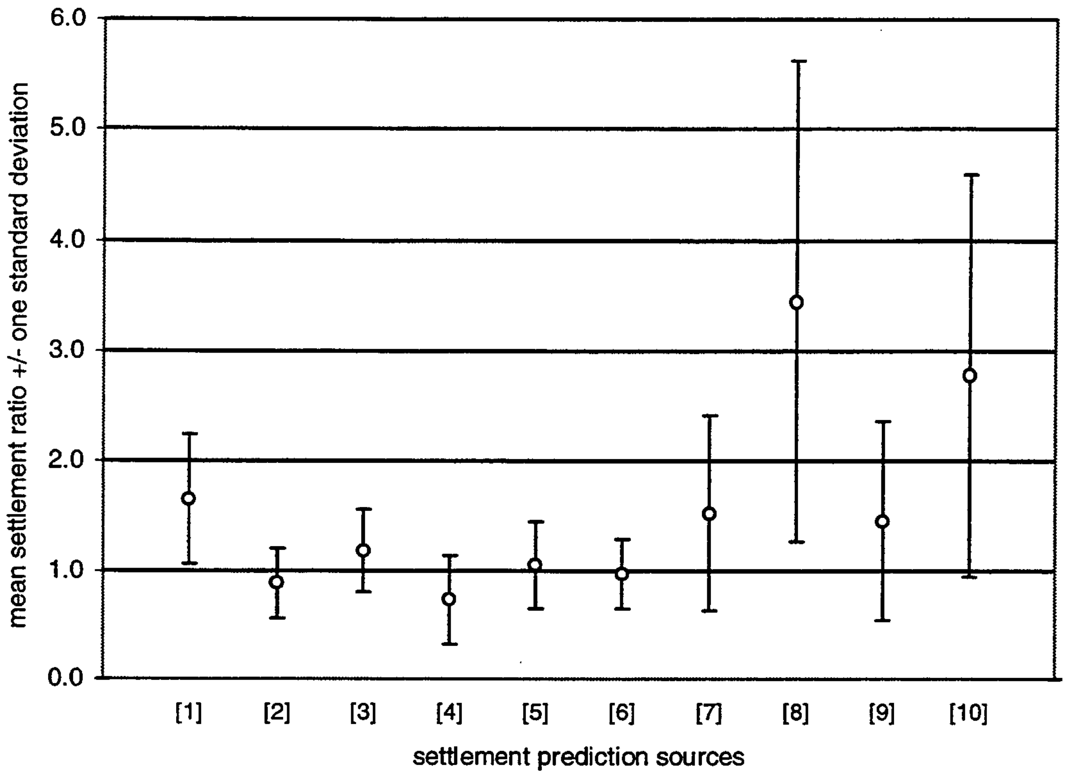
D'Appolonia techniques ([1], [6]) are for clays and sand, respectively.

Figure 5-7 Performance of Settlement Prediction for Footings - Elasticity Methods



[1] D'Appolonia, 1971
[2] Skempton-Bjerrum, 1952
[3] Bowles, 1987
[4] Papadopoulos, 1992
[5] Berardi & Lancellotta, 1994
[6] D'Appolonia, 1970

Figure 5-8 Performance of Settlement Predictions for Footings - Elasticity Methods



[1] Oweis, 1979
[2] Parry, 1977
[3] Schmertmann, 1986
[4] Peck & Bazaraa, 1969
[5] Menard, 1975
[6] Schultze & Sherif, 1973
[7] Meyerhof, 1965
[8] Alpan, 1964
[9] Burland & Burbidge, 1985
[10] Terzaghi & Peck, 1948

Figure 5-9 Performance of Settlement Prediction for Footings - Empirical Methods

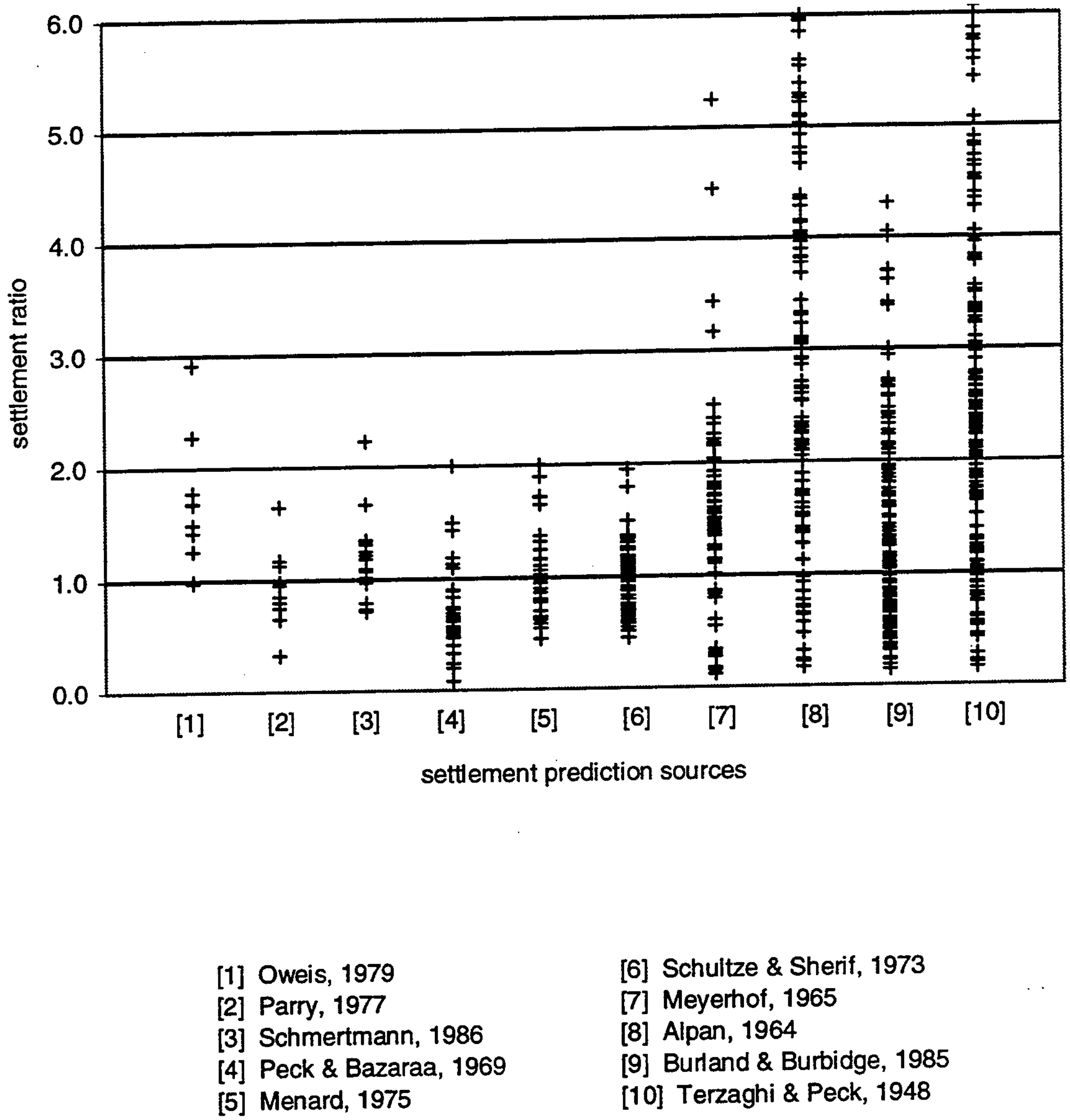
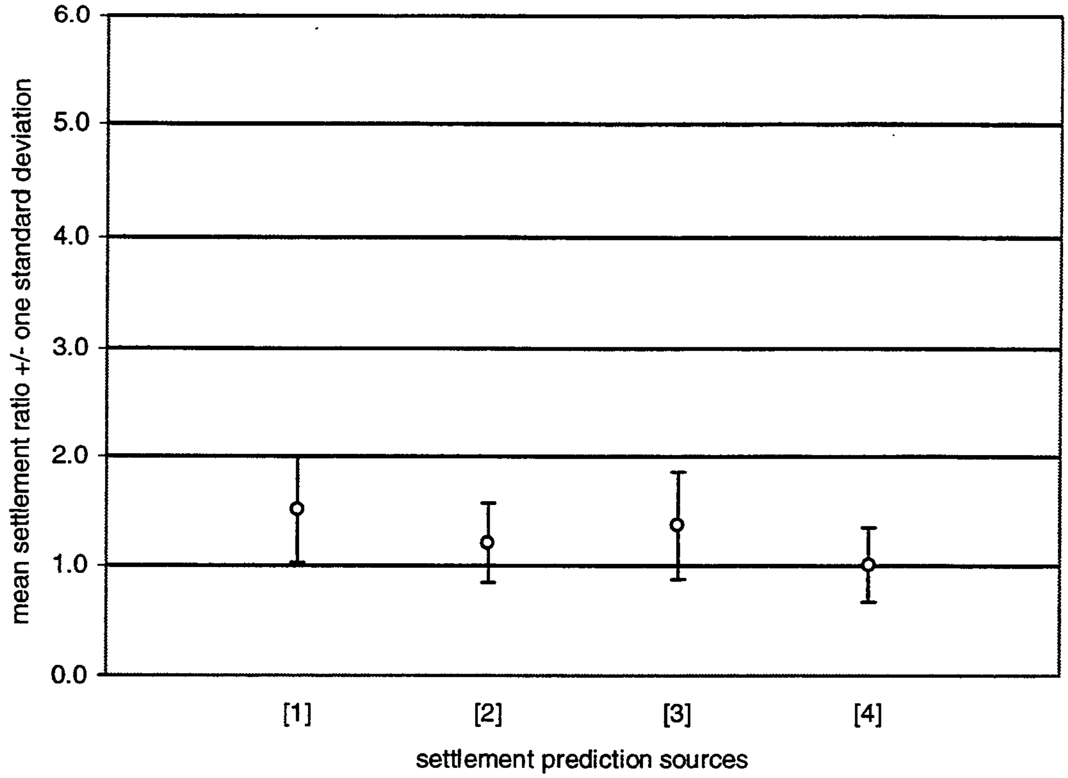


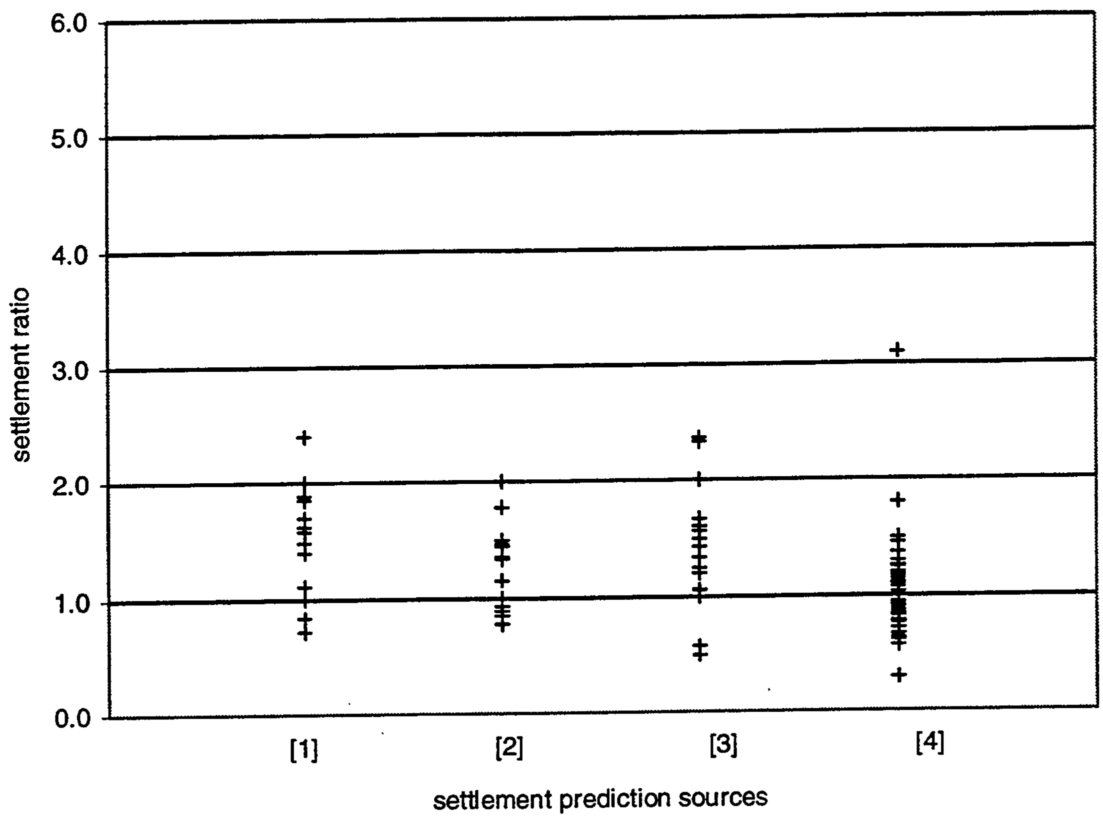
Figure 5-10 Performance of Settlement Predictions for Footings - Empirical Methods



[1] Meyerhof, 1976
[2] Meyerhof, 1976
[3] Yamashita, 1987
[4] Bazaraa & Kurkur, 1986

Meyerhof techniques use CPT ([1]) or SPT values ([2]).
The equations for each technique also differ.

Figure 5-11 Performance of Settlement Predictions for Piles



[1] Meyerhof, 1976
[2] Meyerhof, 1976
[3] Yamashita, 1987
[4] Bazaraa & Kurkur, 1986

Figure 5-12 Performance of Settlement Predictions for Piles

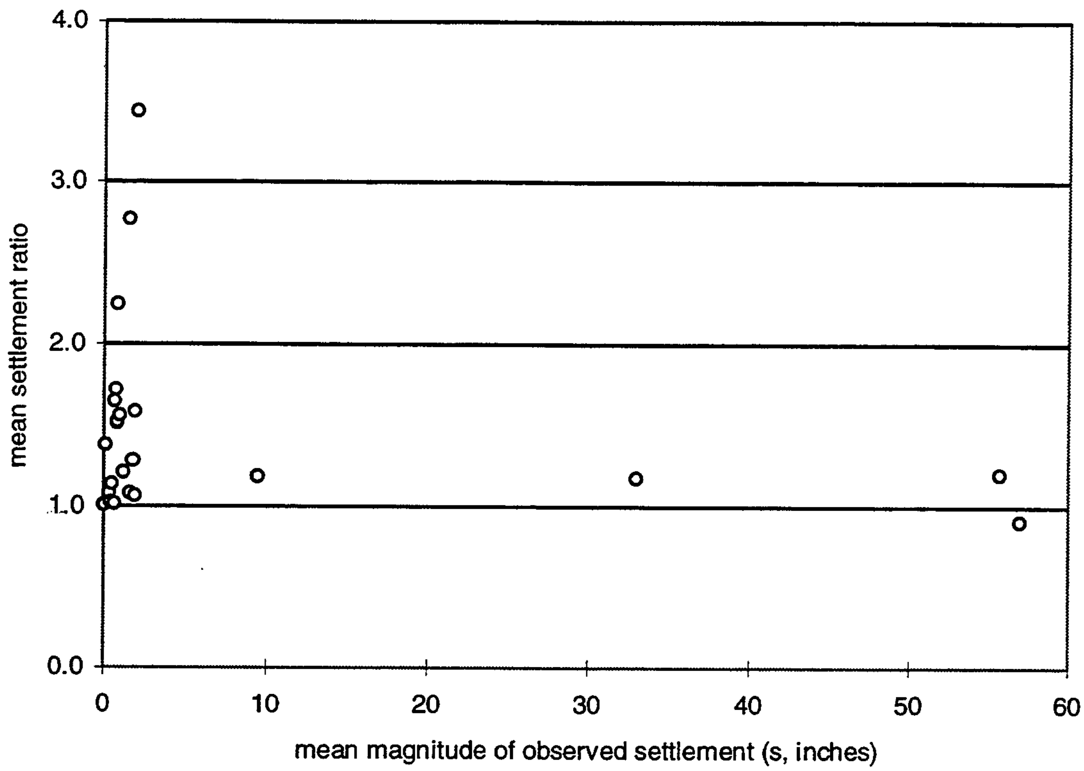


Figure 5-13 Performance of Settlement Prediction versus Settlement Magnitude

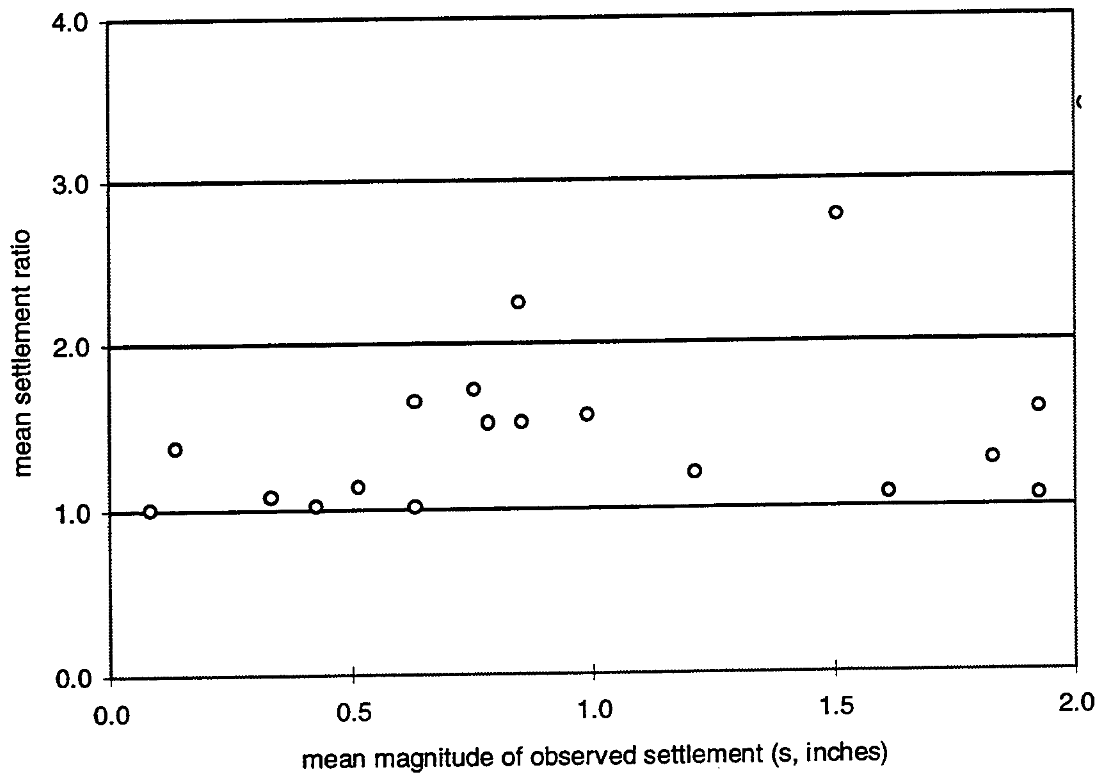


Figure 5-14 Performance of Settlement Predictions versus Settlement Magnitude

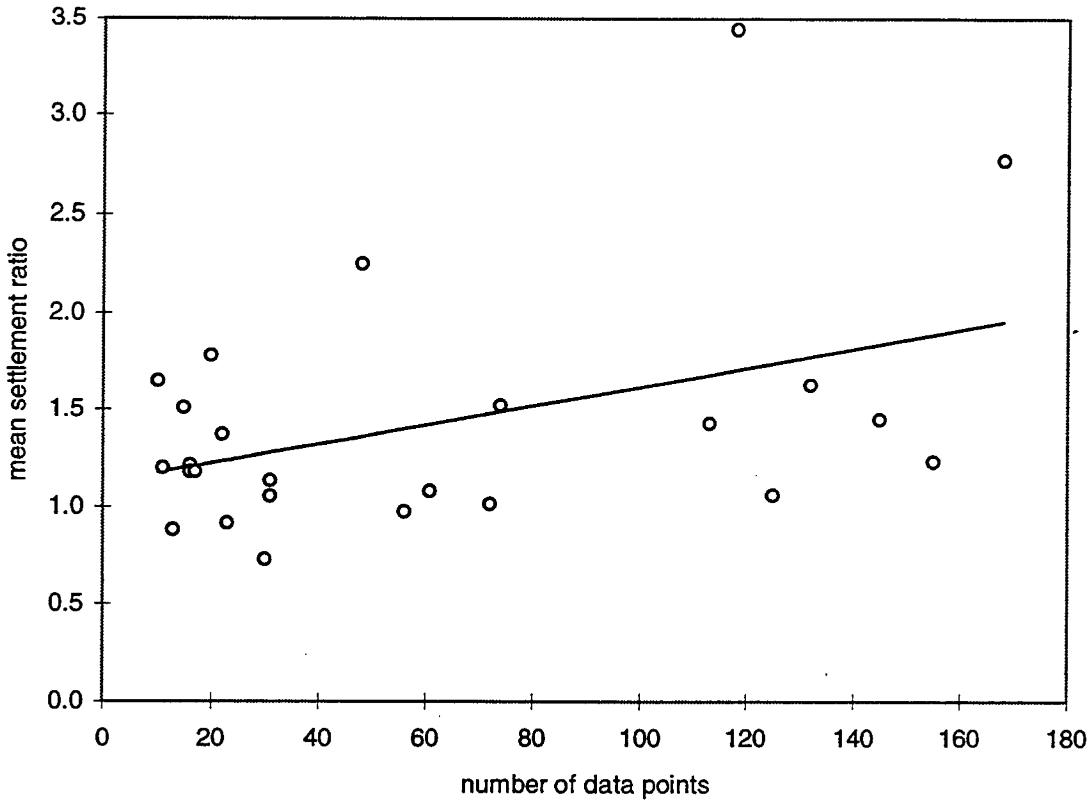


Figure 5-15 Performance of Settlement Prediction versus Number of Data Points

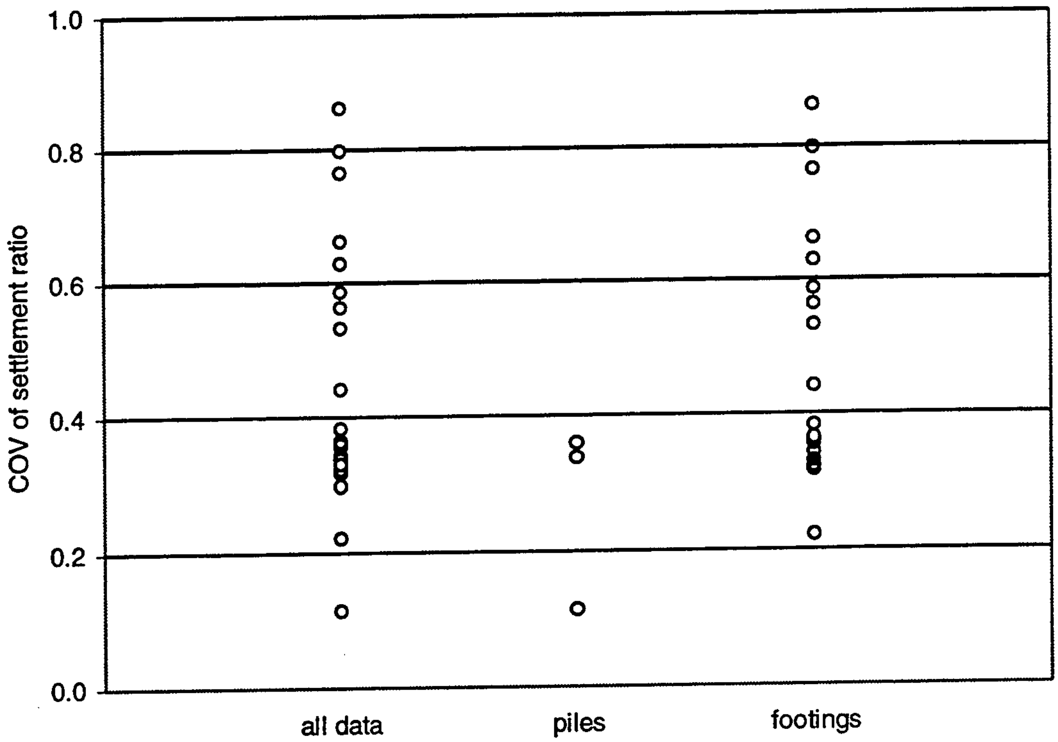


Figure 5-16 Coefficient of Variation for Settlement Predictions

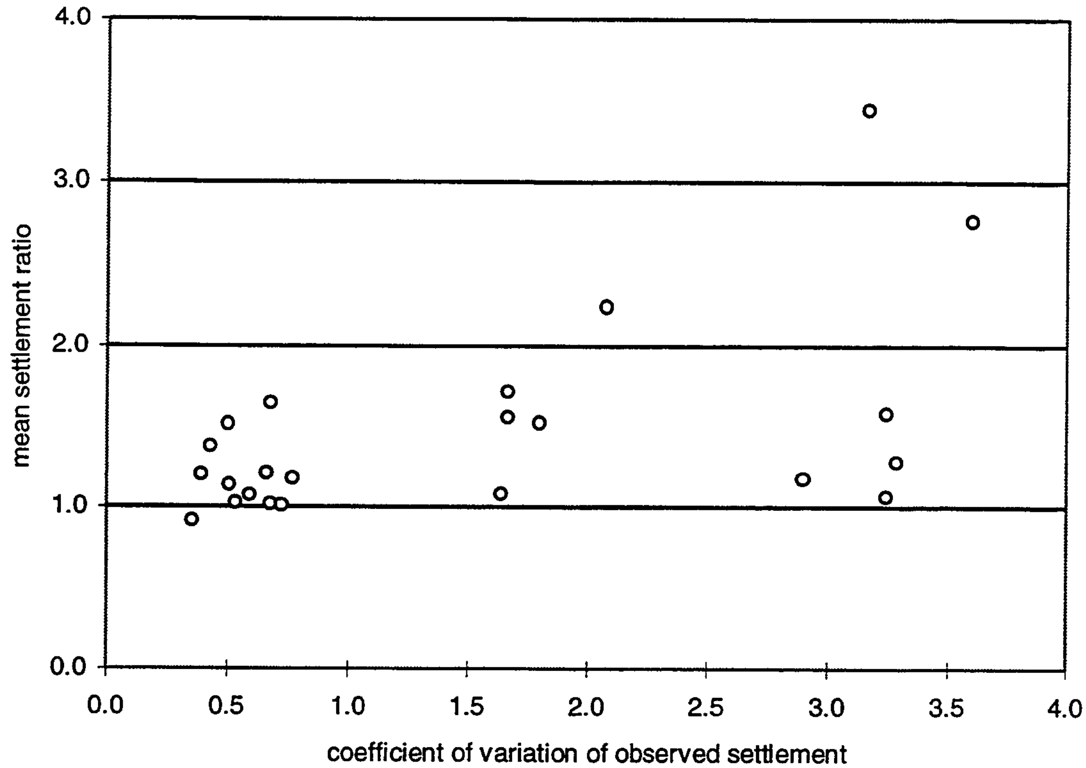


Figure 5-17 Performance of Settlement Predictions versus COV of Settlements

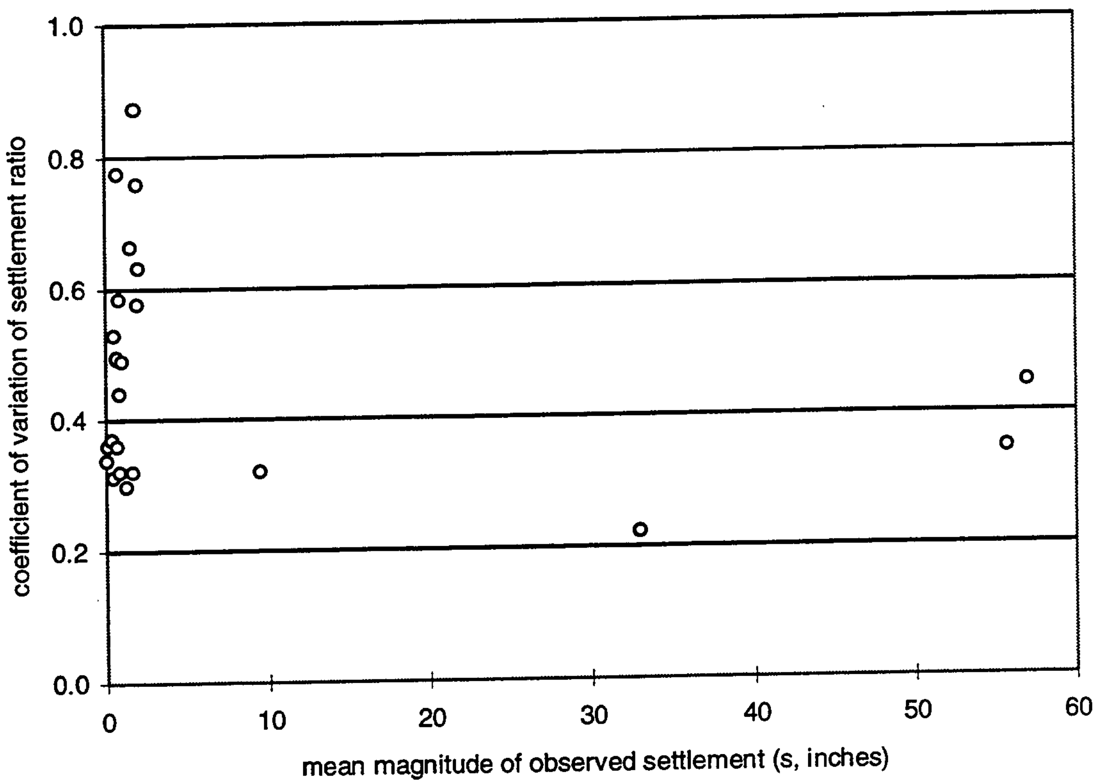


Figure 5-18 COV of Settlement Prediction versus Settlement Magnitude

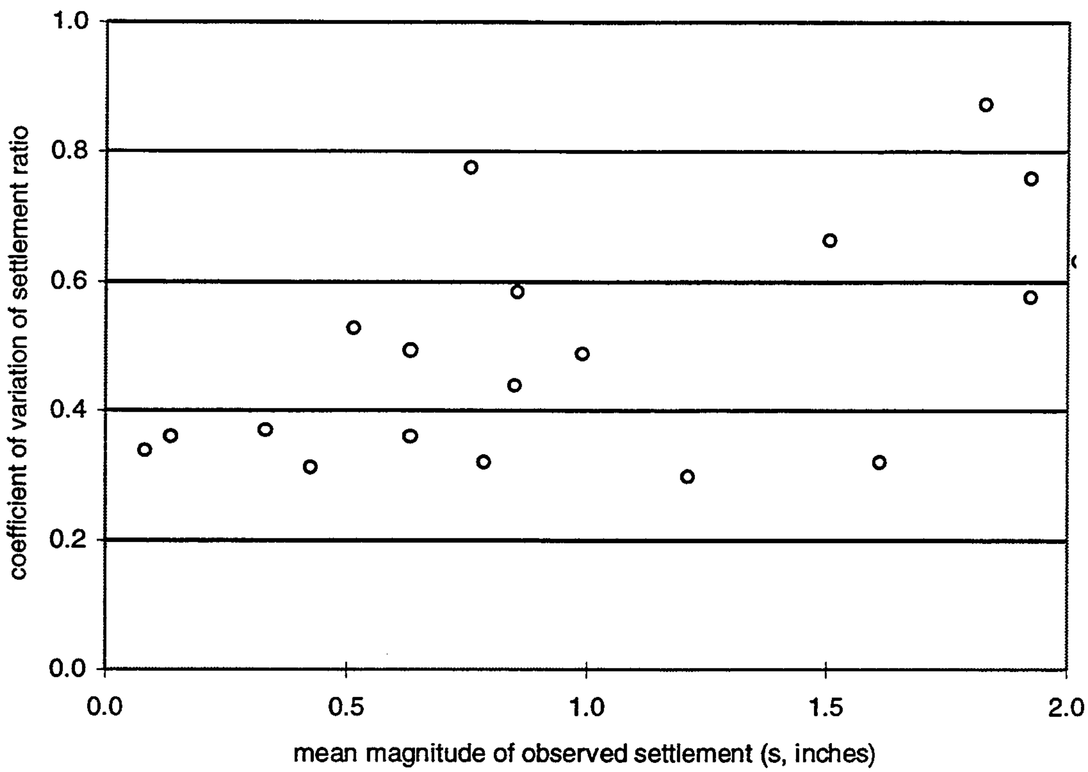


Figure 5-19 COV of Settlement Prediction versus Settlement Magnitude

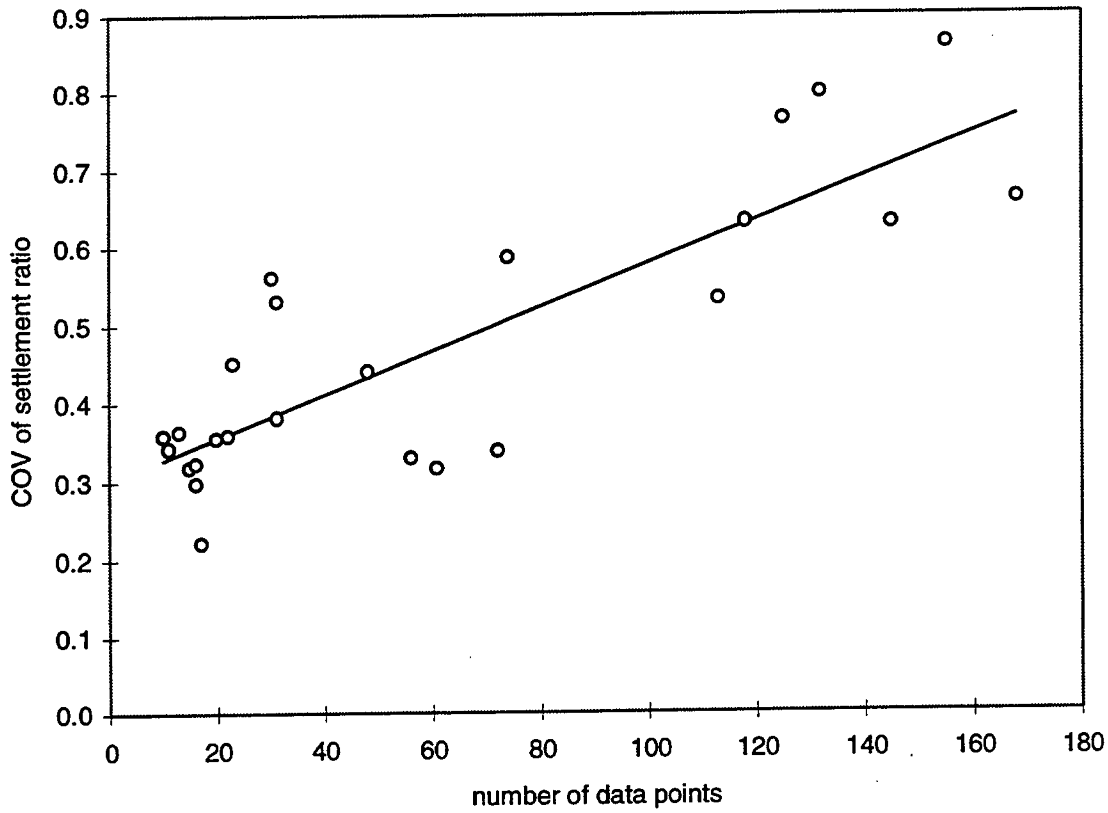


Figure 5-20 COV of Performance of Settlement Prediction versus Number of Data Points

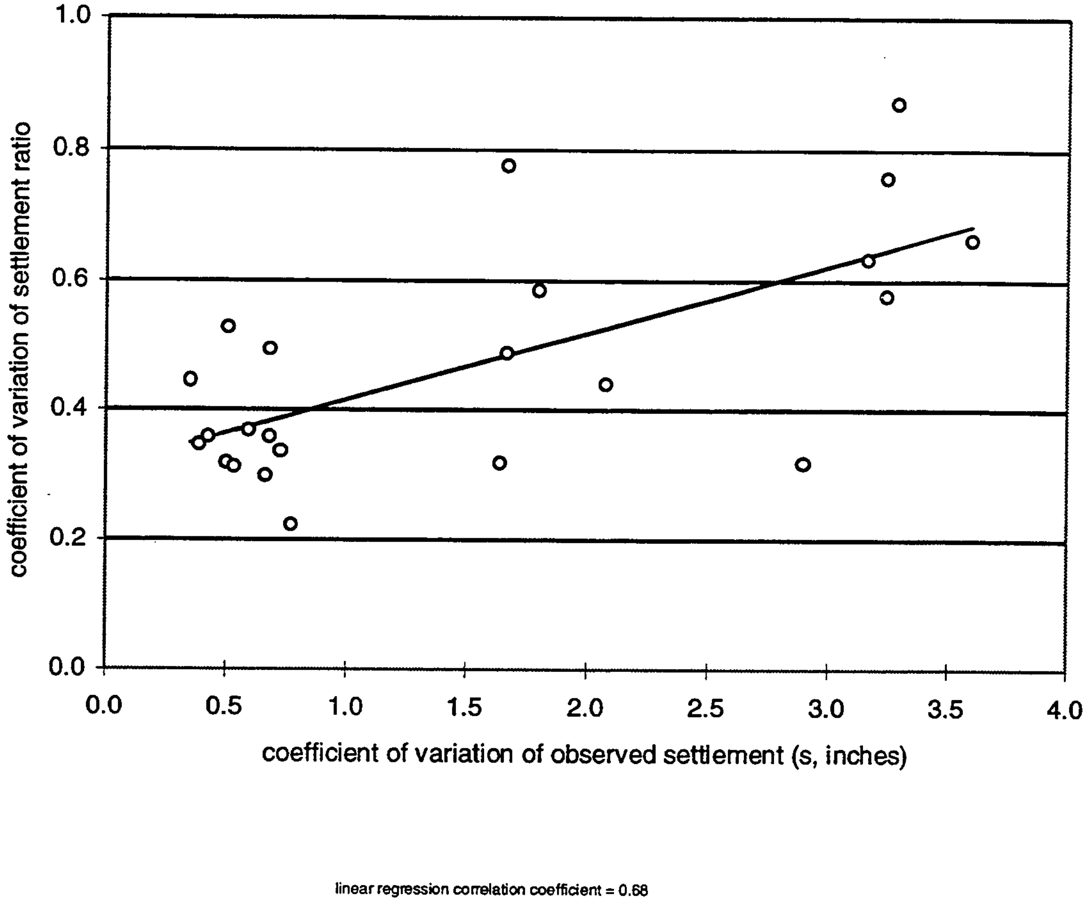


Figure 5-21 COV of Settlement Prediction versus COV of Settlements

Section 6 DIFFERENTIAL SETTLEMENTS IN BRIDGES

Differential settlements cause pavement faults. Differences in total settlements of approaches and abutments are sensed as bumps by traffic. Large differences in settlements are a problem. Large total settlements, if equal for approaches and for bridge substructures, are not a problem. There are two implications of these statements. First, the computation of expected pavement faults for a bridge is necessarily the computation of expected differential settlements. Second, pavement faults are mitigated by reducing differential settlements. Total settlements might remain large. In particular, abutments can be supported on spread footings on the compacted embankment fill to mitigate pavement faulting. For such mitigation, the tolerance of the bridge superstructure for settlements of the abutment becomes the main concern in design. In addition, the design checks for the superstructure must use rational values of expected differential settlements among bridge substructures. These differential settlements not necessarily a fixed fraction of total settlements, but instead may be related to total settlements, variance in total settlements and spatial correlation of total settlements.

In this section, a new method for the estimation of differential settlements is developed. It is found that differential settlements are strongly correlated with the variance in total settlements. Mean values of differential settlement can be computed, and bounds on differential settlement for any level of probability can be identified. Relations for expected values of differential settlement and for bounds on differential settlements are developed.

This section goes on to examine spatial correlation in total settlements and uses field data from several projects to form spatial correlation functions. Spatial correlation in total settlements shows that adjacent foundations will settle in a similar manner and therefore will exhibit relatively low values of differential settlement. Foundations that are farther apart are more likely to settle differently, and to have larger difference in settlements.

DIFFERENTIAL SETTLEMENTS

Statistical predictions of differential settlements and the development of functions of spatial correlation in settlements are developed from data on total settlement of bridges in service. The data on differential settlements of bridges are sparse. Limited data are available as summaries. More data are available on total settlements. Moulton [et al. 1985] provided a summary of differential settlements for bridges. Data in Moulton indicate a median value of differential settlement of 1.3 inches. This value is equal to the median value of total settlement of abutments on piles, and differs by 0.1 inches from the median total settlement of all bridge substructures. Moulton finds that total settlements and differential settlements are equivalent. This equivalence is possible if non-zero settlements occur for one substructure in each bridge. Earlier work, such as the TRB survey of settlements reported in 1978, dealt primarily with total settlements and implicitly relied on an equivalence of total settlements and differential settlements.

The study in this synthesis of relations between differential settlements and total settlements uses data on total settlements at two or more substructures of the same bridge. From such data, mean values and standard deviations of total settlements are compared to differences in total settlements. The comparison is basis of a proposed method for prediction of differential settlements.

THIRTY THREE HIGHWAY BRIDGES

Literature sources that offer data on total settlements at two or more bridge substructures are used for a study of relations between total settlements and differential settlements. Of the several hundred bridges reported in literature sources, many offer data on total settlements at a single foundation, or on differential settlements without separate data on total settlements. Only thirty three bridges are reported with data on total settlements of at least two substructures. Twenty one bridges (of thirty three) offer settlements at four or more substructures. The thirty three bridges include both steel and concrete superstructures and a range of spans from 53 feet to 775 feet. Most of the foundations are shallow foundations, but piles are represented in three of the bridges. The bridges are located in North America or in Europe. Studies of these bridges were conducted as many as fifty years ago and as recently as ten years ago. Information on the thirty three bridges is listed in Table 6-1. Also included in this study of differential settlements are two building projects. Both buildings are supported on footings, and both offer a large number of foundations with data on total settlements.

Settlement data for the thirty three bridges are shown in Table 6-2. The table shows the name of each bridge together with the number of foundations (**Fnds**) where total settlements are reported, the maximum total settlement (M_s), the mean total settlement (μ_s), standard deviation of total settlements (σ_s), coefficient variation (COV_s) of total settlements, and the maximum (M_D) and mean values (μ_D) of differential settlement.

Table 6-1 Summary of Bridges with Detailed Settlement Data

Name	Bridge Location/Description	Bridge ID	Built	Type	Spans	Foundations/soils
Afsnee [DeBeer, 1948]	Bridge over the Motorroad Brussels-Ostend at Afsnee	—	1941	Reinforced concrete continuous beams	3 spans total 175 ft	Shallow foundations in clayey sand
Baarlevelde St. [DeBeer & Martens, 1957]	Bridge over the Brussels-Ostend Motor Road in the 'Baarlevelde' Street at Drongen	LXXII	1954	—	—	Shallow foundations in sand
Branch Ave [Gifford, et al., 1987]	Branch Avenue, Northeast Corridor, Providence, RI	931	1984	—	4 @ 120 ft	Footings in medium-fine sand
Buckland St. [Gifford, et al., 1987]	CD-WB Roadway and Ramp 1 over Buckland St. (I-86), Manchester, CT	76-88-9	1984	Simple span	1 @ 146 ft	Footings in sand and some silt
Burlington Bay Skyway [Matich & Stermac, 1971]	Burlington Bay Skyway, Canada	—	1966	—	North side: 16 @ 53 ft 9 @ 94 ft 6 @ 179 ft 3 @ 217 ft 2 @ 275 ft Midspan 1 @ 500 ft South side same as North.	Piers on footings and piers on piles in compact and dense sand
Dickerman Road [Gifford, et al., 1987]	I-691 under relocated Dickerman Road, Southington/Cheshire, CT	131-132-11	1985	Simple span	1 @ 128 ft	Footings in fine sand and silt
Farmington River [Keene, 1976]	CT-185 over Farmington River, Simsbury, CT	—	1953	Simple span	1 @ 224 ft	Piles in silt and fine sand

Name	Bridge Location/Description	Bridge ID	Built	Type	Spans	Foundations/soils
Folly Brook [Keene, 1976]	CT-15 Expressway over Folly Brook Boulevard, Wethersfield, CT	—	1941	—	2 spans	Abutments on shallow foundations in clay, pier on piles
Gavere [Mariovet, 1953]	Brussels-Ostend Highway over chaussée de Gavere at Merelbeke	18	1952	Simple span	1 @ 44 ft	Shallow foundations in sand
Gersoni Road [Gifford, et al., 1987]	Relocated Gersoni Road. Route 28 over D&H Railroad and Route 7, Colliersville, NY	5	1984	—	2 @ 112 ft	Footings in coarse-fine sand and silty sand
Gentbrugge [DeBeer, 1948]	Bridge over the railway around Ghent in the Highway Ghent-Brussels at Gent-brugge	—	1940	2 continuous spans 1 simple span concrete	1 @ 39 ft 1 @ 78 ft 1 @ 78 ft	Shallow foundations in sand
Ghent-Kortruk Rd [Mariovet, 1953] [DeBeer & Martens, 1957]	Bridge on the Brussels-Ostend Motor Road over the Ghent-Kortruk Road at St. Denys-Westrem	14	1951	Simple span	1 @ 113 ft	Shallow foundations in sand
Highway No. 70 [DeBeer, 1948]	Bridge over the Motorroad Brussels-Ostend in the Highway No. 70 at Beernem	XLIII	1941	Concrete	2 @ 43 ft	Shallow foundations in sand
Huey P. Long [Kimball, 1940]	Huey P. Long Bridge over the Missis-sippi River, New Orleans	—	1935	—	1 @ 775 1 @ 525 1 @ 500 1 @ 325 (other spans not reported.)	Piers on caissons in clay
Hundelgem [Mariovet, 1953]	Brussels-Ostend Highway over chaussée d'Hundelgem at Merelbeke	19	1951	Simple span	1 — 55 ft	Shallow foundations in sand

Name	Bridge Location/Description	Bridge ID	Built	Type	Spans	Foundations/soils
Keuze Street [DeBeer, 1948] [DeBeer & Martens, 1957]	Bridge over the Motorroad Brussels-Ostend in the 'Keuze' Street at Drongen	LXXV	1942	Concrete	3 spans total 172 ft	Shallow foundations in sand
Kluizestraat [DeBeer, 1957]	Brussels-Ostend Highway over Kluizestraat at Oordegem	25	1951	Simple span	1 @ 45 ft	Shallow foundations in sand
Kortruk-Ghent Railroad [DeBeer, 1948] [Mariovet, 1953]	Bridge over the Railroad Kortruk-Ghent (Gand-Courtrai) in the Motorroad Brussels-Ostend at St. Denys-Westrem	13	1951	Concrete	3 @ 37 ft	Shallow foundations in sand
Lackey Dam Road [Gifford, et al., 1987]	Route 145 Southbound over relocated Lackey Dam Road, Uxbridge, MA	U-2-39	1984	Simple span	1 — 112 ft	Footings in coarse to fine sand
Loppem [DeBeer 1948] [Mariovet, 1953] [DeBeer & Martens, 1957]	Provincial route Bruges-Torhout over the Brussels-Ostend Highway at Loppem	XXIX	1950	Concrete	2 @ 43 ft	Shallow foundations on fine sand
Manchester Bridge 7 [Gifford, et al., 1987]	I-86, Manchester, CT	76-88-7	1984	—	1 @ 114 ft 1 @ 132 ft 1 @ 162 ft 1 @ 174 ft	Footings on coarse to fine sand
Maria-Aalter [DeBeer & Martens, 1957]	Bridge over the Brussels-Ostend Motor Road in the Knesselare-Ruiselede at Maria-Aalter	IL	1954	Simple span	1 @ 85 ft	Shallow foundations on fine sand
North Ave Sideline [Gifford, et al., 1987]	North Avenue Sideline over VT 127, Burlington, VT	M5000	1985	Simple span	1 @ 128 ft	Footings on medium-fine sand

Name	Bridge Location/Description	Bridge ID	Built	Type	Spans	Foundations/soils
Route Gand-Charleroi [Mariovet, 1953]	Brussels-Ostend Highway over chemin de fer Gand-Charleroi at Gonrtode	22	1952	Simple span	1 @ 78 ft	Shallow foundations on sand
Route Gand-Grammont [Mariovet, 1953]	Brussels-Ostend Highway over route Gand-Grammont at Gonrtode	21	1950	Simple span	1 @ 66 ft	Shallow foundations on sand
Route Oombergen-Wetteren [Mariovet, 1953]	Brussels-Ostend Highway over route Oombergen-Wetteren at Westrem	24	1951	Simple span	1 @ 66 ft	Shallow foundations on sand
Silas Deane [Keene, 1976]	I-91 northbound over Silas Deane Highway	—	1961	—	—	—
Silver Lane [Keene, 1976]	CT-15 Expressway over Silver Lane, East Hartford, CT	—	1948	Simple span	—	Shallow foundations on clay
Sterrestreet [DeBeer, 1948]	Bridge over the Motorroad Brussels-Ostend in the Sterrestreet at Aalter	—	1941	Concrete	1 @ 66 ft 1 @ 92 ft 1 @ 66 ft	Shallow foundations on clayey sand
Tolland Turnpike [Gifford, et al., 1987]	I-86 and CD Roadway under Tolland Turnpike, Manchester, CT	76-88-8	1984	—	1 @ 145 ft 1 @ 220 ft 1 @ 175 ft	Footings on coarse to fine sand
Wellingstreet [DeBeer, 1948]	Bridge over the Motorroad Brussels-Ostend in the Wellingstreet at Beernem	—	1943	Concrete	2 @ 52 ft	Shallow foundations on sand
Williams River [Gifford, et al., 1987]	VT Route 11 over the Middle Branch of the Williams River, Chester, Vermont	45	1983	Simple span	1 @ 115 ft	Footings on silty sand and silt
Willow Brook [Keene, 1976]	CT-2 Expressway over Willow St. Extension and Willow Brook, East Hartford, CT	—	1960	—	—	—

The distribution of total settlements for the 33 bridges are shown in Figure 6-22. Most of the settlements are less than 2 inches. The set of settlements at 6 inches are mostly from piers on footings of the Burlington Bay Skyway. The distribution of total settlements for these 33 bridges are similar to total settlements of the full set of data discussed earlier. A comparison of total settlements for 33 bridges, and for the full data set of several hundred bridges is shown in Figure 6-23. The two data sets are similar, and therefore the relations between total settlements and differential settlements that are valid for the set of 33 bridges may be valid for bridges in general.

The distribution of mean differential settlements of the 33 bridges is shown in Figure 6-24. Figure 6-25 shows the distribution of differential settlements for bridges reported by Moulton [et al. 1985]. Moulton's data are generally higher, but it is likely that he has reported maximum, not mean, differential settlements. In Figure 6-26 and Figure 6-27, mean and maximum differential settlements for 33 bridges are normalized against total settlements. The distribution of maximum differential settlements for 33 bridges is much like the distribution of differential settlements reported by Moulton [et al. 1985].

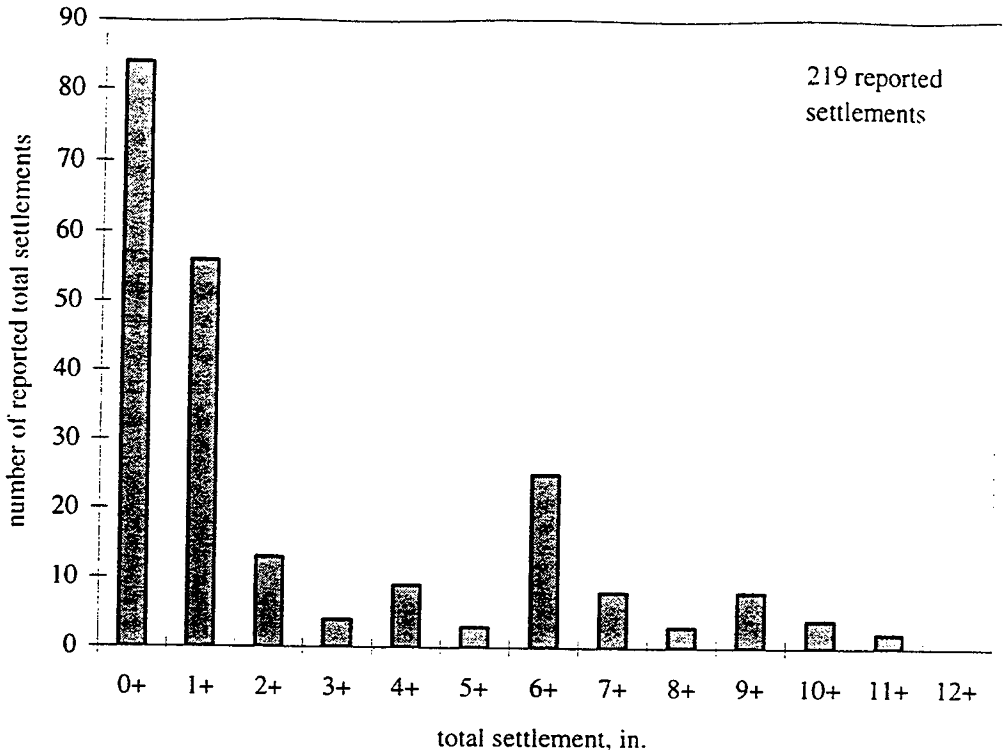


Figure 6-22 Mean Total Settlements for 33 Bridges

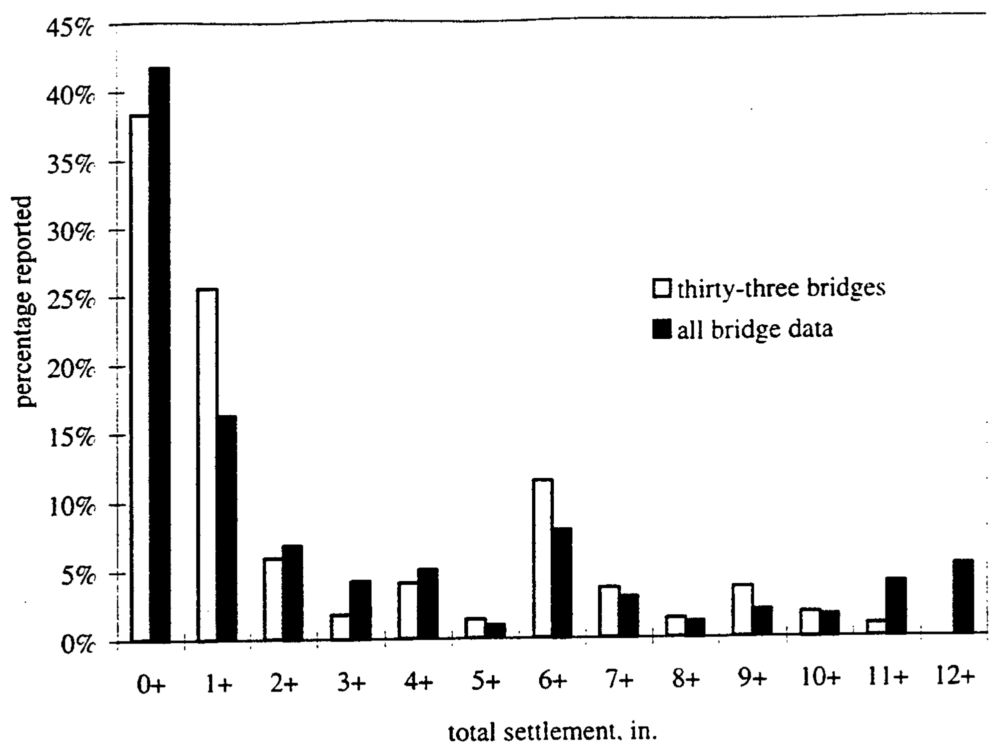


Figure 6-23 Comparison of 33 Bridges with Full Data Set

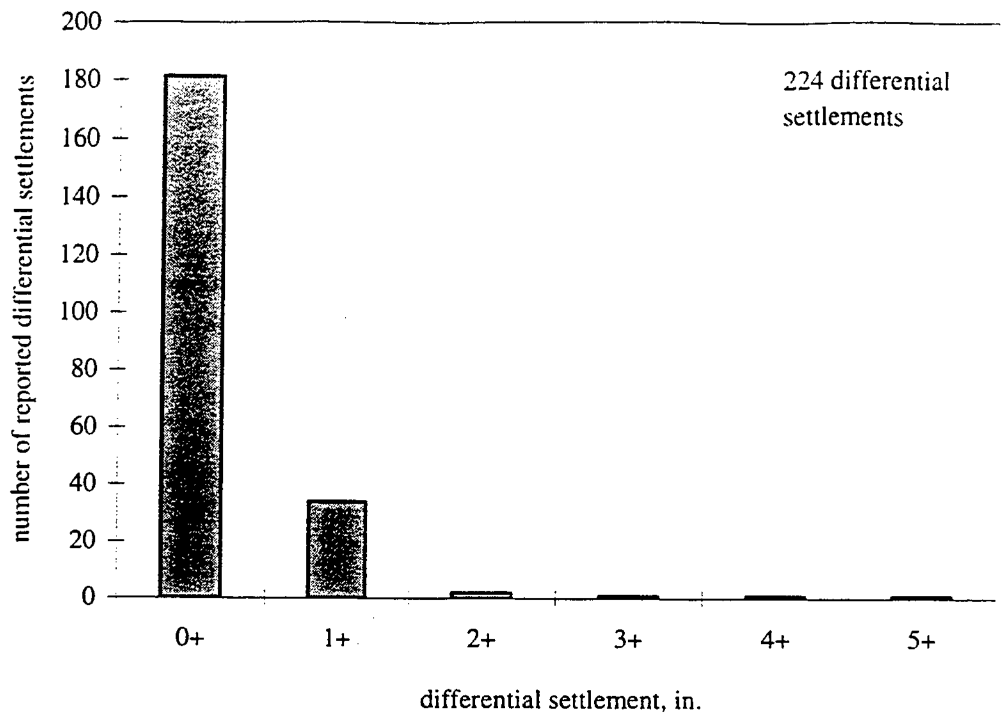
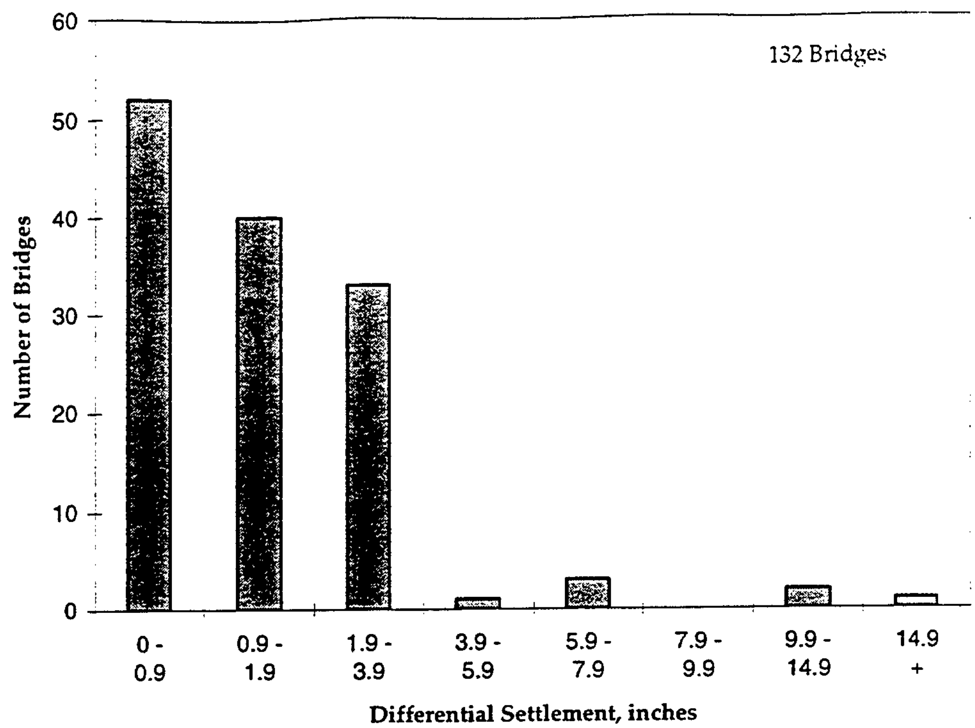


Figure 6-24 Differential Settlements for 33 Bridges



mean differential settlement = 1.8 inches
median differential settlement = 1.3 inches

Figure 6-25 Differential Settlements Reported in Moulton [1985]

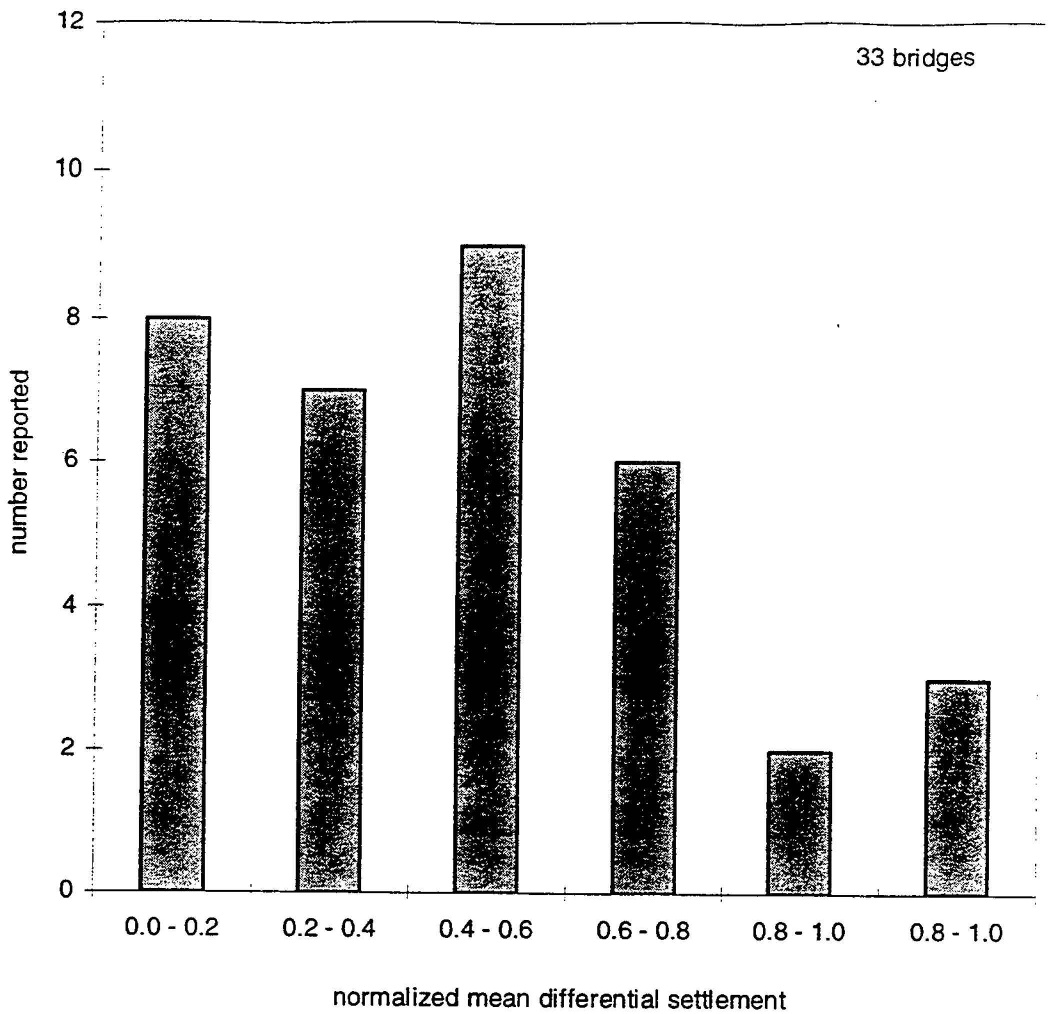


Figure 6-26 Normalized Mean Differential Settlement

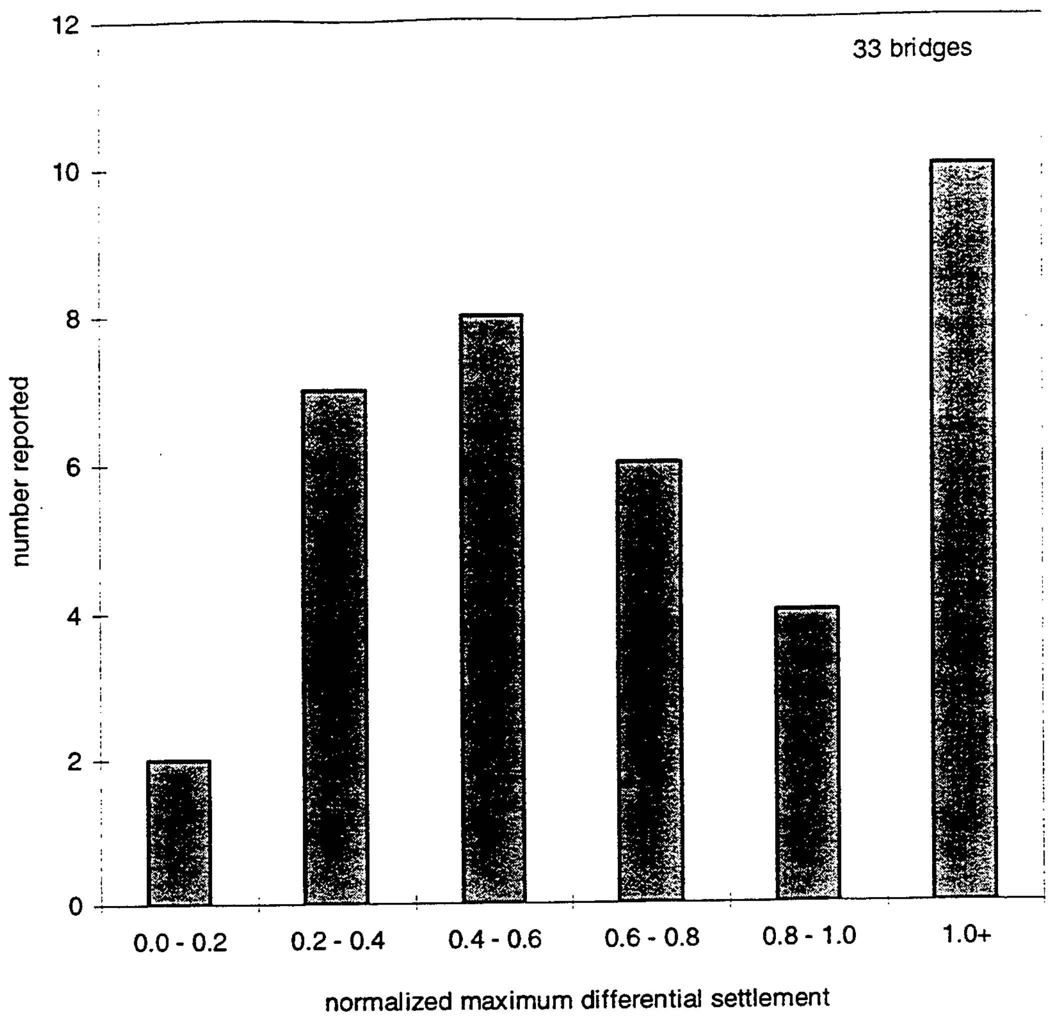


Figure 6-27 Normalized Maximum Differential Settlements

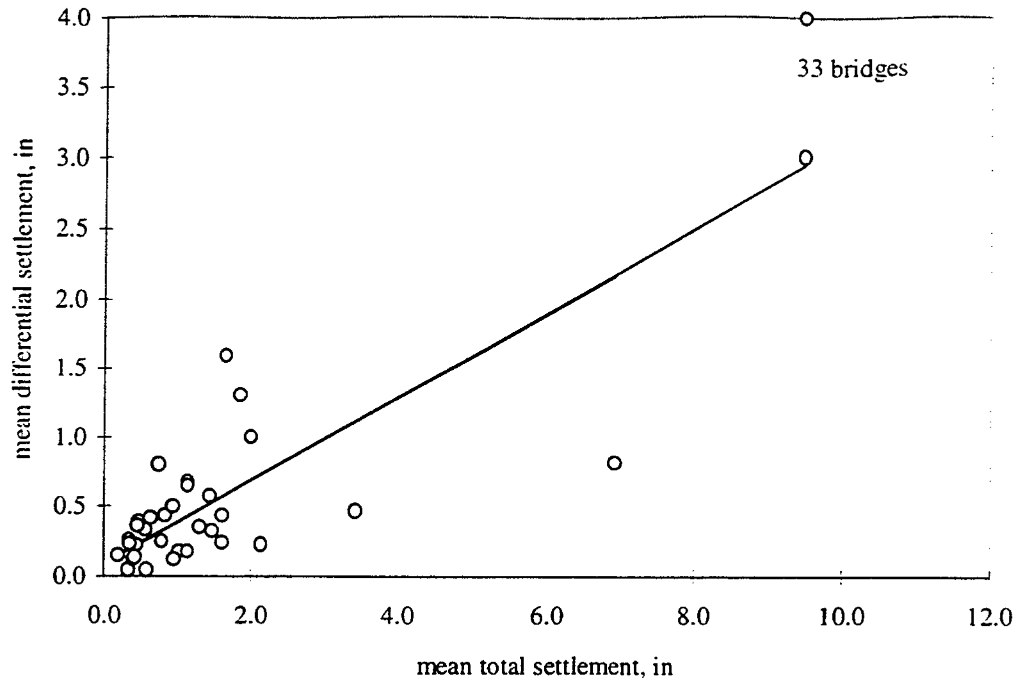
PREDICTION OF DIFFERENTIAL SETTLEMENTS

Differential settlements may be estimated as differences in expected total settlements. If soil conditions differ among foundations, or foundation type differs, then the predicted values of total settlements may be different and this difference is a prediction of differential settlements. If foundations and soils are similar, then the expected values of total settlements are equal and no difference in settlements is expected. Here, the classical approach is an estimate of differential settlements as a fraction of total settlements. Commonly used estimates for differential settlements are 50% of total settlements for similar foundations and 75% of total settlements for dissimilar foundations.

Figure 6-28 shows a plot of mean differential settlements versus mean total settlements for the 33 bridges. Figure 6-29 is a similar plot for maximum differential settlements. Both plots show an apparent correlation between differential settlement and total settlements. This is the classical idea, though these data indicate that differential settlements are about 35% of total settlements. In both plots, the relations between differential settlements and total settlements are determined by a few points at large values of settlement. In Figure 6-30 and Figure 6-31 the relations are examined again excluding data at large settlements. For this subset of data, there is no apparent relation between differential settlements and total settlements. In Figure 6-32 normalized differential settlements are considered. For large total settlements, differential settlements are not greater than 50% of total settlements. For smaller total settlements, there is no consistent relation. Overall, an estimate of differential settlements as a fraction of total settlements may be useful to establish an upper bound on differential settlements for large total settlements, but it is not useful to establish a bound when total settlements are moderate, and in no range of total settlements can differential settlements be predicted from total settlements.

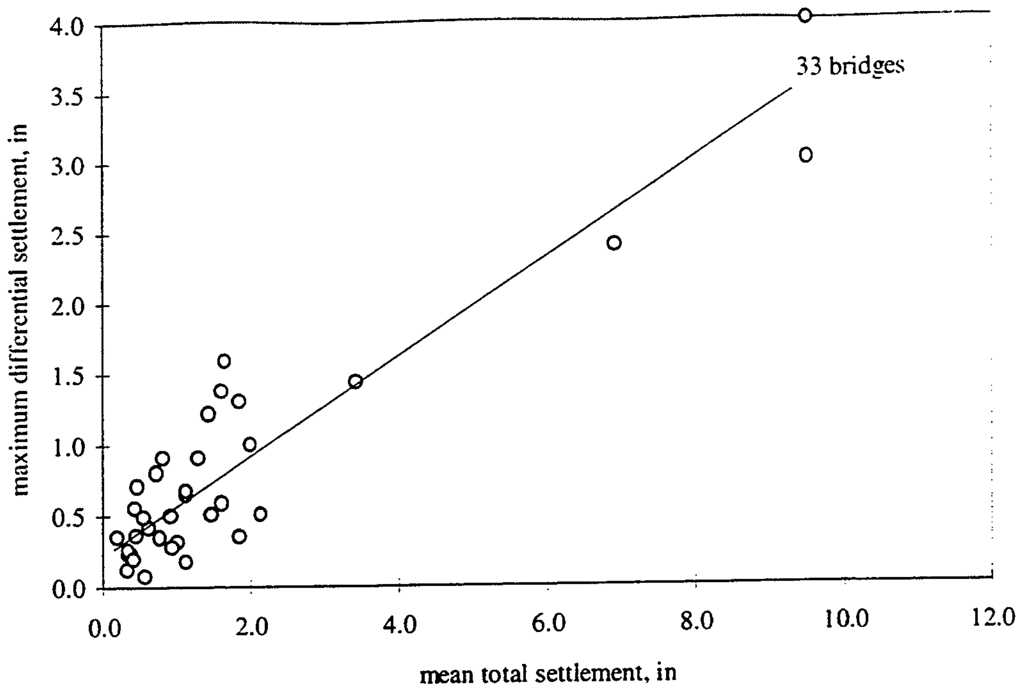
Bridge	Fnds	M_v in	μ_v in	σ_v in	COV_s	M_{Dv} in	μ_{Dv} in
Afsnee	4	1.97	1.86	0.17	0.09	0.35	0.18
Baarlevelde Street	4	0.63	0.58	0.04	0.06	0.08	0.04
Branch Ave	5	0.98	0.47	0.33	0.69	0.71	0.39
Buckland St.	2	0.64	0.46	0.25	0.55	0.36	0.36
Burlington Bay Skyway	58	10.20	6.92	1.67	0.24	2.40	0.81
Dickerman Road	3	0.97	0.79	0.18	0.22	0.35	0.25
Farmington River	2	0.47	0.35	0.17	0.47	0.24	0.24
Folly Brook	2	2.50	2.00	0.71	0.35	1.00	1.00
Gavere	4	0.55	0.40	0.11	0.28	0.24	0.12
Gentbrugge	12	1.34	0.82	0.38	0.46	0.91	0.43
Gersoni Road	2	1.14	0.74	0.57	0.76	0.80	0.80
Ghent-Kortruk Road	8	1.97	1.48	0.28	0.19	0.51	0.32
Highway No. 70	6	0.35	0.33	0.05	0.15	0.12	0.04
Huey P. Long	5	3.96	3.42	0.69	0.20	1.44	0.47
Hundelgem	4	1.22	1.01	0.14	0.14	0.31	0.18
Keuze Street	2	1.46	1.12	0.47	0.42	0.67	0.67
Kluizestraat	8	2.68	2.14	0.28	0.13	0.51	0.22
Kortruk-Ghent Railroad	12	2.36	1.61	0.58	0.36	1.38	0.44
Lackey Dam Road	2	0.48	0.35	0.18	0.53	0.26	0.26
Loppem	9	1.14	0.94	0.11	0.11	0.98	0.43
Manchester Bridge 7	2	0.84	0.63	0.30	0.47	0.42	0.42
Maria-Aalter	4	0.51	0.42	0.09	0.21	0.20	0.14
North Ave Sideline	2	1.17	0.92	0.35	0.38	0.50	0.50
Route Gand-Charleroi	8	2.68	1.38	0.60	0.43	1.22	0.57
Route Gand-Grammont	8	1.77	1.29	0.40	0.31	0.91	0.35
Route Oombergen-Wetteren	8	1.89	1.61	0.21	0.13	0.59	0.24
Silas Deane	2	2.50	1.85	0.92	0.50	1.30	1.30
Silver Lane	2	11.50	9.50	2.83	0.30	4.00	4.00
Sterrestreet	8	0.39	0.20	0.12	0.63	0.35	0.15
Tolland Turnpike	3	0.83	0.56	0.25	0.44	0.49	0.33
Wellingstreet	6	0.79	0.44	0.20	0.45	0.55	0.22
Williams River	2	2.45	1.66	1.12	0.68	1.59	1.59
Willow Brook.	2	11.00	9.50	2.12	0.22	3.00	3.00

Table 6-2 Total and differential settlement data for thirty-three bridges



$$\mu_D = 0.07 + 0.35\,\mu_s$$
$$r = 0.91$$
$$\sigma_{est} = 0.35$$

Figure 6-28 Mean Differential Settlement versus Mean Total Settlements - Bridges



$$M_D = 0.28 + 0.34\mu_S$$

$r = 0.92$

$s(Err) = 0.33$

Figure 6-29 Maximum Differential Settlement versus Mean Total Settlement

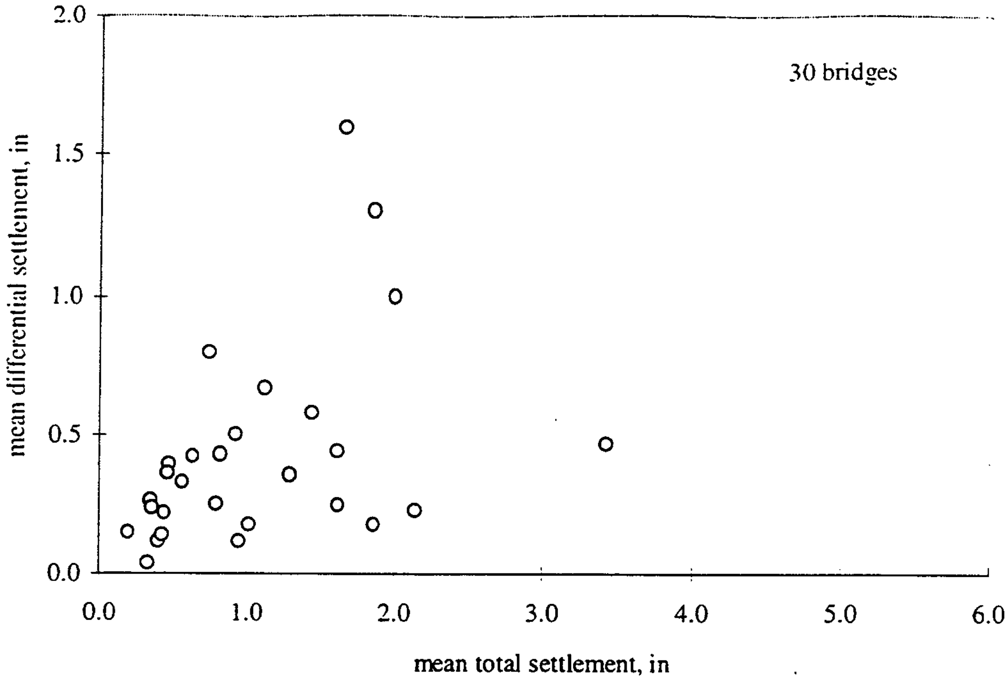


Figure 6-30 Mean Differential Settlement versus Mean Total Settlements - Bridges

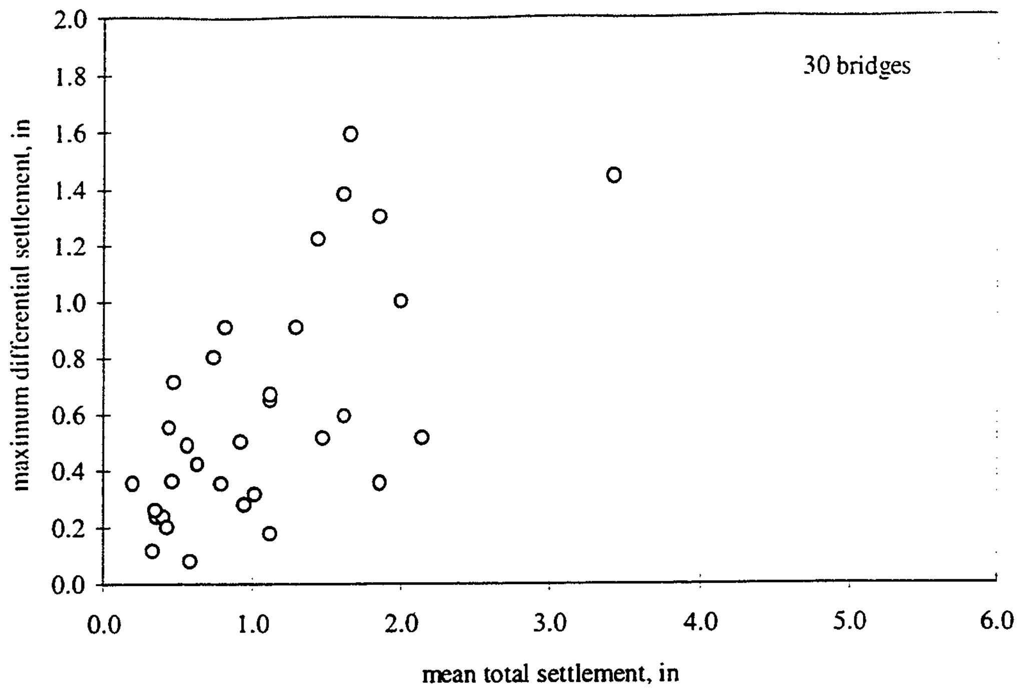


Figure 6-31 Maximum Differential Settlement versus Mean Total Settlement

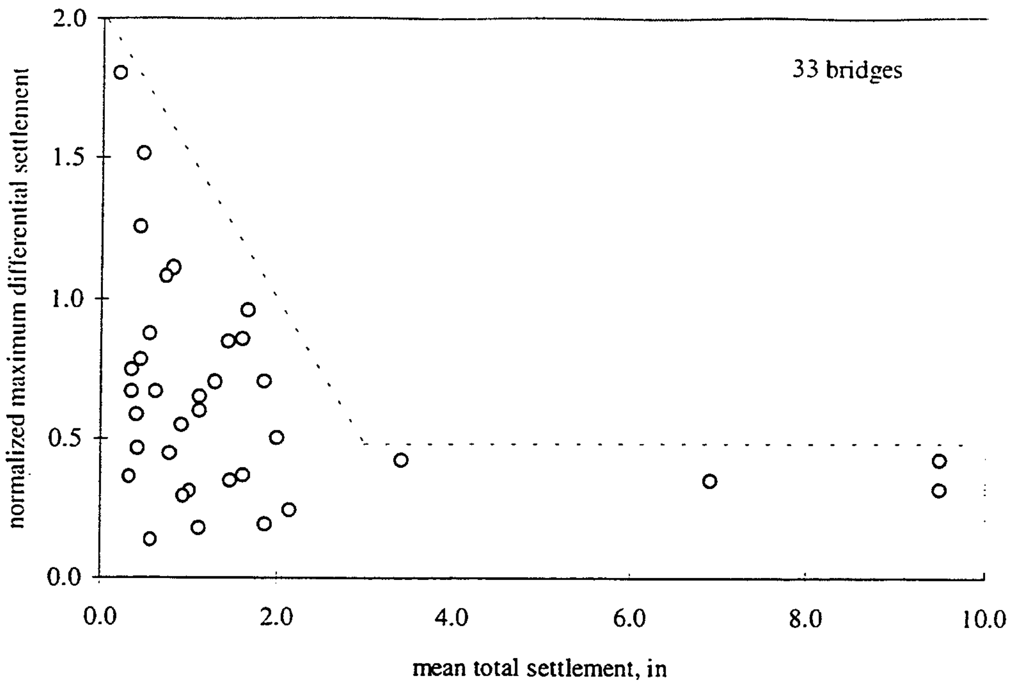


Figure 6-32 Normalized Maximum Differential Settlement versus Mean Total Settlement

DIFFERENTIAL SETTLEMENT AS A FUNCTION OF VARIANCE IN TOTAL SETTLEMENTS

Total settlements are variable. Among a set of foundations of similar type and founded in similar soils, there will be foundation to foundation differences in settlement. An estimate of the expected value of differential settlements can be obtained by assuming that settlements are normally distributed, and that the settlements of individual foundations are independent. Both assumptions must be shown to be consistent with data, but the purpose here is to establish the basic merit of a probabilistic approach to the computation of differential settlements.

Differential settlements are first computed as the absolute value of the difference in total settlements. Consider foundations *i* and *j* with total settlement S_i and S_j, the differential settlement D for the pair of foundations is

$$D = | S_i - S_j | \qquad \textbf{Eq. 6-1}$$

If both settlements S_i and S_j are part of a population of settlements with mean settlement value μ_S and standard deviation of settlements σ_S, then the expected value of differential settlement can be computed as

$$E[D] = E[|S_i - S_j|] \qquad \textbf{Eq. 6-2}$$

where $E[]$ is the expected value operator. For the assumed normal distribution of settlements, it can be shown that the expected value of differential settlement is

$$E[D] = 1.13\,\sigma_S \qquad \textbf{Eq. 6-3}$$

$$\mu_D = 1.13\,\sigma_S \qquad \textbf{Eq. 6-4}$$

This last statement predicts that differential settlements will be related only to standard deviation of total settlements and will not be related mean value of total settlements. This prediction is examined in Figure 6-33. In the figure mean differential settlements are plotted against standard deviation of total settlements. For several of the bridges, total settlements are reported at two foundations only, and so there is not properly a value of σ_S available. For the bridges with more than two settlement points, there is a strong correlation between differential settlement and standard deviation of total settlement. The best-fit line for these data is

$$\mu_D = 0.03 + 1.03\,\sigma_S \qquad \textbf{Eq. 6-5}$$

The relation has a near-zero intercept, and a constant multiplier standard deviation that is close to the predicted value of 1.13. The correlation coefficient for this line is excellent. This same relation can be conveniently written as a normalized relation between differential settlement and mean total settlement. Introducing the coefficient of variation of total settlement COV_S, the normalized differential settlement N_μ can be written as

$$\begin{aligned} COV_S &= \frac{\mu_S}{\sigma_S} \\ N_\mu &= -0.03 + 1.19\,COV_S \end{aligned} \qquad \textbf{Eq. 6-6}$$

This relation is plotted in Figure 6-34. As before, the near-zero intercept and the high value of the correlation coefficient indicate that the differential settlements are a function only of the variability of total settlements.

While the dependence of differential settlements on variability of total settlements is the strong relation, there remains an apparent, though weaker, correlation of differential settlements with total settlement. This relation is examined further by plotting the standard deviation of total settlements against the mean value of total settlements in Figure 6-36. There is an apparent relation of linearly increasing σ_S for increasing mean value of settlements μ_S. This could be equally well be stated as a near-constant value of 0.25 for the coefficient of variation of total settlement. The an apparent relation between differential settlements and total settlement is the outcome of 1) A relation between differential settlements and variability in total settlements, and 2) A relation between magnitude of total settlements and variability of total settlement.

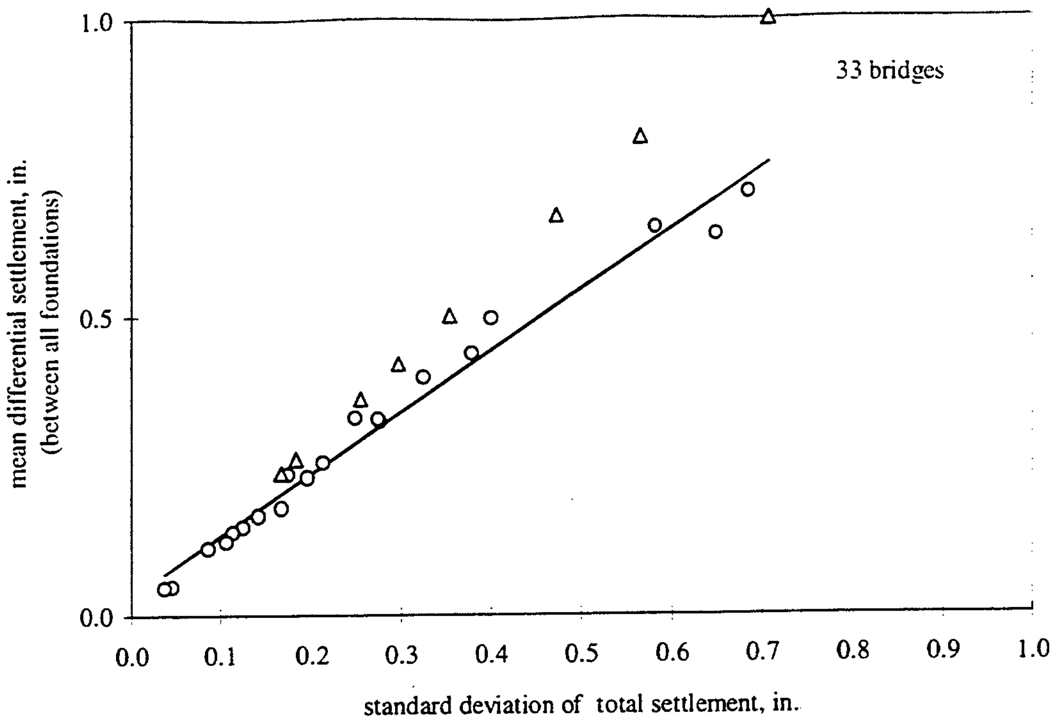
The relation between differential settlement and standard deviation of total settlement should pass through zero. If there is no variability in total settlements, there can be no differences in settlement. In Figure 6-37, mean differential settlements are again plotted against standard deviation of total settlements, but now the linear regression though the data is constrained to pass through zero. For this fit, differential settlements are predicted as

$$\mu_D = 1.10\,\sigma_S \qquad \text{Eq. 6-7}$$

which is very nearly equal to the theoretical value of 1.13 σ_S.

Maximum differential settlements can be predicted as well (Figure 6-38). Here a 90% inclusion bound on differential settlements is used to compute a maximum differential settlement. Rule-of-thumb estimates for maximum differential settlement are also shown in the figure. Rule-of-thumb estimates may be conservative or unconservative depending on the value of *COV* for total settlements. For the typical value of *COV* equal to 0.25, rule-of thumb estimates are conservative, but inaccurate.

Figure 6-39 shows the relation for differential settlements between adjacent foundations only. Differential settlements for adjacent foundations are smaller. This is not consistent with the idea of independent settlements at all foundations. Instead, this indicates that settlements of foundations are correlated.



$\mu_D = 0.03 + 1.03\,\sigma_s$

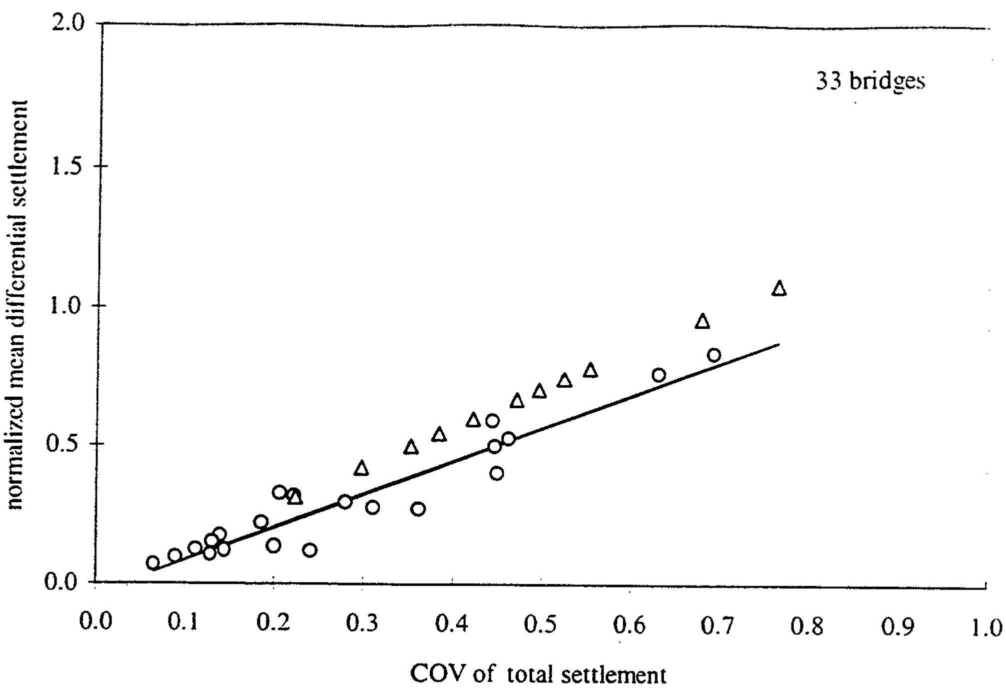
$r = 0.99$

$\sigma_{est} = 0.03$

o data used in calculating regression equation

Δ bridges with only two known settlements

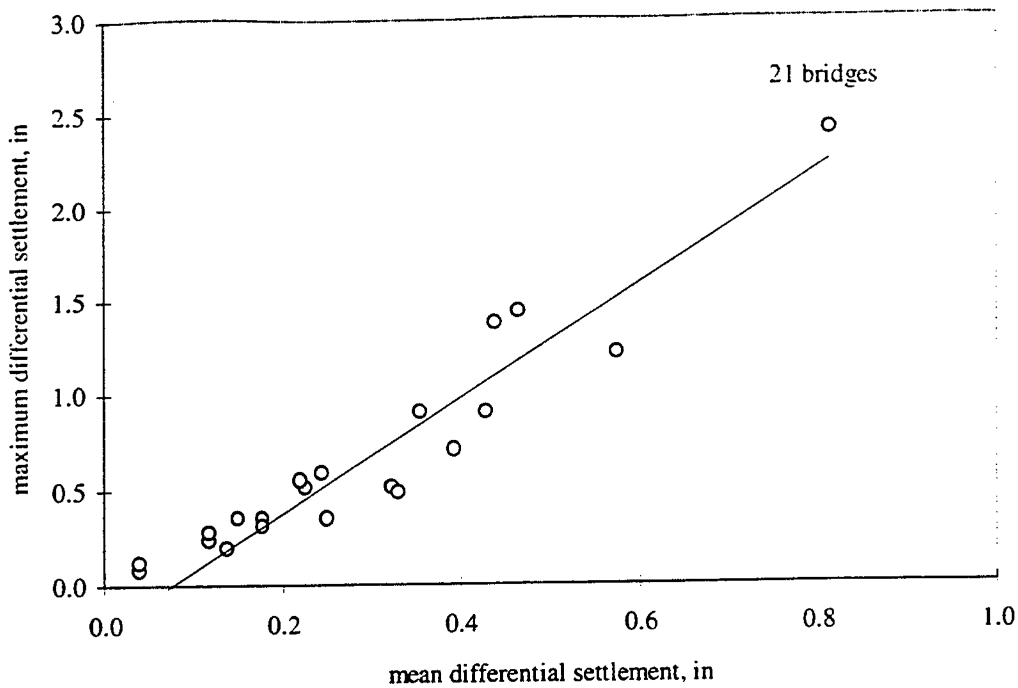
Figure 6-33 Mean Differential Settlement versus Standard Deviation of Total Settlements



$$N_{\mu} = -0.03 + 1.19\ COV_s$$
$$r = 0.95$$
$$\sigma_{est} = 0.07$$

o data used in calculating regression equation
Δ bridges with only two known settlements

Figure 6-34 Normalized Mean Differential Settlement versus COV of Total Settlements

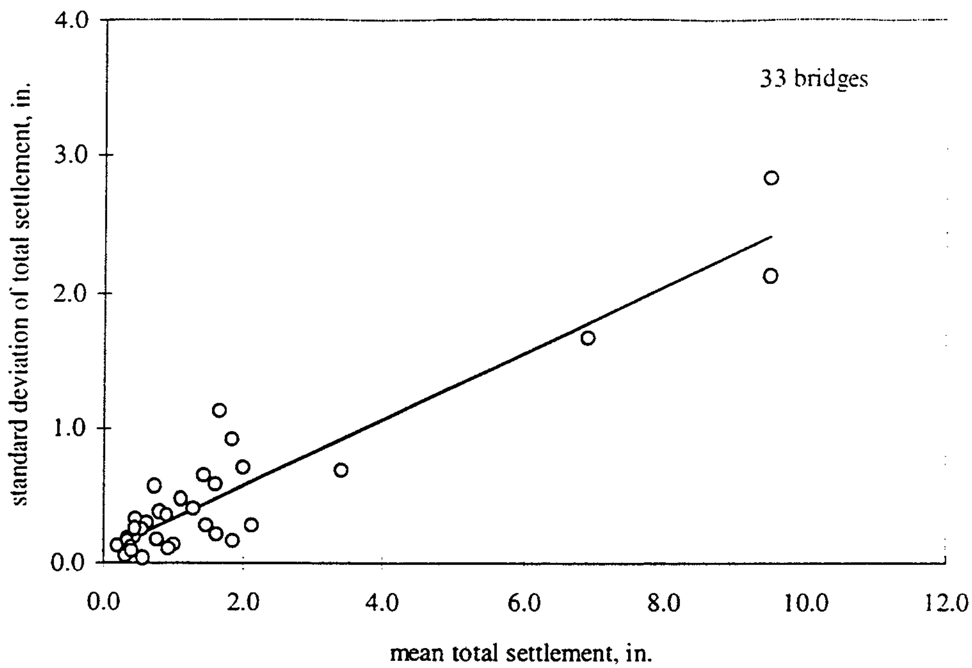


$$M_D = -0.15 + 2.82\,\mu_D$$

$$r = 0.95$$

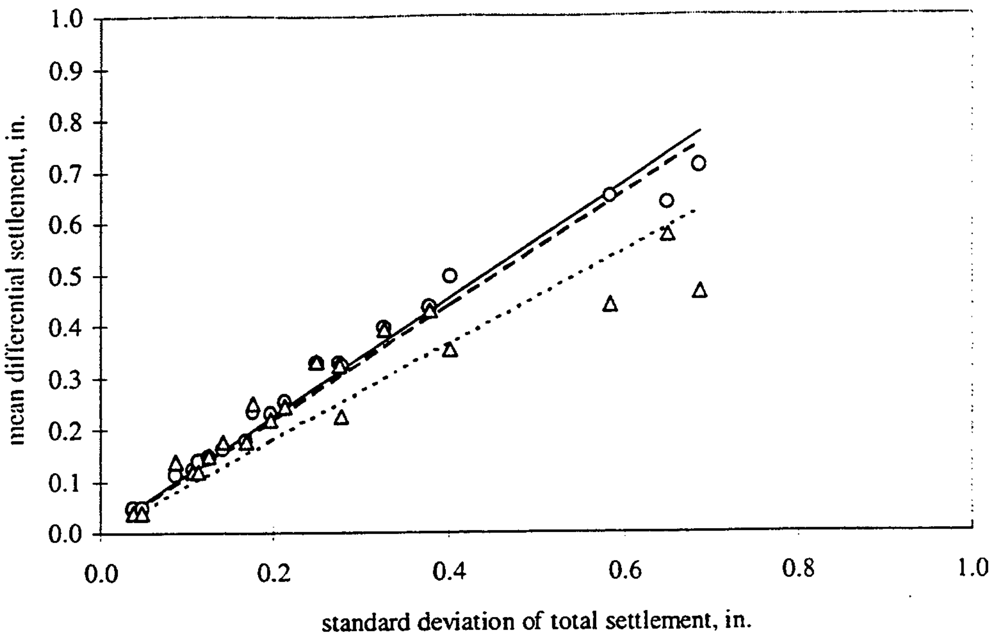
$$\sigma(Err) = 0.18$$

Figure 6-35 Maximum Differential Settlement versus Mean Differential Settlement



$$\sigma_s = -0.06 + 0.25\ \mu S$$
$$r = 0.93$$
$$\sigma_{est} = 0.23$$

Figure 6-36 Standard Deviation of Total Settlements versus Mean Total Settlements



——— theoretical relationship
$\mu_D = 1.13\sigma_s$ $r = 0.99$ $\sigma_{est} = 0.03$ in

– – – – best fitting line through zero for all foundations
$\mu_D = 1.10\sigma_s$ $r = 0.99$ $\sigma_{est} = 0.03$ in

······· best fitting line through zero for adjacent foundations
$\mu_D = 0.91\sigma_s$ $r = 0.89$ $\sigma_{est} = 0.07$ in

○ mean differential settlement computed from differential settlements of all foundations

△ mean differential settlement computed from differential settlements for adjacent foundations

Figure 6-37 Mean Differential Settlement versus Standard Deviation of Total Settlements

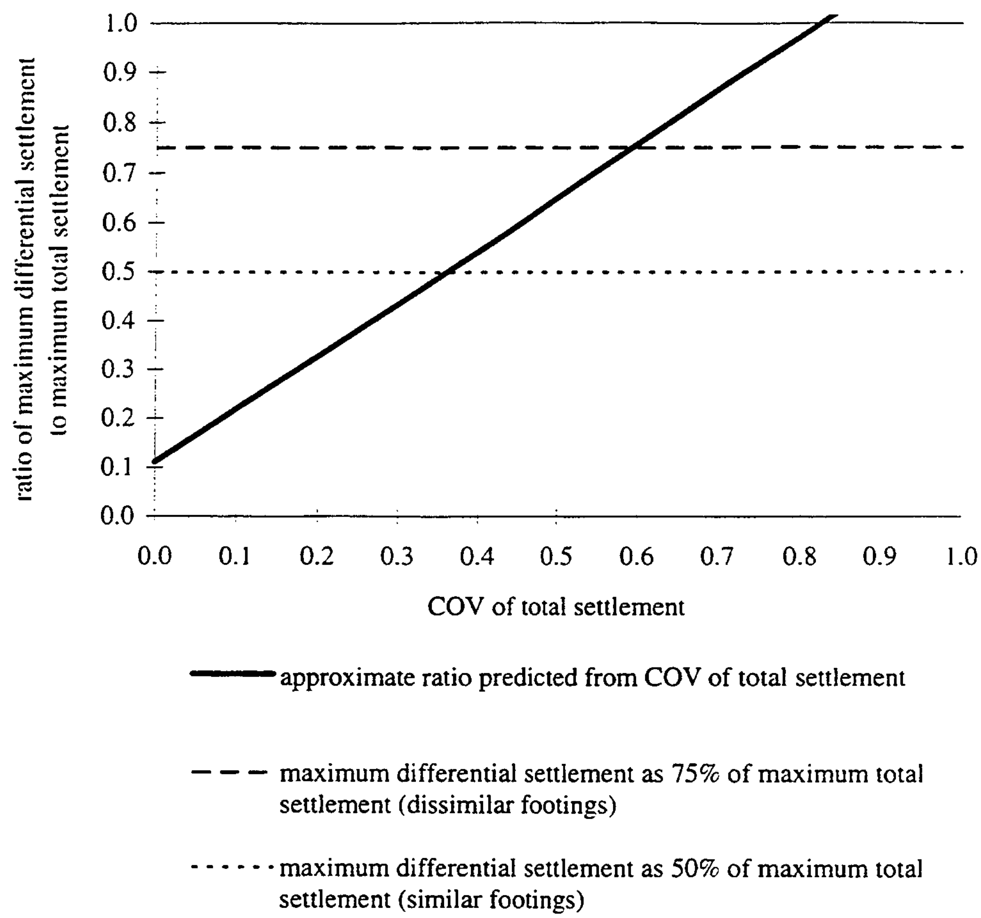
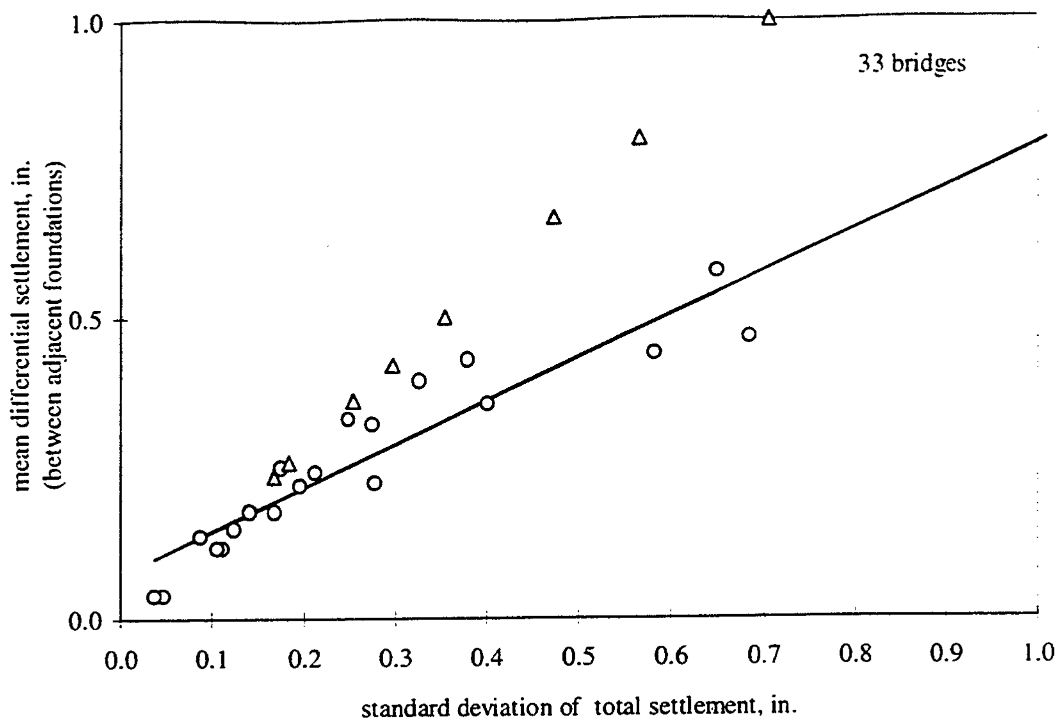


Figure 6-38 Estimates of Differential Settlements



$$\mu_D = 0.07 + 0.72\,\sigma_s$$
$$r = 0.93$$
$$\sigma_{est} = 0.06$$

o data used in calculating regression equation
Δ bridges with only two known settlements

Figure 6-39 Mean Differential Settlement versus Standard Deviation of Total Settlements

INFLUENCE OF DISTANCE ON DIFFERENTIAL SETTLEMENTS

Differential settlements appear to be correlated with distance between foundations as well as overall variability in total settlements. It appears, and it is reasonable, that the total settlements of foundations that are near each other will be similar provided that the soil conditions are similar, that foundation are of the same type and are subject to the same bearing pressures. If the total settlements are similar then the differential settlement of two nearby foundations will be small. In contrast, two foundations that are farther apart are subject to total settlements that may differ more despite a similarity in soil conditions, foundation type and bearing pressures.

The relation between differential settlement and distance between foundations is first examined by plotting mean values of differential settlements for ranges of distance between foundations. The examination is performed only for those bridges or building projects that offer settlement data at five or more foundations. The plots of mean differential settlement versus distance between foundation are shown in Figure 6-40 to Figure 6-51. The projects represented here include two buildings and ten bridges. Of particular interest are Figure 6-40 (Arts and Commerce Bldg), Figure 6-41 (Burlington Bay Skyway), Figure 6-45 (Huey P. Long Bridge), Figure 6-55 (Loppem Bridge), and Figure 6-51 (Stratford Bus Station). In each of these, there is a visible trend of lower differential settlements at smaller distances between foundations, and larger differential settlement at larger distances. The implication of these data is that the statistical approach to prediction of differential settlements addresses only the mean value of a more complicated settlement process that exhibits greater or lesser magnitude according a correlation between settlement and distance.

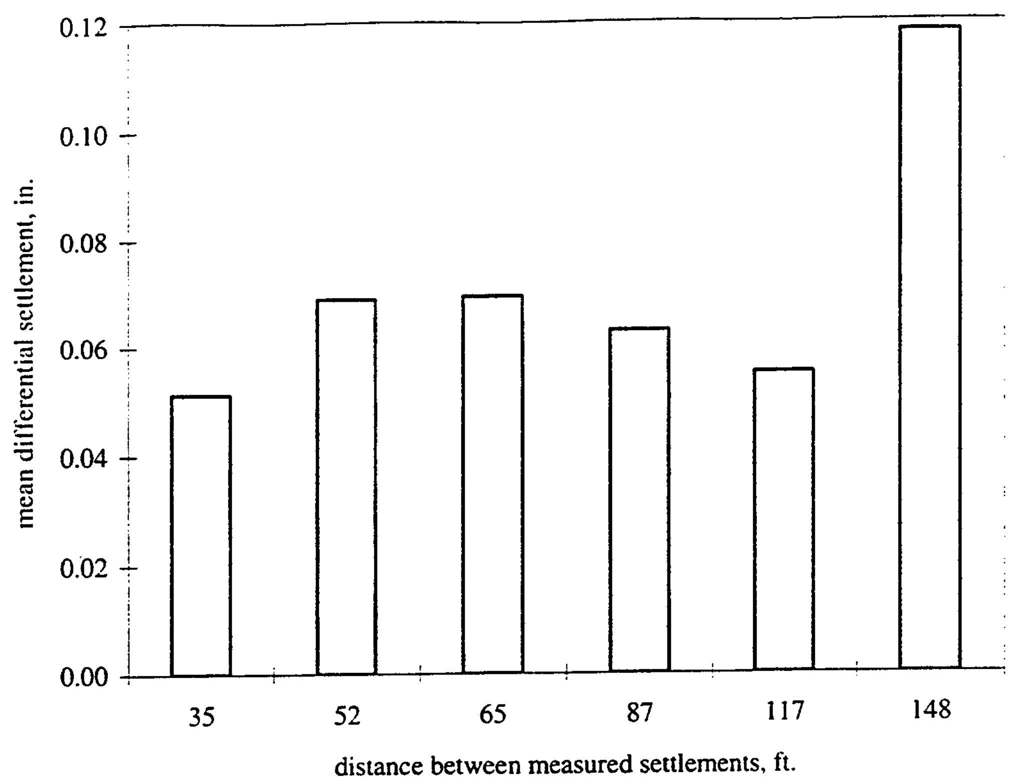


Figure 6-40 Differential Settlement versus Distance - Arts and Commerce Building

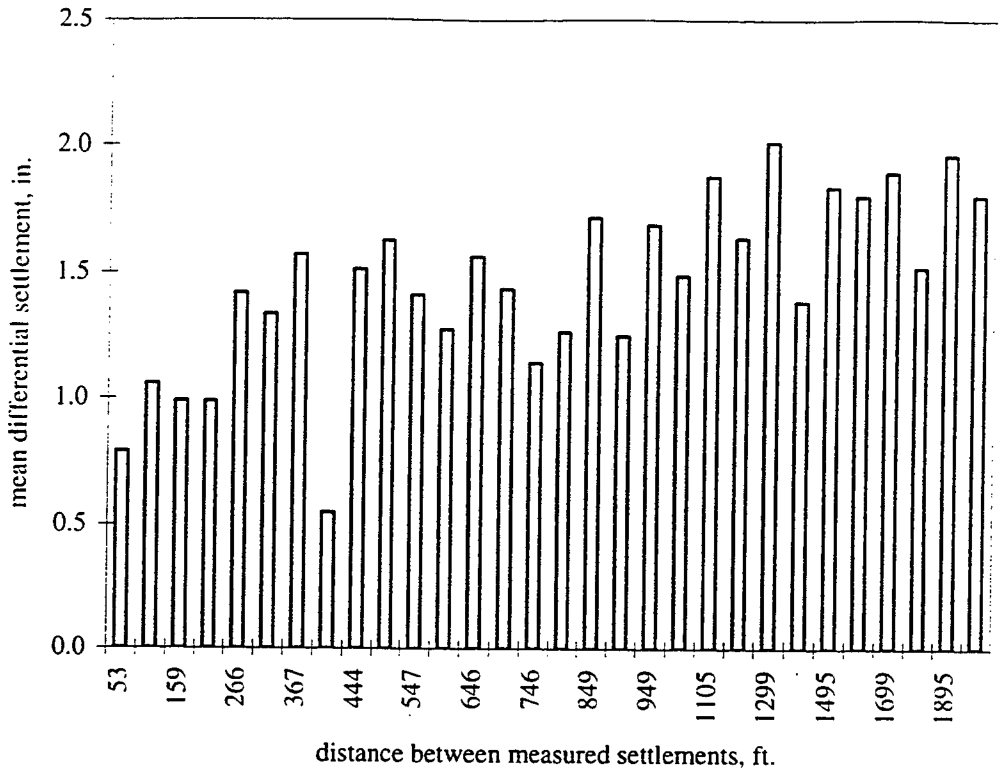


Figure 6-41 Differential Settlement versus Distance - Burlington Bay Skyway

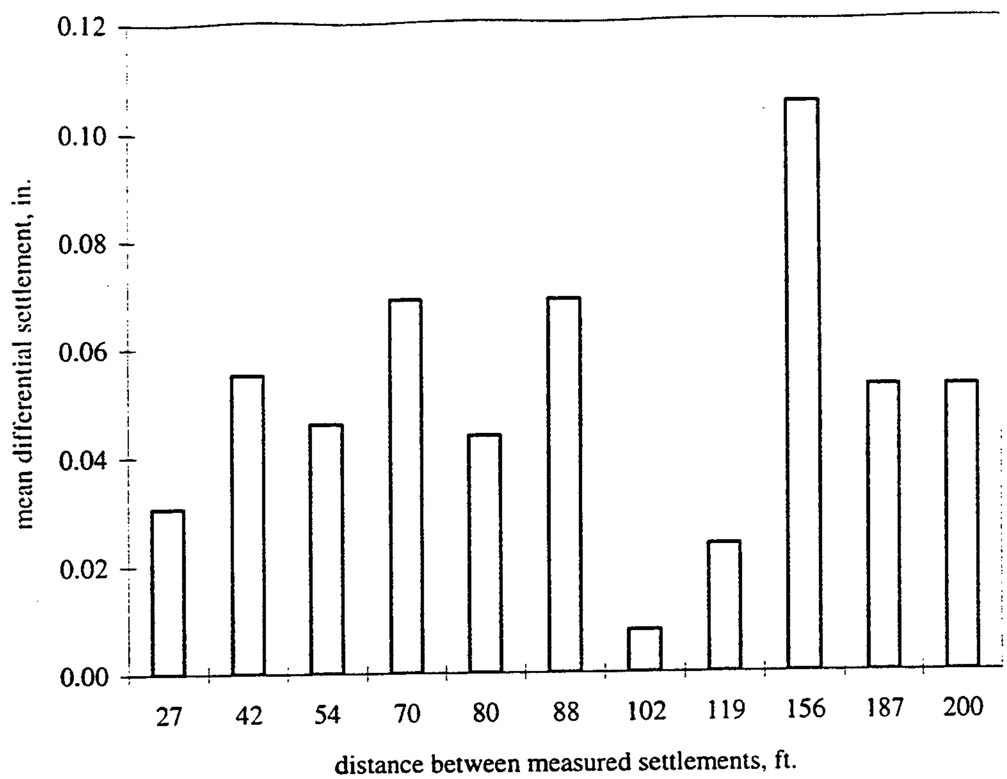


Figure 6-42 Differential Settlement versus Distance - Gentbrugge Bridge

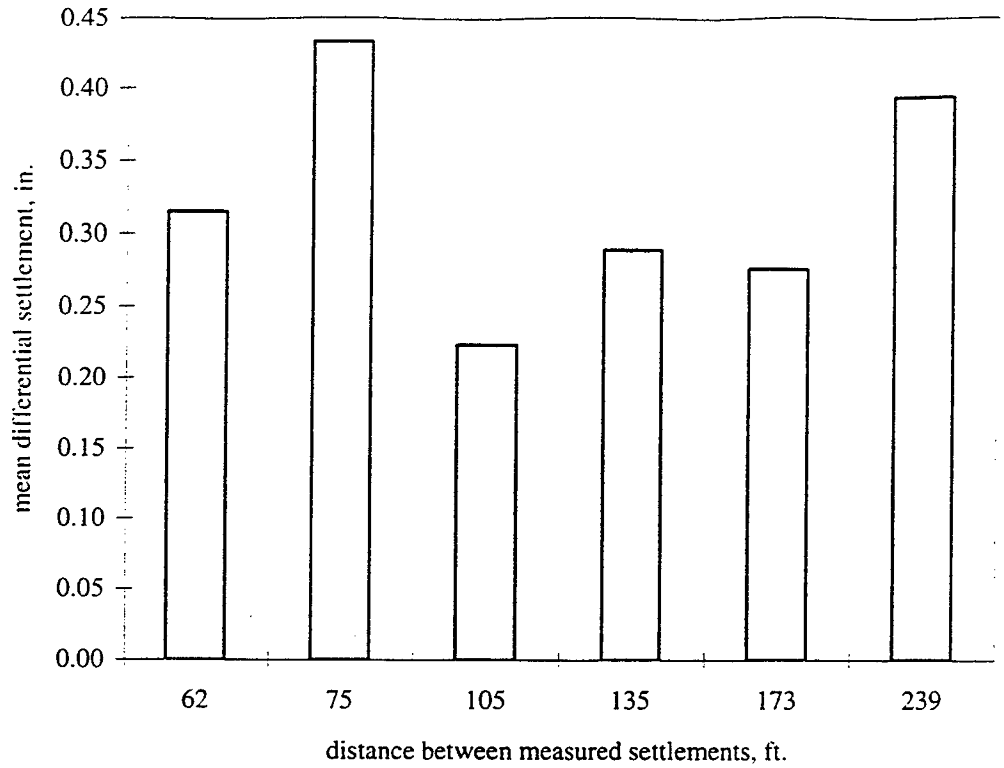


Figure 6-43 Differential Settlement versus Distance - Ghent-Kortruk Road Bridge

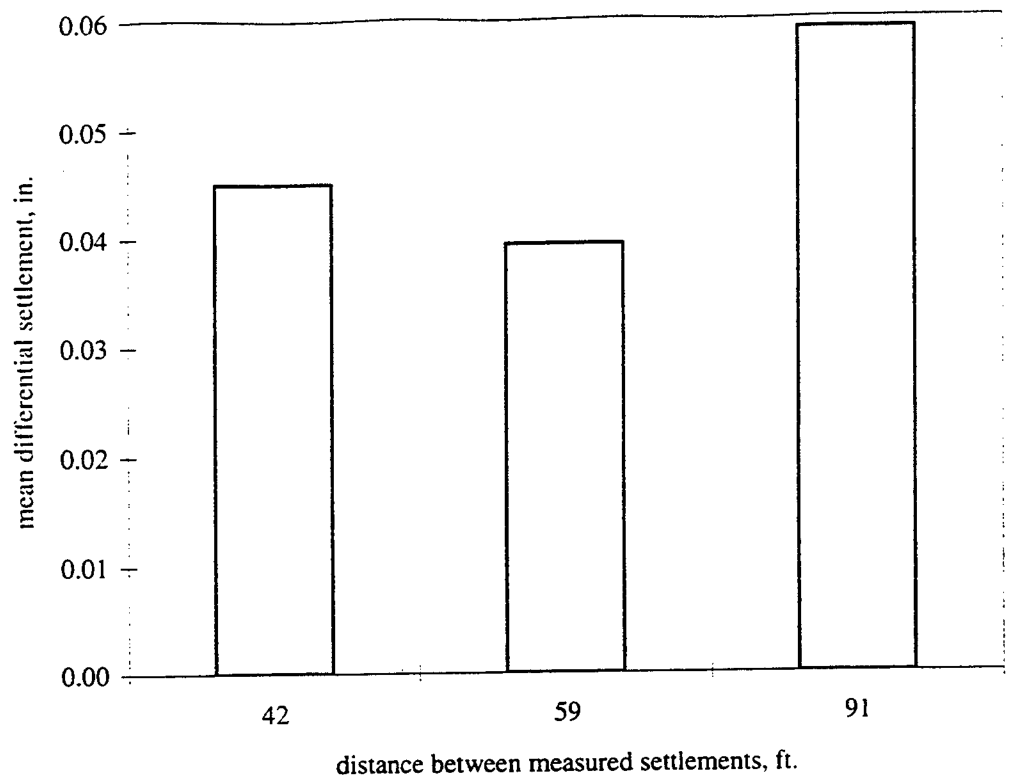


Figure 6-44 Differential Settlement versus Distance - Highway 70 Bridge

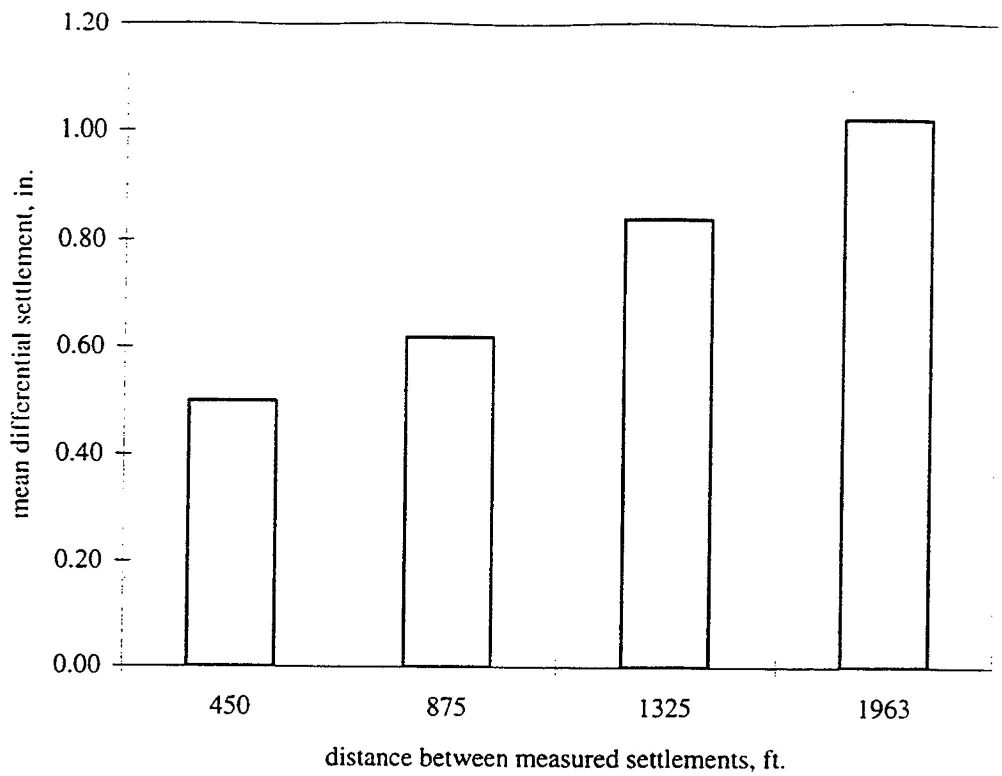


Figure 6-45 Differential Settlement versus Distance - Huey P. Long

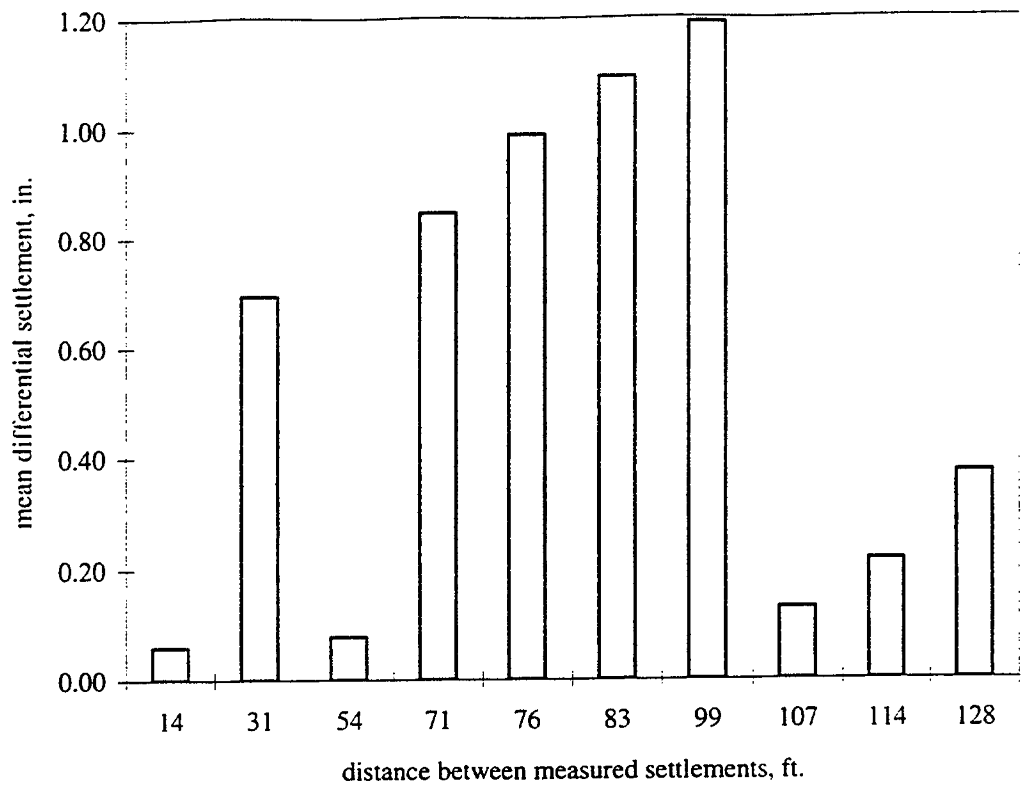


Figure 6-46 Differential Settlement versus Distance - Kortruk-Ghent Railroad Bridge

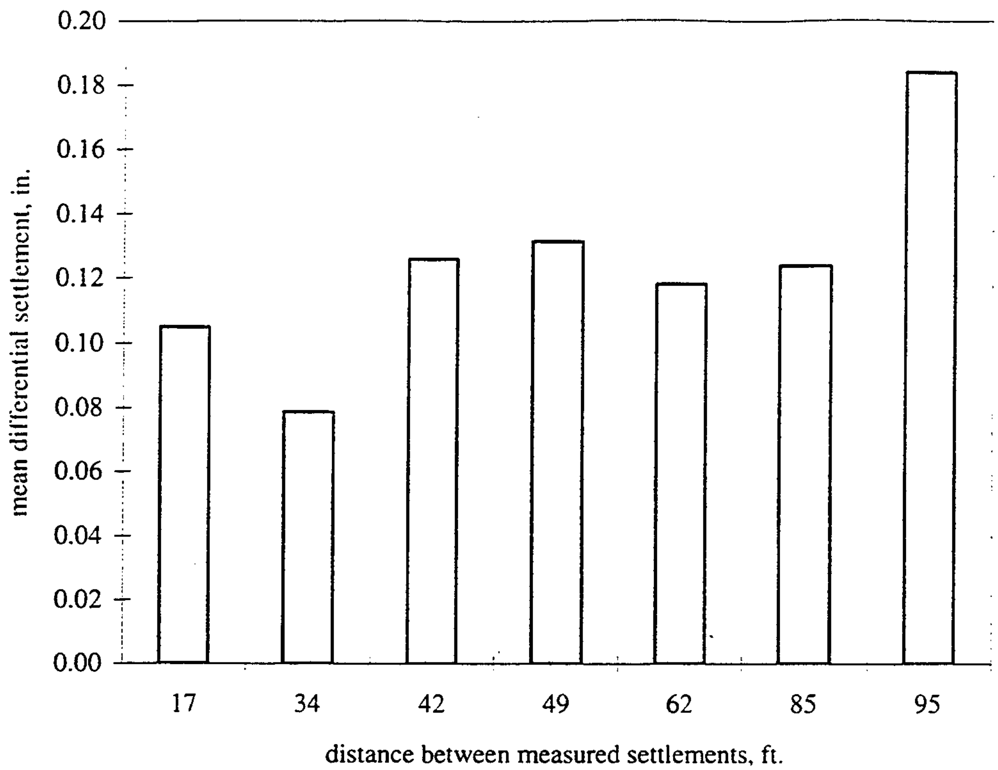


Figure 6-47 Differential Settlement versus Distance - Loppem Bridge

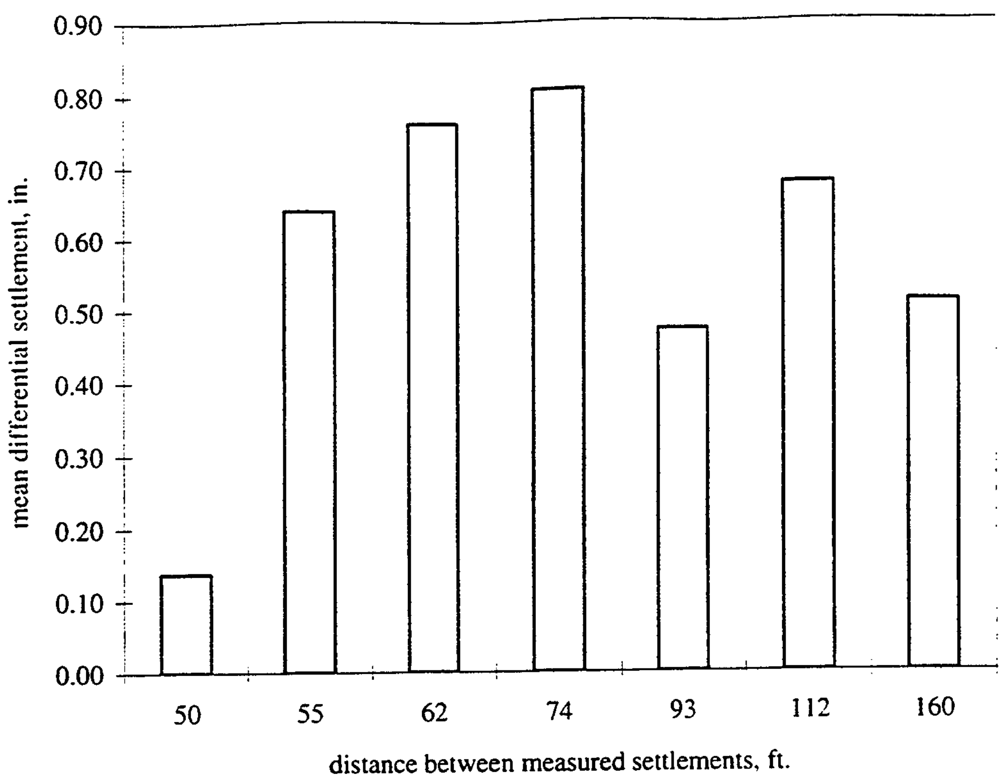


Figure 6-48 Differential Settlement versus Distance - Route Gand-Charleroi Bridge

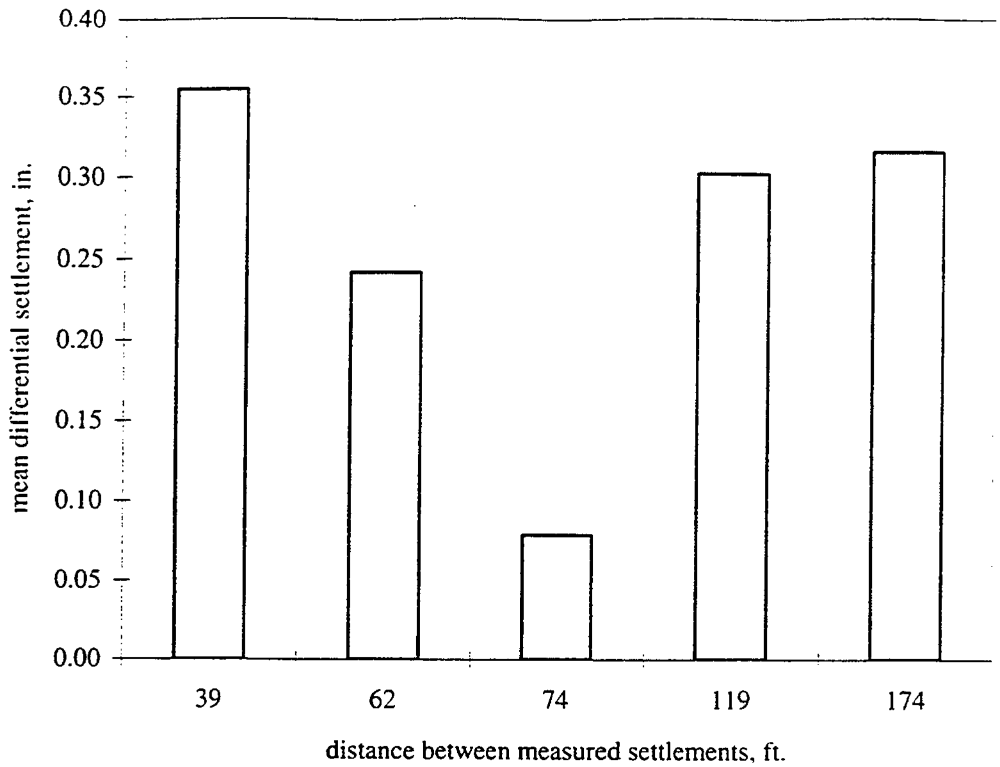


Figure 6-49 Differential Settlement versus Distance - Route Oombergen-Wetteren Bridge

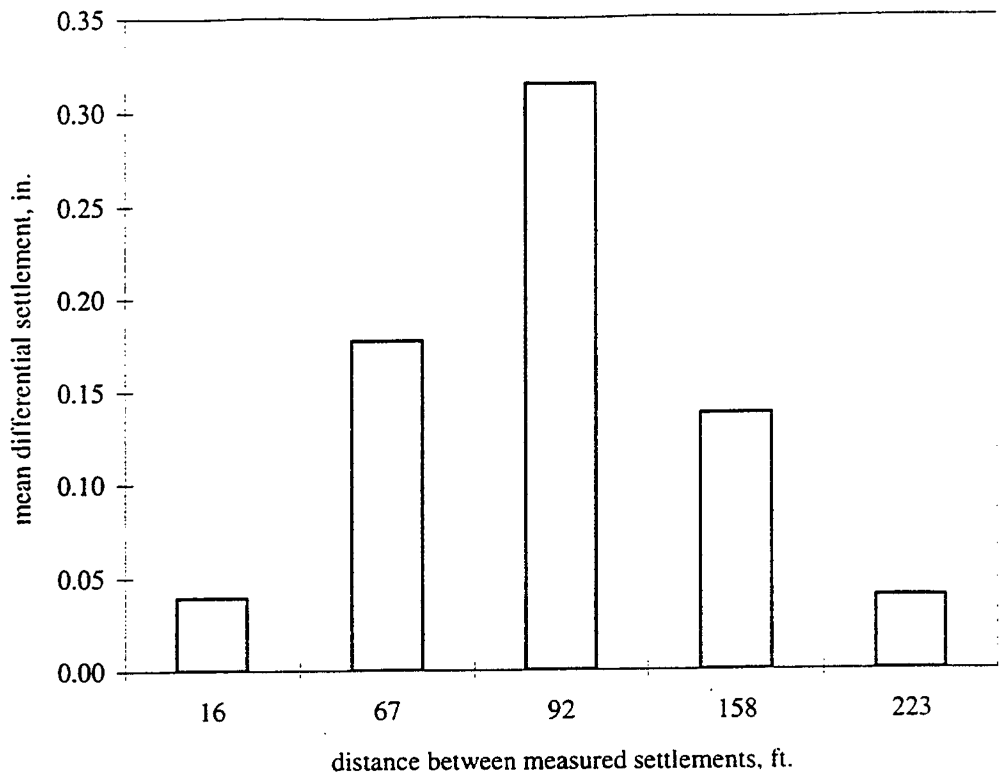


Figure 6-50 Differential Settlement versus Distance - Sterrestreet Bridge

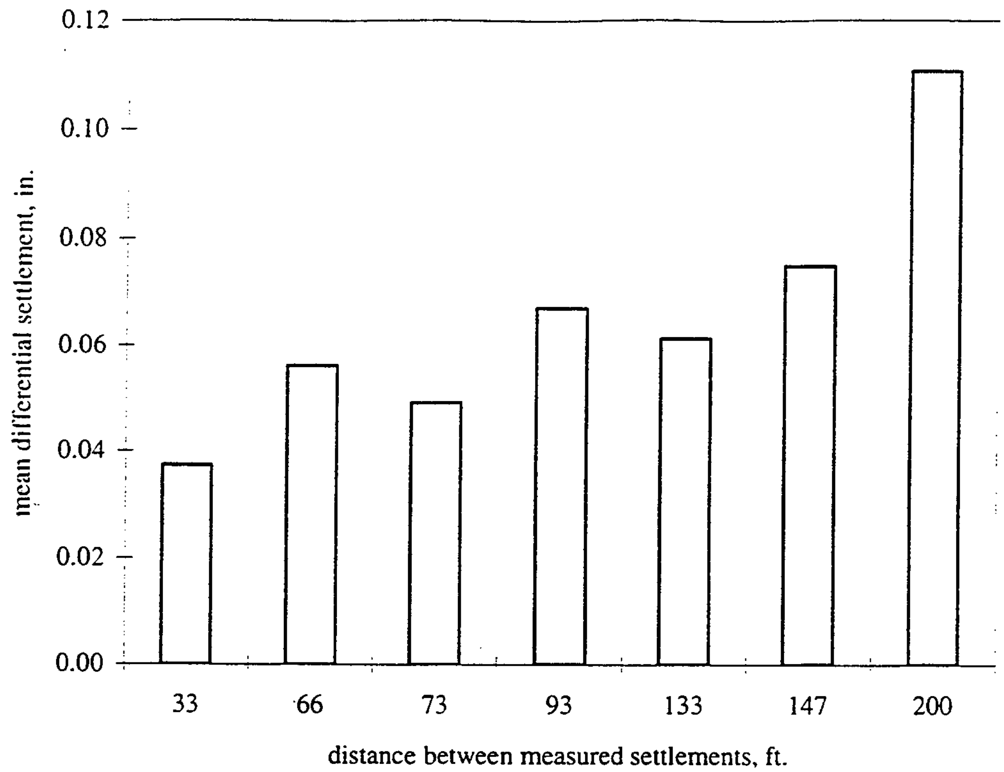


Figure 6-51 Differential Settlement versus Distance - Stratford Bus Station

SPATIAL CORRELATION IN SETTLEMENTS

The relation between differential settlement and total settlements developed earlier uses differences in total settlement of all foundations in a project. The relation indicates that the mean differential settlement equal to about 110% of the standard deviation of total settlements. If this relation is reexamined for differential settlement among adjacent foundations only, it is found that the differential settlements equal about 91% of the standard deviation of total settlements, a lower value. For a specific instance, the mean differential settlement for all combinations of foundations in the Huey P. Long bridge is 0.74 inches. If only the adjacent foundations are considered, then the average differential settlement is 0.47 inches, about one third less.

Settlements that exhibit differences that depend on distance are said to be spatially correlated. Since the correlation exists among similar items in one set of data, this is an autocorrelation. The correlation is the expected value of the product of the total settlement at a location ξ and the total settlement at a location $\xi+\tau$, where ξ is spatial coordinate and τ is a distance added to this coordinate.

$$E[S(\xi)S(\xi+\tau)] = \text{Autocorrelation of Settlement} \qquad \textbf{Eq. 6-8}$$

If the process is stationary, then the correlation is the same for all foundations separated by the distance τ and is not a function of the absolute position ξ.

$$\begin{aligned} E[S(\xi)S(\xi+\tau)] &= E[S(0)S(0+\tau)] \\ &= R_{SS}(\tau) \end{aligned} \qquad \textbf{Eq. 6-9}$$

where $R_{SS}(\tau)$ is the stationary autocorrelation function.

It is simpler to work with a centered process when considering autocorrelation. A centered process has a mean value of zero, and therefore variations in the process occur above or below zero to an equal extent. The use of a centered process also allows for a simple evaluation of the influence of spatial correlation in terms of the basic statistical properties μ_S and σ_S of the underlying process of total settlements. For this purpose, a centered settlement process δ is defined as the difference of the total settlement of one foundation at location ξ and the mean value of total settlements for all foundations in the process.

$$\delta(\xi) = S(\xi) - \mu_S \qquad \textbf{Eq. 6-10}$$

The process δ shows only the differences in settlements at foundations. Differential settlements D can be computed directly in terms of δ as

$$\begin{aligned} D &= S(\xi_1) - S(\xi_2) \\ D &= \delta(\xi_1) - \delta(\xi_2) \end{aligned} \qquad \textbf{Eq. 6-11}$$

where ξ_1 and ξ_2 are the locations of two foundations. The mean value μ_D and variance $\sigma_D{}^2$ of the centered process δ can be evaluated in terms of the mean value μ_S and the standard deviation σ_S of the total settlement process as

$$\begin{aligned} E[\delta] &= E[S - \mu_S] \\ &= 0 \\ E\left[\delta^2\right] &= E\left[(S-\mu_S)^2\right] \\ &= \sigma_S{}^2 \end{aligned} \qquad \textbf{Eq. 6-12}$$

The mean value and variance of differential settlement D can be computed as well.

$$E[D] = E[\delta(\xi 1) - \delta(\xi 2)]$$
$$= 0$$
$$E\left[D^2\right] = E\left[(\delta(\xi 1) - \delta(\xi 2))^2\right] \qquad \text{Eq. 6-13}$$
$$= 2\sigma_S^{\,2}$$

These results are all for uncorrelated or independent processes. The existence of a spatial correlation in total settlements requires that the computation of variance for the centered process δ be modified to recognize the correlation.

$$E[\delta(\xi_1)\delta(\xi_2)] = E[\delta(\xi_1)\delta(\xi_1 + \tau)]$$
$$= \sigma_\delta^{\,2}(1 - \rho(\tau)) \qquad \text{Eq. 6-14}$$
$$= \sigma_S^{\,2}(1 - \rho(\tau))$$

where $\rho(\tau)$ is a normalized autocorrelation function. For independent processes, the autocorrelation function is equal to one when τ equals zero. For all other values of τ the function ρ is zero. That is, for independent processes (with zero mean, this is one area where the use of a centered process simplifies things) the variance exists, but the product of values of the process at separate points sums to zero.

$\rho(\tau)$ is empirical. Its form is determined by the available data for a process. A common form of $\rho(\tau)$ indicates strong correlation at small values of τ with a gradual transition to zero correlation at larger values of τ. As an example, consider a function of the form

$$\rho(\tau) = e^{-(\tau/\beta)^2} \qquad \text{Eq. 6-15}$$

Using this function $\rho(\tau)$, the expected value of the square of differential settlements can be computed recognizing the spatial correlation of settlements.

$$E\left[D^2(\tau)\right] = E\left[(\delta(\xi) - \delta(\xi + \tau))^2\right]$$
$$= 2\sigma_S^{\,2}(1 - \rho(\tau))$$
$$= 2\sigma_S^{\,2}\left(1 - e^{-(\tau/\beta)^2}\right) \qquad \text{Eq. 6-16}$$
$$\sqrt{E\left[D^2(\tau)\right]} = \sqrt{2\sigma_S^{\,2}\left(1 - e^{-(\tau/\beta)^2}\right)}$$

The influence of spatial correlation is shown in Figure 6-52. Here the function $\rho(\tau)$ is plotted along with the $\sqrt{E\left[D^2(\tau)\right]}$ and $E[\,|D(\tau)|\,]$. The *COV* for total settlements is taken as 0.25, a typical value. The normalized autocorrelation function ρ has a value of 1 at zero distance τ, and ρ decreases to near zero at a distance τ about equal to two times β. As a result, differential settlements are zero at zero separation distance, and increase to the expected value of $1.13\sigma_S$ for independent, uncorrelated settlements at large separation distance. Notice that the root-mean-square estimate of differential settlements is larger than the estimate of absolute value of differential settlements. Figure 6-52 also shows the rule-of-thumb estimate of differential settlement at 50% of total settlement.

Expected values of differential settlement increase as distances between foundations increase, and differential settlements level off to a maximum value equal to the value for differential settlements of independent, uncorrelated foundations. Distances between foundations are relative. It is the magnitude of distance relative to the parameter β that determines expected values of differential settlements. Large values of β indicate a correlation of settlements over greater real distances.

The existence of spatial correlation of settlements is investigated here using data on total settlements from twelve projects. For each of these projects, the average values of differential settlements are computed in several ranges of distance between foundations. Next, a search is made for spatial correlation functions that fit the observed pattern in differential settlements. Evidence of correlation is found for six projects. Figure 6-53 through Figure 6-58 show the data and the fits of four forms of normalized spatial correlation function. The four forms are

$$\rho(\tau) = e^{(-|\tau|/\beta)}$$
$$\rho(\tau) = e^{-(\tau/\beta)^2}$$
$$\rho(\tau) = e^{(-|\tau|/\gamma)} cos(\tau / \gamma) \qquad \textbf{Eq. 6-17}$$
$$\rho(\tau) = e^{-(\tau/\lambda)^2} cos(\eta\tau / \lambda)$$

The four forms of $\rho(\tau)$ are adaptations of spatial correlation functions found in the literature. All are empirical. β, γ, η and λ are parameters of the correlation functions. Values for the parameters are found by a search for minimum error in the prediction of differential settlements.

Data on differential settlements are used to compute discrete expected values of the product

$$D(0)D(\tau) \qquad \textbf{Eq. 6-18}$$

This product is the basic input data for fitting a function to describe $R(\tau)$. Data are available for specific values of τ determined by the distances between foundations. Parameters for each of the four function forms are then selected to match the estimates of $D(0)D(\tau)$ from data.

Basic results of the search for spatial correlation are listed in Table 6-3. For each of the twelve projects, the best form of function is listed together with two estimates of error. $S_{r,\rho=0}$ is the standard error with no spatial correlation considered. S_r is the standard error using the best spatial correlation function discovered by search. Spatial correlation may exist where S_r is less than $S_{r,\rho=0}$. The correlation coefficient, r, is also shown. A further summary of results is listed in Table 6-4. In the table, the number of D(0)D(t) estimates is listed for each project, together with the minimum and maximum values of τ, the age of the project at the time the settlement data were collected.

There is spatial correlation in at least two projects; the Arts and Commerce Building and the Loppem Bridge. Three other projects, Burlington Bay Skyway, Sterrestreet Bridge and Stratford Bus Station show some correlation in space. Plots of D(0)D(t), and of the fits for correlation functions are shown in Figure 6-53 through Figure 6-57. The remaining seven projects have total settlements that do not exhibit a spatial correlation.

The spatial correlation functions are used to computed expected values of differential settlements. The results are shown in Figure 6-59 through Figure 6-63. In these figures, the spatial correlation is used to compute the expected value of maximum differential settlements. Notice that for each of the projects, the full value of the maximum differential settlement is reached at a characteristic distance between foundations, and that lesser values of differential settlement are observed at smaller distances.

Data indicate that spatial correlation may exist for settlements of foundations for some bridges, and that differential settlements will be less for correlated settlements of nearby foundations. The work here is based on performance of completed structures and does not provide a basis of prediction of correlation of settlements prior to construction. What is needed are methods for the use of site exploration data in the quantification of spatial correlation in expected settlements.

SUMMARY

Pavement faults are a problem in differential settlements. Quantitative methods to predict differential settlements are needed. Relations between differential settlements and variability in total settlements are demonstrated in this section. The existence of spatial correlation is explored, and the importance of spatial correlation to the prediction of differential settlements is noted.

project	function	$S_{r,\rho=0}$	S_r	r
Arts and Commerce Building	*d*	0.51	0.03	0.97
Burlington Bay Skyway	*a*	2.08	0.85	0.57
Gentbrugge Bridge	*d*	1.79	1.88	-
Ghent-Kortruk Road Bridge	*c*	1.03	1.04	-
Highway No. 70 Bridge	*d*	0.34	0.39	-
Huey P. Long Bridge	*c*	0.08	0.01	-
Kortruk-Ghent Railroad Bridge	*d*	2.69	2.63	-
Loppem Bridge	*d*	0.68	0.11	0.90
Rt. Gand-Charleroi Bridge	*d*	1.47	1.72	-
Rt. Oombergen-Wetteren Bridge	*b*	1.15	1.15	-
Sterrestreet Bridge	*d*	3.20	1.00	0.82
Stratford Bus Station	*d*	1.02	0.51	0.64

Table 6-3 Errors of best-fitting correlation functions

Project	pts	L_{min}, ft	L_{max}, ft	t_{p-c}, yrs	μ_s, in	COV_s	$L_{0.5D}$, ft	$L_{0.75D}$, ft	L_D, ft
Arts and Commerce Building	8	33	52	3.7	0.49	0.12	19	30	45
Burlington Bay Skyway	58	53	500	10.0	6.92	0.24	54	155	384
Gentbrugge Bridge	12	38	75	0.0	0.82	0.46	33	51	75
Ghent-Kortruk Road Bridge	8	113	113	2.0	1.48	0.19	21	35	64
Highway No. 70 Bridge	6	43	43	5.7	0.33	0.15	11	18	28
Huey P. Long Bridge	5	325	775	4.0	3.42	0.20	104	253	640
Kortruk-Ghent Railroad Bridge	12	37	37	0.9	1.61	0.36	9	15	33
Loppem Bridge	9	43	43	5.0	0.96	0.11	16	25	37
Rt. Gand-Charleroi Bridge	8	78	78	0.3	1.38	0.43	20	32	49
Rt. Oombergen-Wetteren Bridge	8	66	66	0.5	1.61	0.13	3	5	8
Sterrestreet Bridge	8	66	92	5.7	0.20	0.63	25	39	56
Stratford Bus Station	10	33	137	0.0	0.31	0.18	30	48	74

Table 6-4 Correlation distances of projects

Project	span type	L_{min}, ft	ρ_l	$F(L_{min}))$	σ_s, in	COV_s	μ_s, in	allowabl μ_s, in
Burlington Bay	continuou	53	0.75	2.0	1.67	0.24	6.92	7.66
Gentbrugge	continuou	38	0.69	1.8	0.38	0.46	0.82	2.57
Highway No. 70	continuou	43	-0.22	0.9	0.05	0.15	0.33	4.50
Huey P. Long	—	325	0.31	1.2	0.69	0.20	3.42	33.79
Loppem	continuou	43	-0.16	0.9	0.11	0.11	0.94	6.29
Sterrestreet	continuou	66	-0.26	0.9	0.12	0.63	0.20	1.62

Table 6-5 Comparison of allowable to observed mean total settlements, including effects of spatial correlation

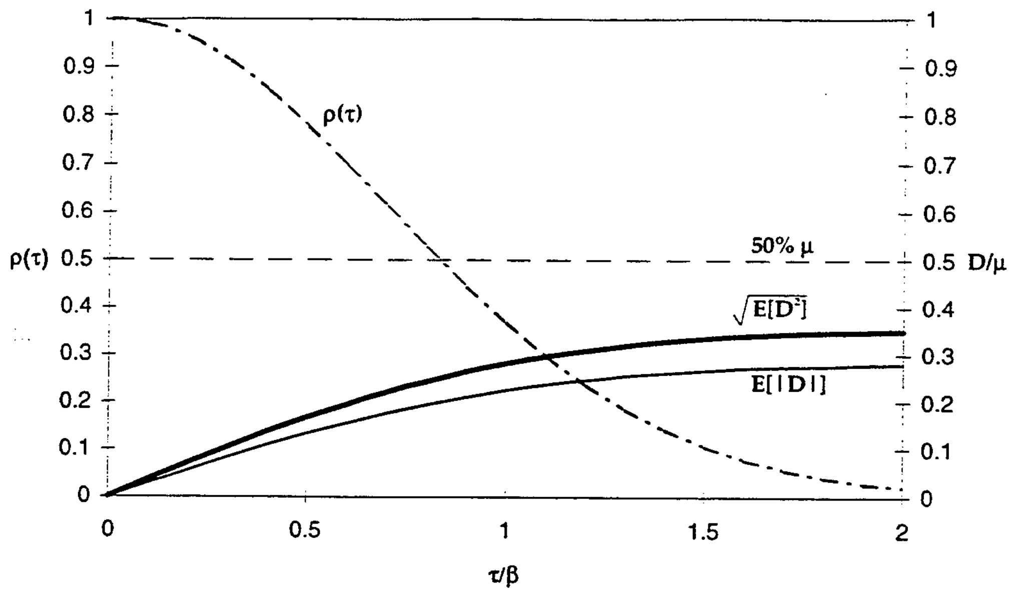


Figure 6-52 Example of Spatial Correlation and Differential Settlements

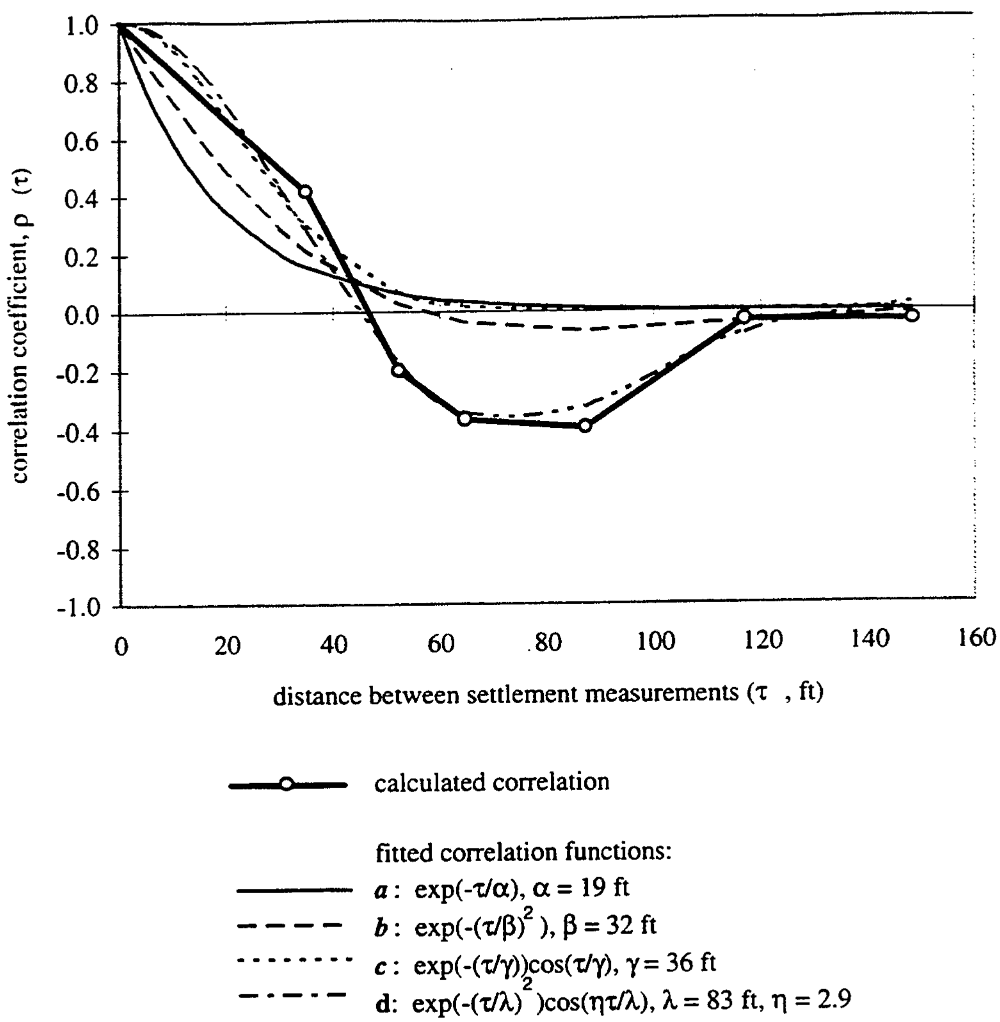


Figure 6-53 Spatial Correlation of Settlements - Arts and Commerce Building

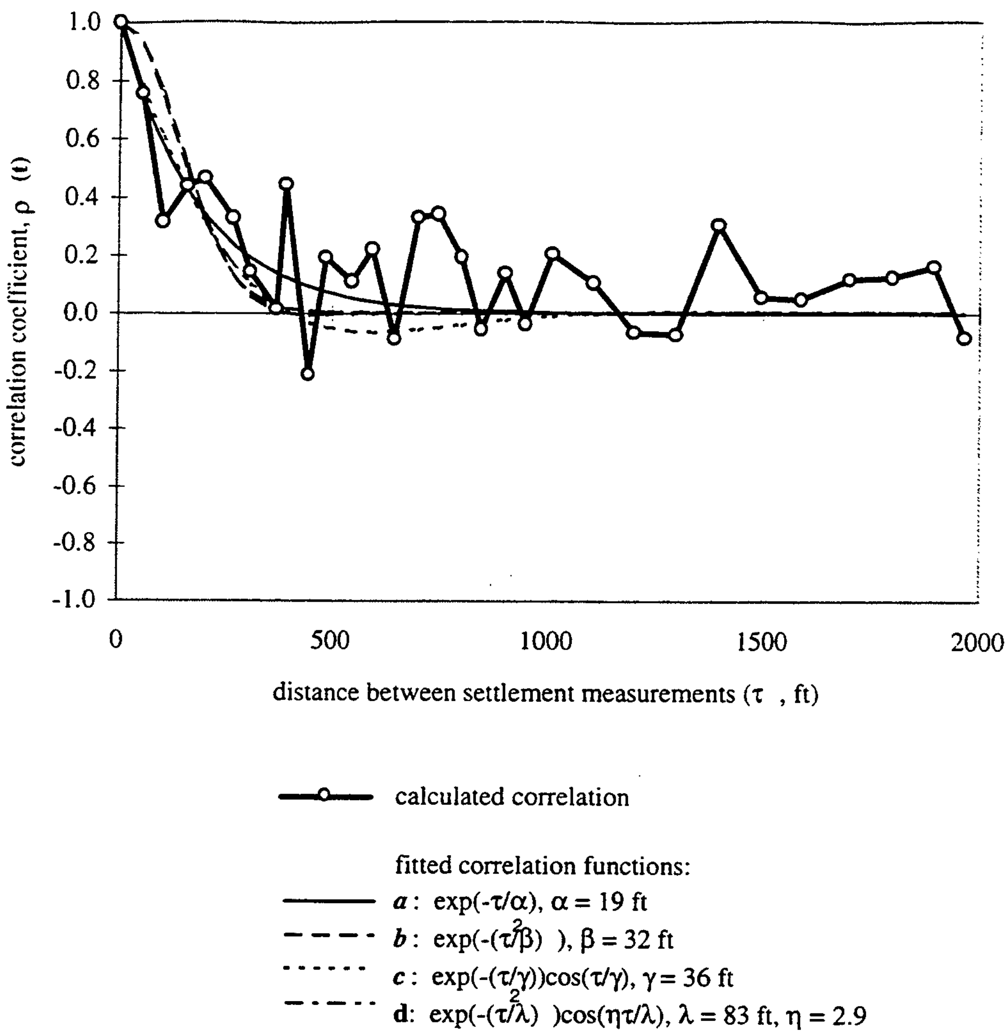


Figure 6-54 Spatial Correlation of Settlements - Burlington Bay Skyway

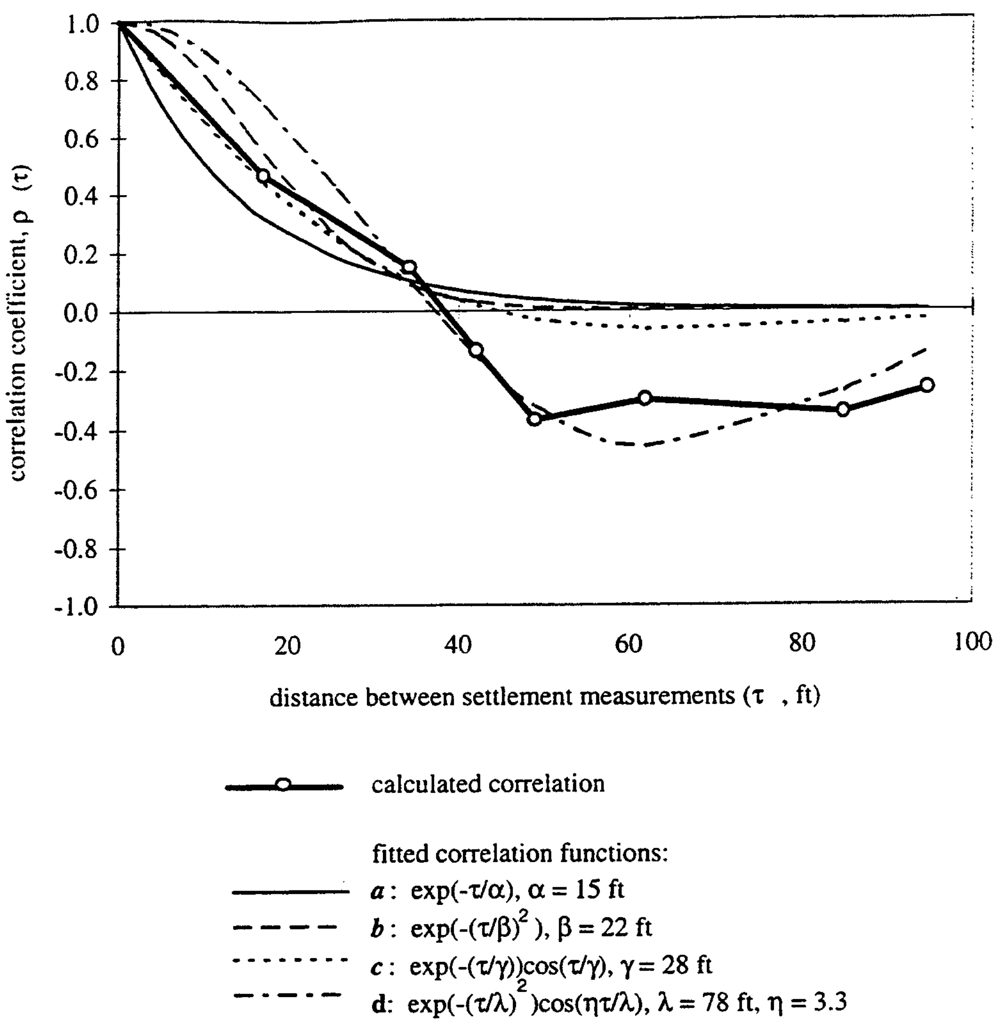


Figure 6-55 Spatial Correlation of Settlements - Loppem Bridge

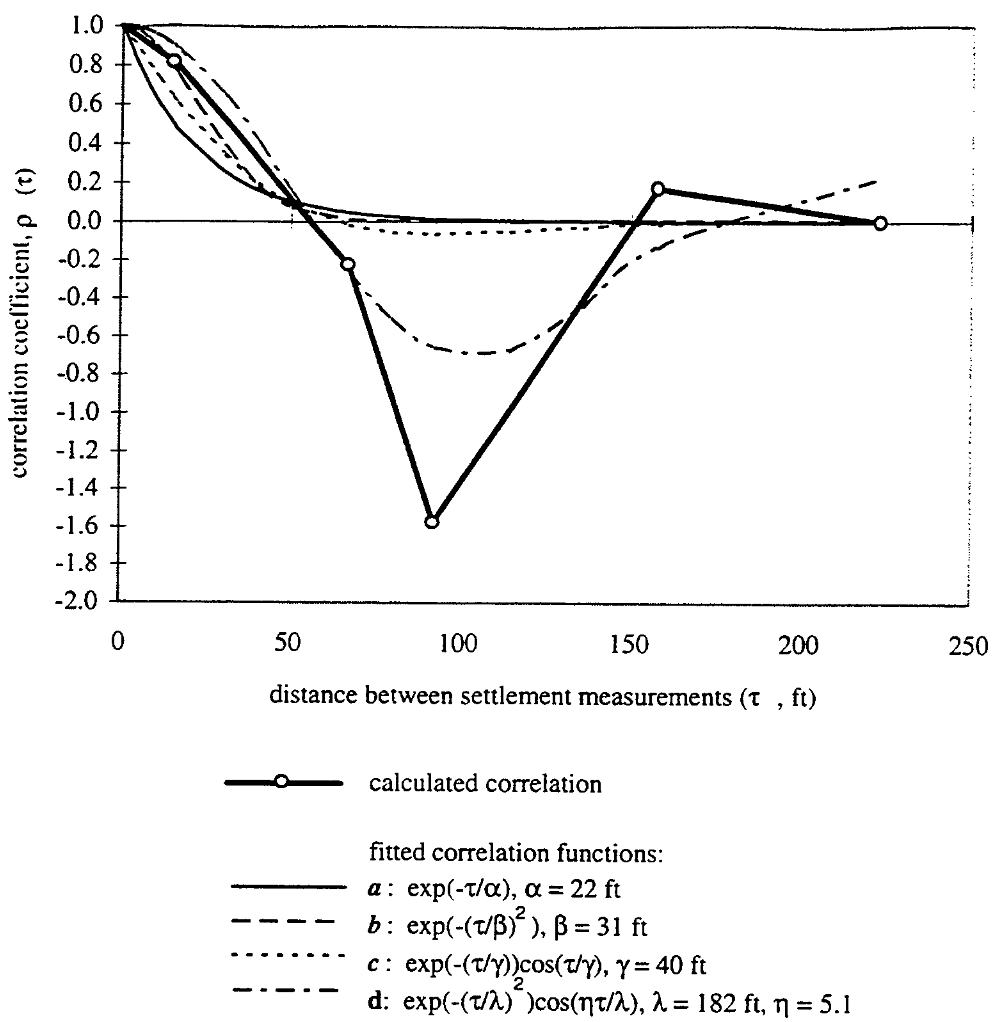


Figure 6-56 Spatial Correlation of Settlements - Sterrestreet Bridge

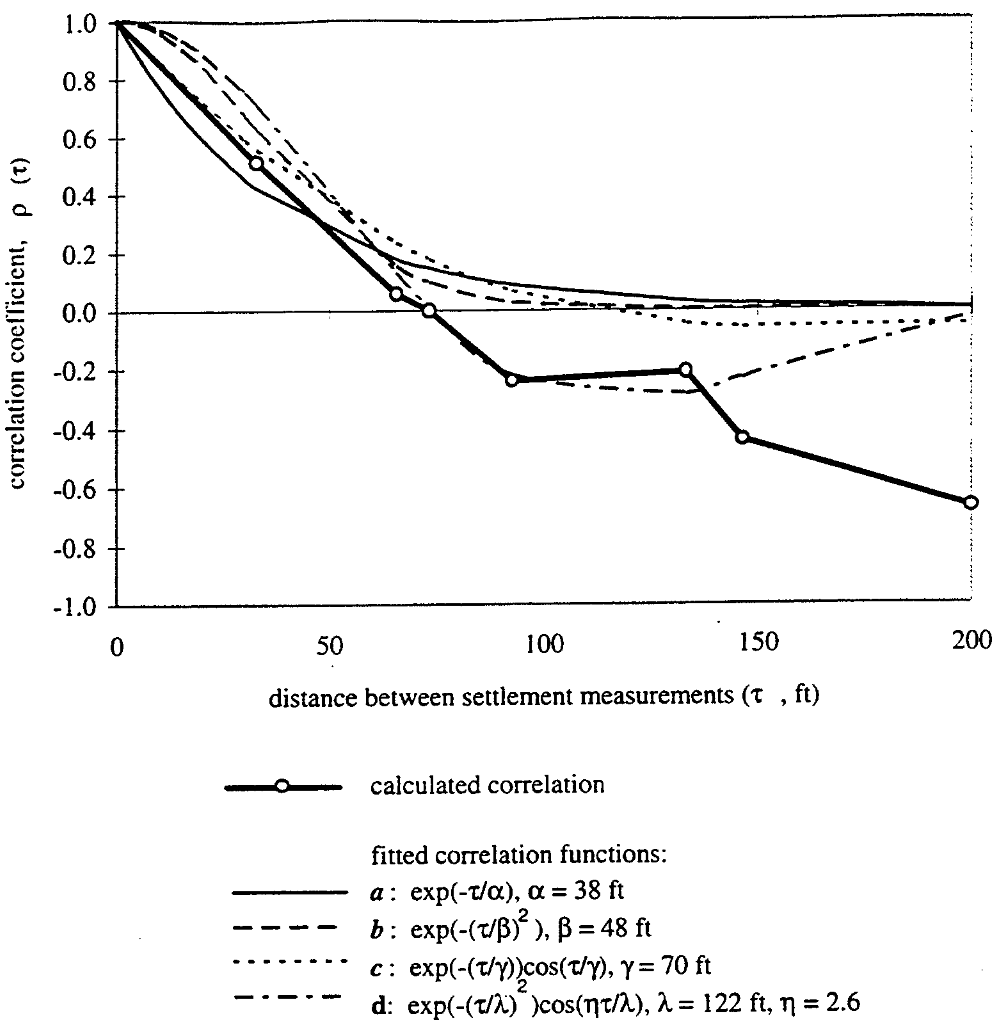


Figure 6-57 Spatial Correlation of Settlements - Stratford Bus Station

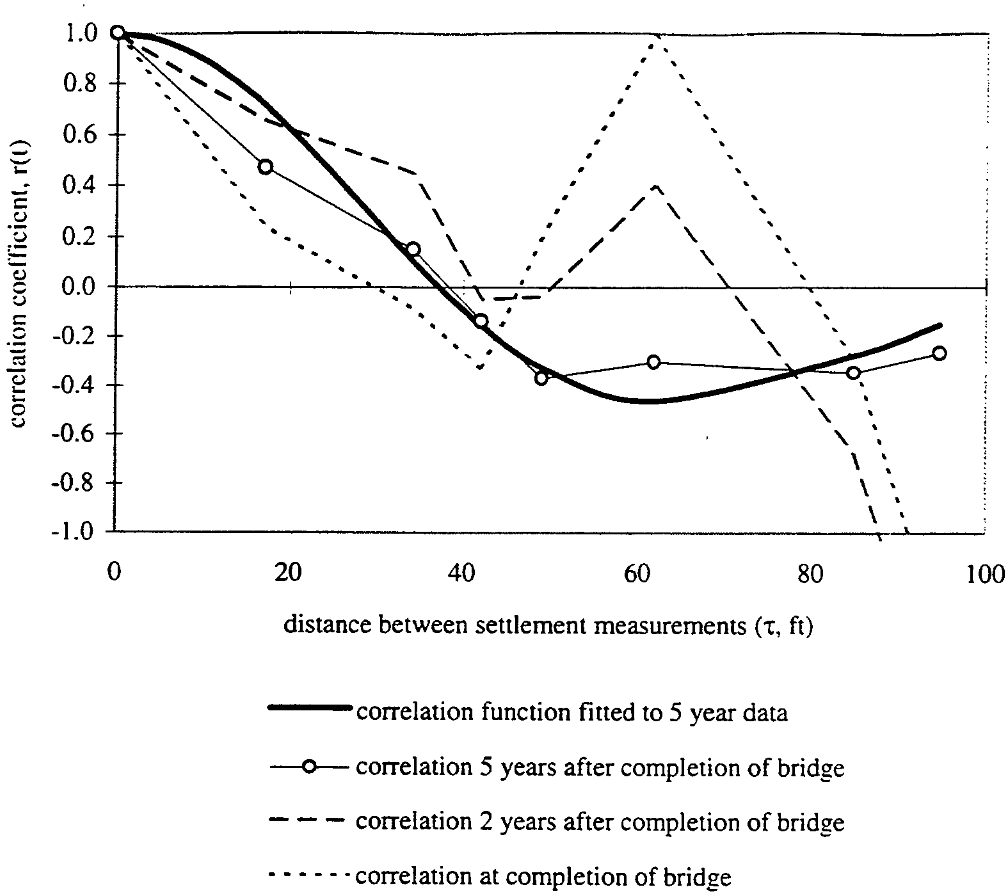


Figure 6-58 Spatial Correlation of Settlements - Dependence on Time, Loppem Bridge

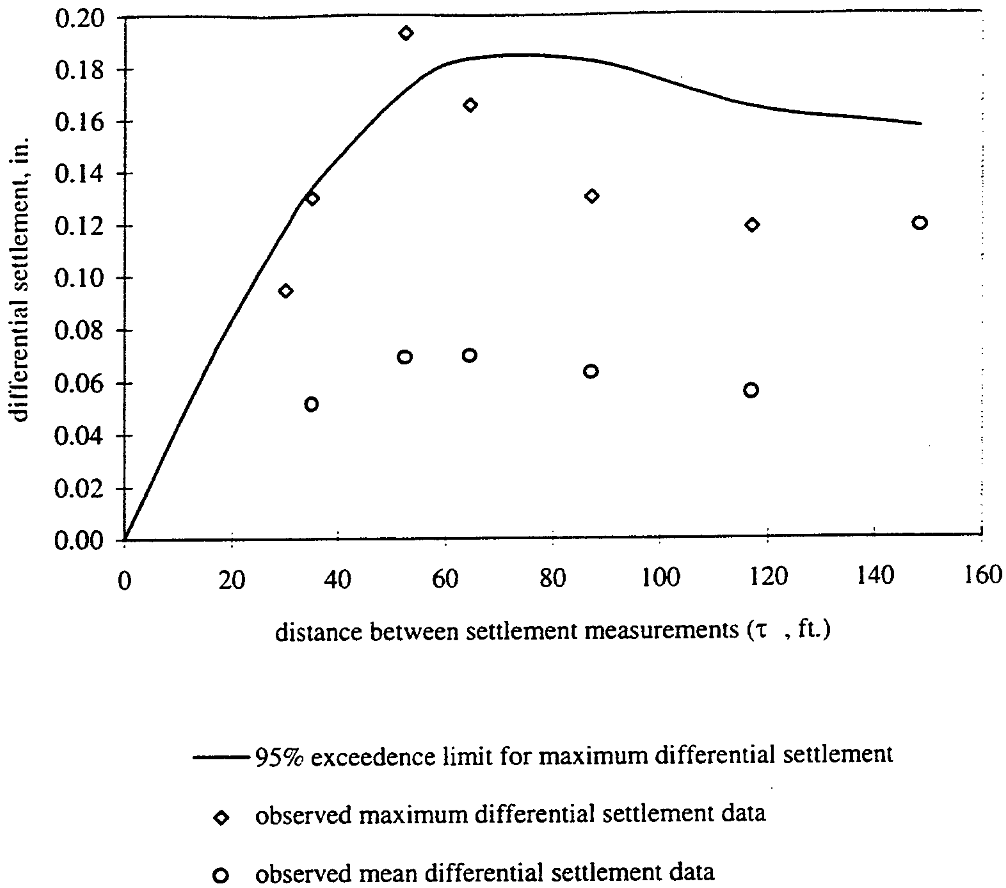


Figure 6-59 Predicted and Observed Differential Settlements - Arts and Commerce Building

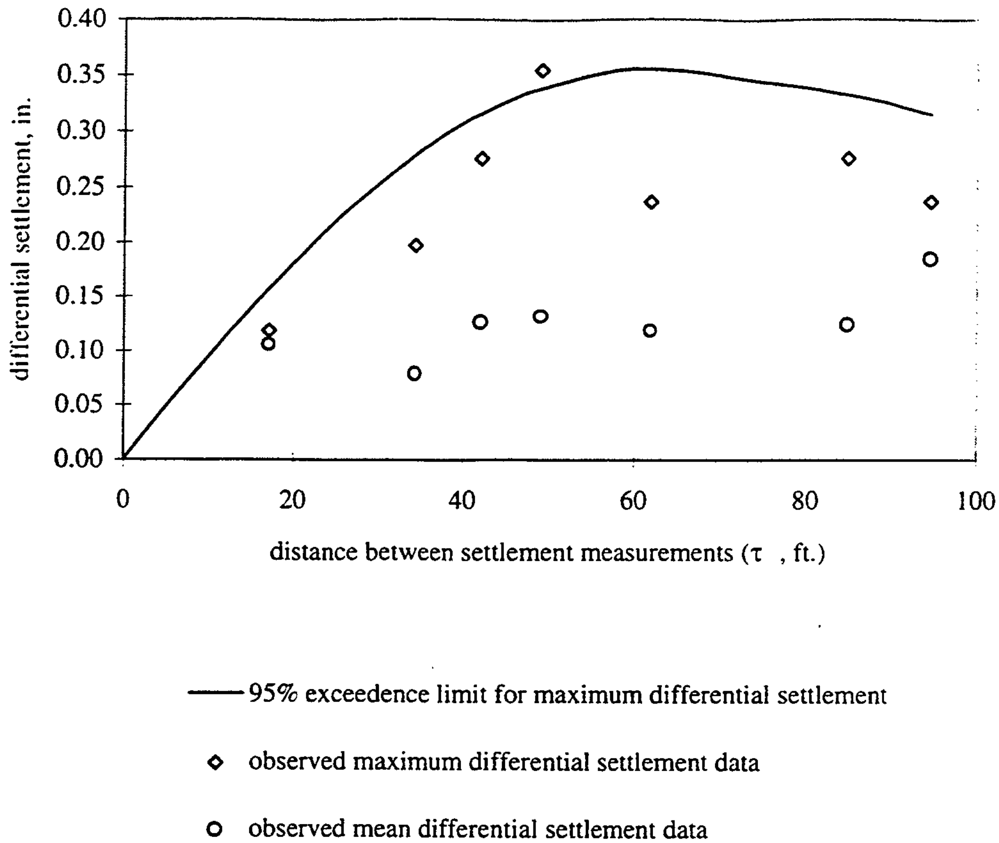


Figure 6-60 Predicted and Observed Differential Settlements - Loppem Bridge

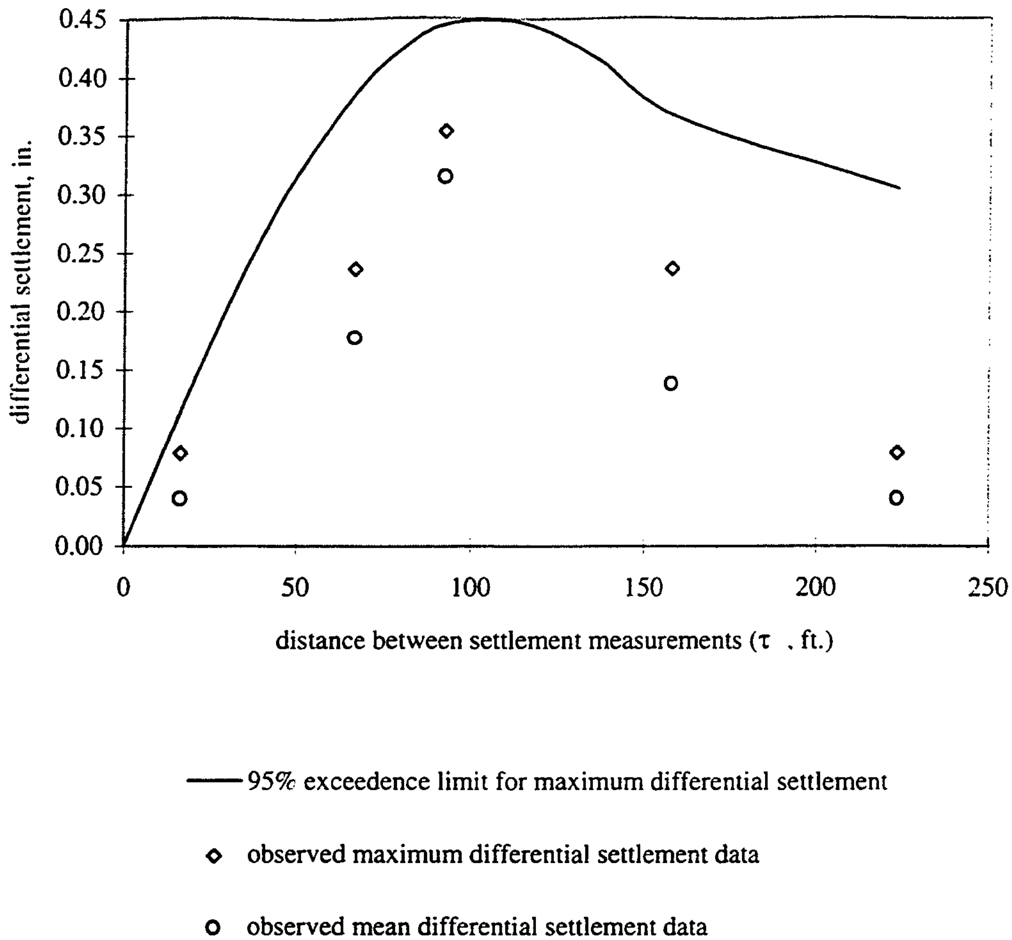


Figure 6-61 Predicted and Observed Differential Settlements - Sterrestreet Bridge

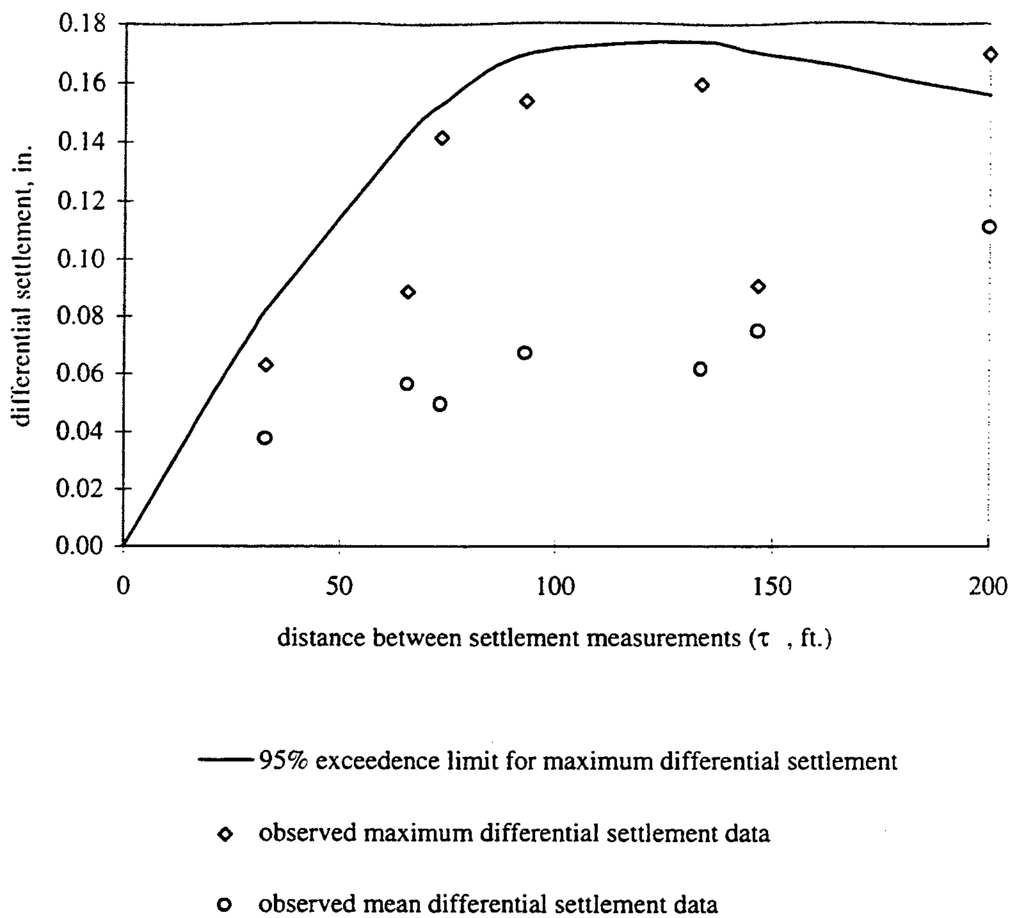


Figure 6-62 Predicted and Observed Differential Settlements - Stratford Bus Station

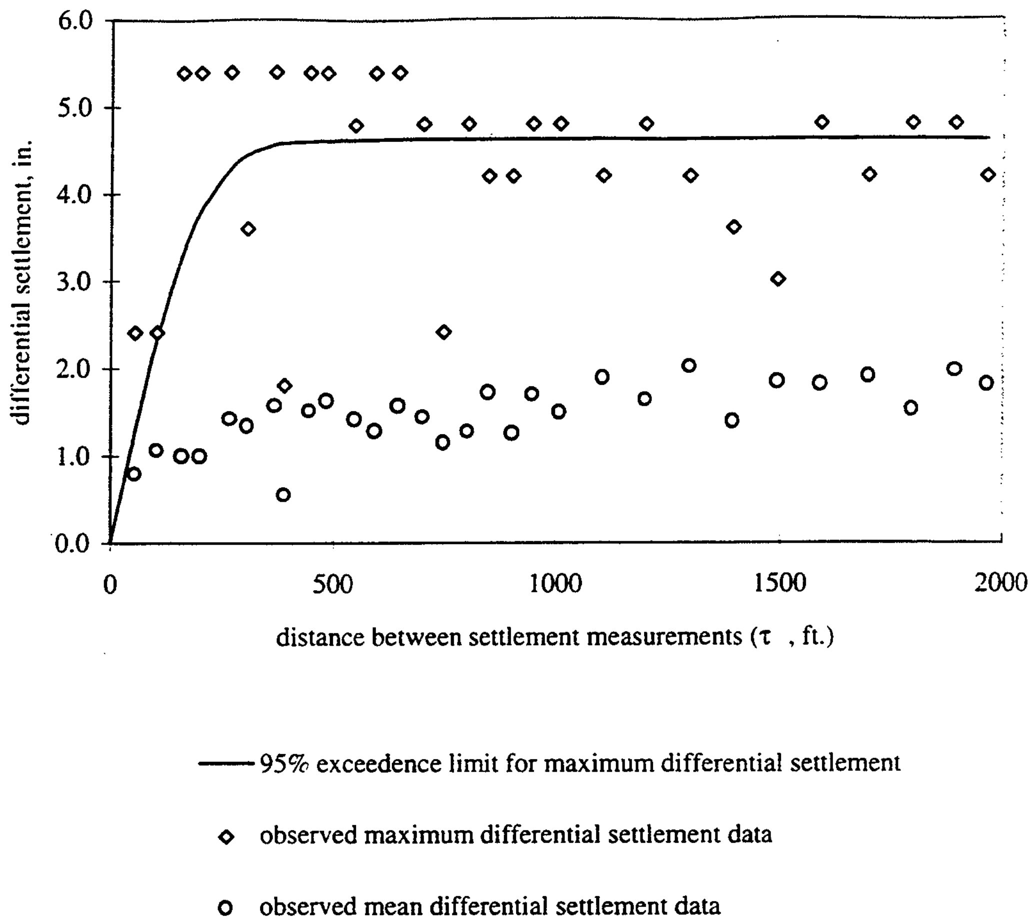


Figure 6-63 Predicted and Observed Differential Settlements - Burlington Bay Skyway

Section 7 LIMITS ON TOLERABLE SETTLEMENTS FOR BRIDGES

One method to mitigate pavement faults is to allow settlement of bridge abutments. Abutments on spread footings bearing on embankment fills will settle with the embankment. This eliminates differences in settlement between approach pavements and bridge decks. A basic reference in this area is DiMillio [1981] who reports good performance for bridges with abutments supported on spread footings. Settlement of abutments can mitigate pavement faults that are due to the global mechanism. Settlement of abutments does not mitigate pavement faults due to a local mechanism.

The use of spread footings for bridge abutments and the expectation that abutments must settle to prevent pavement faults are tolerable only if settlements do not damage the bridge. The capacity for tolerable settlements in bridges limits the magnitude of pavement faults that can be mitigated. This section reviews limits on tolerable settlements of bridges reported by others, and examines the basis for limits on tolerable settlement.

TOLERABLE SETTLEMENTS

Limits on tolerable settlements of highway bridges are reported by Duncan and Tan [1991], by Moulton [et al. 1985], and in a set of papers published as a Transportation Research Record in 1978. These sources variously propose limits on angular distortion, on differential settlements, and on total settlements.

Duncan and Tan [1991] reviewed studies of settlement of highway bridges and proposed limits on angular distortion. Angular distortion is the differential settlement in one span divided by the length of the span.

$$\text{Angular Distorion} = \frac{D}{L} \qquad \text{Eq. 7-1}$$

Duncan and Tan [1991] proposed limits on angular distortion of 0.004 for continuous spans, and 0.008 for simple spans (Table 7-1). Limits on angular distortion were proposed earlier by Moulton [et al. 1985]. Duncan and Tan differ from Moulton in their interpretation of field data on settlements of bridges.

DiMillio [1981] recommended a limit on differential settlement of between 1 and 3 inches. DiMillio reported on settlements of 148 highway bridges supported by spread footings on compacted fill. Of these 148 bridges, 141 were concrete bridges and 7 were steel bridges. None of the bridges showed any sign of functional distress. Differential settlements between the abutment and the adjacent pier were measured for 46 abutments. The mean differential settlement was 1.5 inch.

Several authors have recommended limits on total settlement. Several of papers are the result of a TRB survey of bridge settlements conducted in 1975. Walkinshaw [1978] reported on the 1975 survey using the data collected in seventeen western states. These data included thirty-five structures with fifty-four structural elements, abutments, and piers. Settlement and horizontal movement were evaluated in terms of maintenance requirements. Costly maintenance identified intolerable settlements. Walkinshaw recommends limits of 2.5 inches for total vertical settlement and 2 inches for total horizontal movements.

Bozozok's [1978] report on the 1975 TRB survey included 270 bridge abutments and piers. Of these, 120 were on spread footings, 60 on friction piles, and 90 on end-bearing piles. Settlements were evaluated as being tolerable or intolerable based on the need and cost of maintenance. Bozozuk concluded that bridges are more sensitive to large horizontal movements than to large vertical settlements. For total vertical settlement, Bozozuk classified a 2 inch settlement as tolerable, a 2-3.9 inch settlement as harmful but tolerable, and settlement greater than 3.9 inch as intolerable. For total horizontal movements, Bozozuk classified a 1 inch movement as tolerable, 1-2 inch movement as harmful but tolerable and movement in excess of 2 inch as not tolerable.

Grover [1978] combined data from the 1975 TRB survey with data from a 1961 survey of bridges in Ohio. The 1961 survey included over 1500 bridges, of which 75 had significant settlements. Grover reported that differential settlements between approach slabs and abutments were less for abutments on spread footings than for abutments on deep foundations. At the same time, total settlements of abutments on footings were greater than settlements for abutments on deep foundations. Grover concluded that a 1 inch total settlement was tolerable. Settlements of 2 to 2.8 inches produced minor damage in bridges. Settlements in excess of 3.9 inch were intolerable.

Table 7-2 lists limits on tolerable settlement.

Criteria for tolerable settlement are compared to field studies of settlements of bridges in Table 7-3. Two of the 1978 TRB papers are included (Walkinshaw 1978, Grover 1978). Other studies included in the table are the Burlington Bay skyway [Matich and Stermac 1971], and the work by DeBeer [1948]. Overall, the field studies indicate that approximately 2 inches of total settlement can be tolerated by most bridges.

Source	Maximum Angular Distortion	Classification	Data Set
Moulton et al. 1985	0.004 0.005	Maximum angular distortion for continuous span bridges Maximum angular distortion for simple span bridges	175 bridges (56 simple span, 119 continuous span)
Duncan and Tan 1993	0.004 0.008	Maximum angular distortion for continuous span bridges Maximum angular distortion for simple span bridges	Data from Moulton et al. 1985

Table 7-1 Proposed Limits on Angular Distortion

Source	Total settlement	classification	data set used to determine criteria
Bozozuk 1976	2″ 2″ → 4″ 4″	Tolerable Harmful but tolerable Not tolerable	270 US bridges
Grover 1976	1″ 1″ → 2″ 2″ → 3″ 3″ → 4″ 4″	Tolerable May need maintenance Noticeable to drivers Maintenance needed Objectionable to drivers	More than 1500 bridges. 75 bridges with significant settlement.
Walkinshaw 1976	2.5″	Poor riding quality	35 bridges in 10 western states

Table 7-2 Criteria for Tolerable Total Settlements of Bridges

Author	Study	Observation	Settlement observed Abut. μ in.	Abut. σ^2 in.^{2}	Piers μ in.	Piers σ^2 in.^{2}
Moulton, Gangarao, and Halvorsen 1985	Field study of 314 bridges, 580 abutments and 1068 piers.	Total Settlement	2.58	21.78	0.57	4.46
Walkinshaw 1978	Field study of 54 bridge abutments and piers that moved, a total of 35 structures.	Total Settlement	6.67	38.37	2.50	20.8
Grover, 1978	Field study, 1961 Ohio, on 1525 bridges of which 75 moved significantly. 68 bridges, 133 abutments, are reported. Also, field study in 1975 on 158 abutments on piles.	Total Settlement 1961 study: 1975 study:	 2.24 0.20	 2.43 0.21		
Matich, and Stermac, 1971	Monitoring of the Burlington Bay Skyway Bridge, Canada, up to 150 months after construction.	Total Settlement	2.22	0.18	0.65	0.04
DiMillio 1981	Field study of 148 highway bridges in the state of Washington. 46 abutments supported by spread footings on compacted fill were measured for differential settlements.	Differential settlement Continuous: Simple: Both:	 1.35 2.98 1.56	 1.26 6.41 2.61		
Gifford, Wheeler, Kramer, and McKown 1987	During a period of three years, 21 foundations, 10 bridges, supported by spread footings on cohesionless soil were monitored for settlements, differential settlements and angular distortion in an effort to confirm that spread footing foundations in sand can support bridges well.	Total settlements: Post construction Differential settlement Angular distortion	0.66 0.17 0.42 3E-4	0.24 0.01 0.18 1E-7	0.53 0.26 0.30 2E-4	0.08 0.10 0.07 3E-8
DeBeer, 1948	During the years 1939-1947, eight concrete bridges in Belgium were monitored for settlements from beginning of construction to up to seven years after construction.	Total Settlement Construction: Long Term	 0.71 0.82	 0.30 0.41	 0.19 0.36	 0.03 0.15
Note: μ = mean value σ^2 = variance Mean and variance calculated on spread sheet from information given by author except for *			h = horizontal movement r = vertical settlement d = differential settlement			

Table 7-3 Data on Settlement of Bridges

USING LIMITS ON TOLERABLE SETTLEMENTS

If the tolerance of bridges for settlement becomes the basis for mitigation of pavement faults, then settlements must not exceed the accepted limits on angular distortion. During design, engineers must compute the expected value of differential settlements and the expected angular distortion and compare these to limits on settlement and distortion. The outcome of such a check depends on the estimate of differential settlements. In particular, a probabilistic approach to estimating differential settlement, and the consideration of spatial correlation in settlements will be useful here.

SIMPLE METHOD USING RULE-OF-THUMB ESTIMATE OF DIFFERENTIAL SETTLEMENTS

Differential settlements are estimated as a fraction of total settlements, and therefor a limit on angular distortion is effectively a limit on total settlement. If the largest settlements are expected at abutments on spread footings, then a limit angular distortion imposes a limit on total settlement of embankments.

As an example, consider the deterministic estimate that differential settlements are equal to 50% of total settlements.

$$D = 0.5S \quad \textbf{Eq. 7-2}$$

where D is differential settlement and S is total settlement. This estimate is valid only if all substructures are supported on spread footings. If abutments are on spread footings and piers are on piles, then differential settlements could be 100% of the total settlement of the abutment.

For a continuous span bridge, the angular distortion limit is 0.004, and so the upper bound on differential settlements is computed from the distortion limit and the span length.

$$\frac{D}{L} \leq 0.004 \quad \textbf{Eq. 7-3}$$
$$D \leq 0.004L$$

Finally, substituting the relation between D and S, the upper bound on total settlement for the abutment is found.

$$S \leq 0.008L \quad \textbf{Eq. 7-4}$$

This single result is valid only for continuous bridges with all substructures on shallow foundations. Limits on total settlement for simple spans, or for bridges with piers on deep foundations can be found by similar relations.

The limit on total settlement S for this case is plotted in Figure 7-1. Overall, the bound on total settlement is large. Short spans tolerate only small settlements. Short structures are very sensitive to differential settlements.

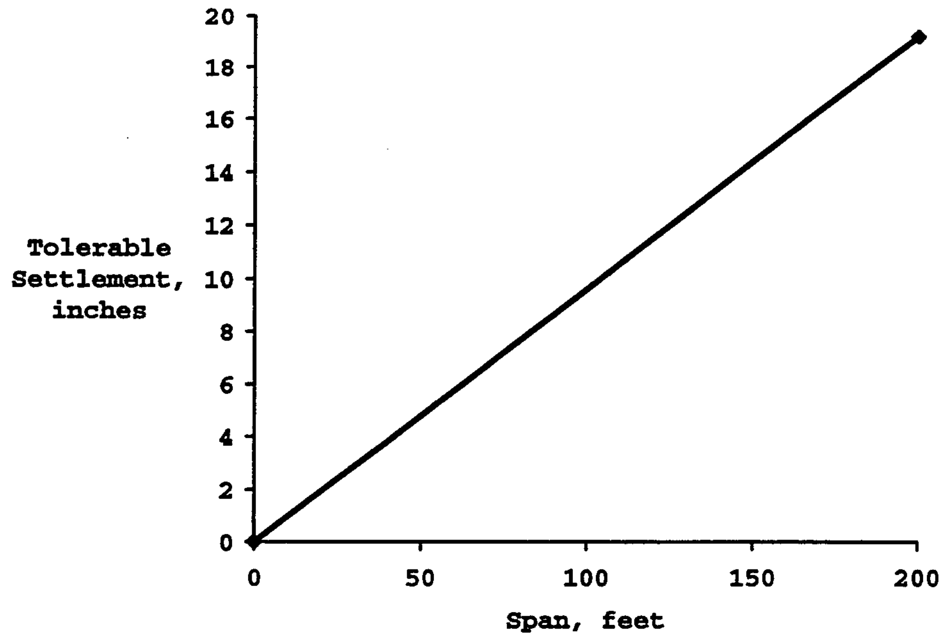


Figure 7-1 Limit on Total Settlement. Rule of Thumb.

PROBABILISTIC ESTIMATE OF DIFFERENTIAL SETTLEMENTS

If probabilistic estimates of differential settlements are used, a different and potentially larger total settlement may be tolerated. Assuming independent settlement among similar foundations, the relation between the mean value of differential settlements and mean total settlements is

$$\mu_D = 1.13\sigma_S$$
$$\mu_D = 1.13COV_S\mu_S \qquad \textbf{Eq. 7-5}$$

where COV_S is the coefficient of variation of total settlements. Using a limit of 0.004 for tolerable angular distortion, the upper bound on mean differential settlements is computed.

$$\mu_D \leq 0.004L \qquad \textbf{Eq. 7-6}$$

And the limit on mean total settlement is computed.

$$\mu_S \leq \frac{0.0035L}{COV_S} \qquad \textbf{Eq. 7-7}$$

This limit on total settlement is generally larger than the rule-of-thumb limit. But this limit on mean total settlement is based on mean differential settlements. A design limit requires a conservative estimate of differential settlements. For independent foundations, 90% of all differential settlement will be less than $2.3\sigma_S$. This conservative limit is expressed in terms of the mean total settlement.

$$D_{90} = 2.3\sigma_S$$
$$D_{90} = 2.3COV_S\mu_S \qquad \textbf{Eq. 7-8}$$

And the limit on total settlement is computed.

$$\mu_S \leq \frac{0.0017L}{COV_S} \quad \textbf{Eq. 7-9}$$

This conservative limit on total settlements is plotted in Figure 7-2 for three values of COV_S. The plot also shows the limit based on a rule-of-thumb estimate of differential settlements. The conservative, probabilistic limit on total settlement may be greater or lesser than the rule of thumb limit depending on the variability of total settlements.

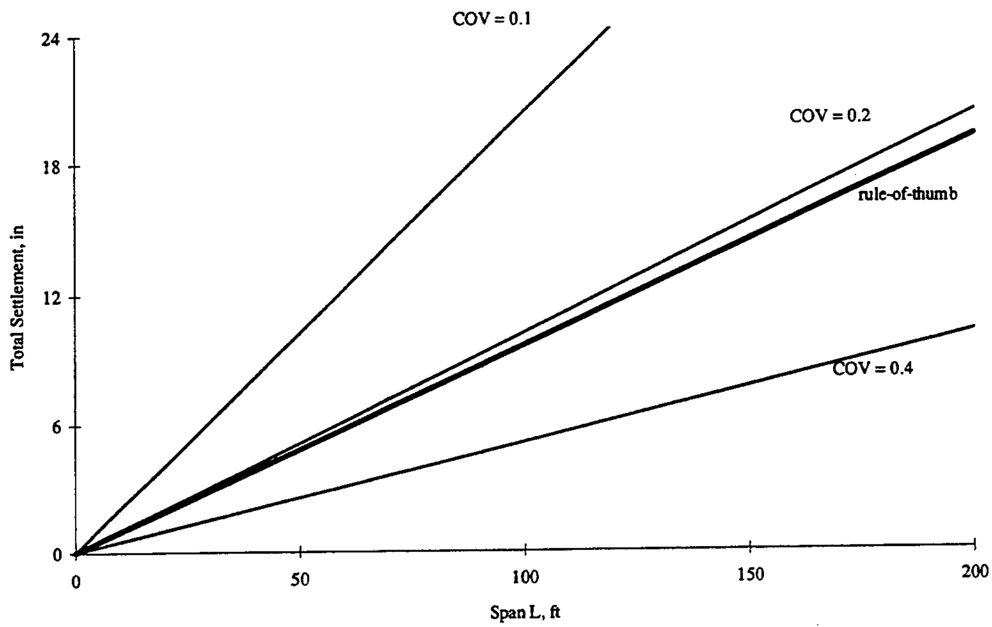


Figure 7-2 Probabilistic Limits on Total Settlement

PROBABILISTIC ESTIMATE OF DIFFERENTIAL SETTLEMENTS WITH SPATIAL CORRELATION

If settlements are correlated in space, then differences in settlements are less for nearby foundations. Estimates of differential settlements depend on the variability in total settlements and on the relative distance between foundations. Estimates of mean differential settlements when total settlements are correlated are

$$E[|D|] = 1.13\sigma_S\sqrt{1-\rho(\tau)} \quad \textbf{Eq. 7-10}$$

The conservative estimate that includes 90% of all differential settlements is

$$E[|D|] = 2.3\sigma_S\sqrt{1-\rho(\tau)} \quad \textbf{Eq. 7-11}$$

Differential settlements are related to mean total settlements by the coefficient of variation.

$$E[|D|] = 2.3COV_S\mu_S\sqrt{1-\rho(\tau)} \quad \textbf{Eq. 7-12}$$

The limit on mean total settlement then becomes

$$\mu_S \leq \frac{0.0017L}{COV_S\sqrt{1-\rho(\tau)}} \quad \textbf{Eq. 7-13}$$

In Figure 7-3, the limits on mean total settlement are shown for three values of COV_S using an assumed correlation function of

$$\rho(\tau) = e^{\left(-|\tau|/\beta\right)} \qquad \textbf{Eq. 7-14}$$

where β is taken equal to 40 feet. For large distances between foundations, the limits here are the same as the limits for independent settlements. For small distances between foundations, limits on settlement are higher when correlation is present. Notice that the limit on total settlement does not go to zero as the span length goes to zero. This is an important, and realistic, outcome of spatial correlation in settlements.

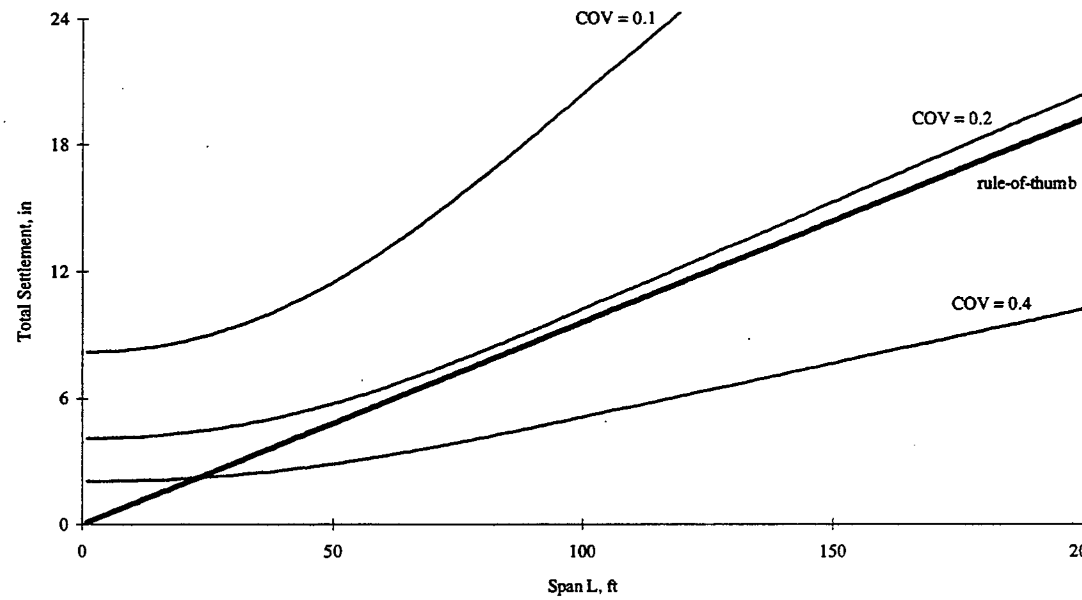


Figure 7-3 Limits on Total Settlement with Spatial Correlation

LIMITS ON NORMALIZED TOTAL SETTLEMENTS FOR HIGHWAY BRIDGES.

The probabilistic estimates of differential settlement are used to develop a new limit on mean total settlement in a simple, dimensionless form. Here, the capacity for angular distortion for bridges is taken equal to the distortion limits proposed by Duncan and Tan [1991]. The limit on the ratio of mean total settlement to span length for simple spans is shown in Figure 7-4. The limit for continuos spans is shown in Figure 7-5.

The limits for angular distortion with spatial correlation are

Simple Bridges

$$\frac{\mu_S}{L} \leq \frac{0.008}{2.3COV_S\sqrt{1-\rho(\tau)}} \qquad \textbf{Eq. 7-15}$$

Continuous Bridges

$$\frac{\mu_S}{L} \leq \frac{0.004}{2.3COV_S\sqrt{1-\rho(\tau)}} \qquad \textbf{Eq. 7-16}$$

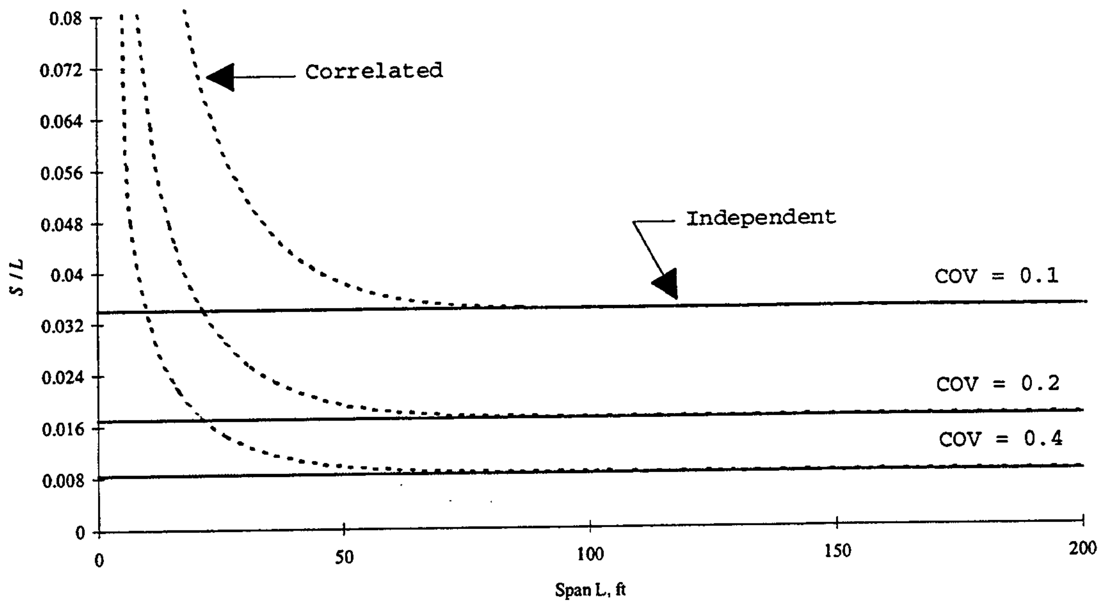


Figure 7-4 Settlement to Span Limits - Simple Bridges

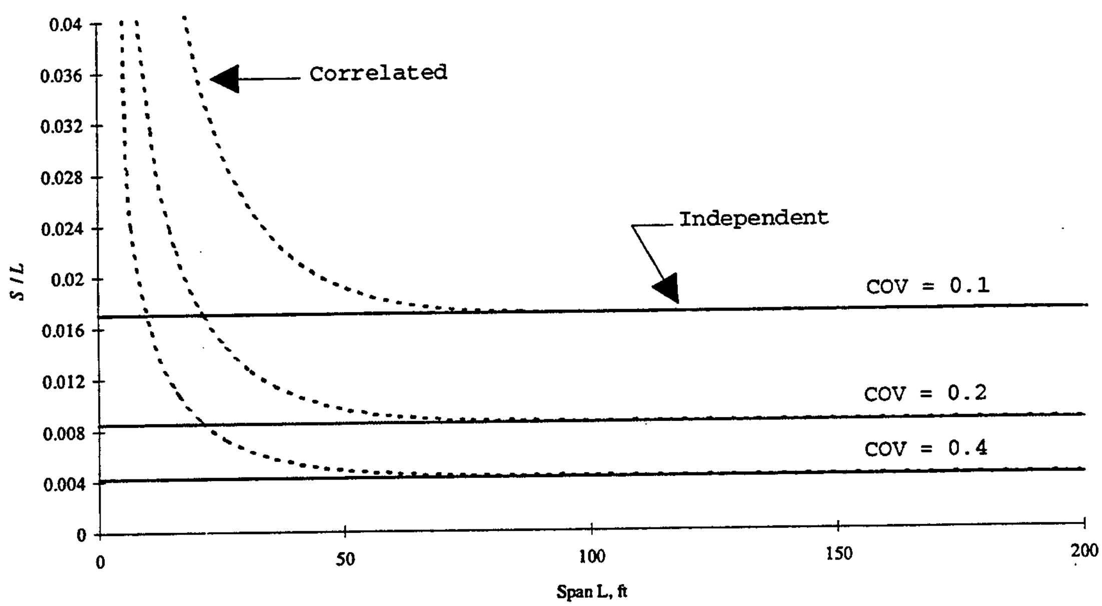


Figure 7-5 Settlement to Span Limits - Continuous Bridges

STRESS ANALYSIS FOR TOLERABLE SETTLEMENT LIMITS

Moulton [et al. 1985] studied the tolerance of bridges for settlements using both field observations and stress analysis. Moulton used elastic analysis mainly. Moulton also computed the inelastic response of prestressed concrete bridge beams. He had found that elastic analysis results for prestressed beams differed strongly from field observations of capacity for settlement. Bridges in service tolerate much larger settlements than elastic analysis indicates. Moulton's analysis of the effect of creep in prestressed concrete beams demonstrated that the tolerance for settlement can be as much as 300% greater than that indicated by elastic analysis alone. Moulton did not report an inelastic analysis for steel bridge beams. The capacity of steel bridge beams is examined here.

Elastic analysis is used for allowable stress design for steel bridge beams. This requires that the sum of stresses imposed by loads, by temperature effects and by settlement remain below a total value of settlement that is a specific fraction of the yield stress of the steel. This design relation may be shown as

$$a\sigma_y \geq \Sigma\sigma_i \qquad \textbf{Eq. 7-17}$$

$$a\sigma_y \geq \sigma_D + \sigma_L + \sigma_I + \sigma_T + \sigma_\delta \qquad \textbf{Eq. 7-18}$$

where σ_y is the yield stress, a is a constant less than 1.0, and σ_i are the computed stresses from dead load, live load, impact, temperature and settlements. For routine bridge design, there may be no explicit analysis for stresses due to settlement. Instead, the selection of a steel beam is based on the set of stresses due to loads and temperature effects. The tolerance for settlement then is limited to the excess stress capacity that the beam may have.

$$\sigma_\delta \leq \sigma_y - (\sigma_D + \sigma_L + \sigma_I + \sigma_T) \qquad \textbf{Eq. 7-19}$$

Steel bridge beams often have somewhat greater strength than the minimum required for the design. The amount of excess strength varies from design to design, and can be nearly zero for some designs.

In an allowable stress design, stresses due to settlements σ_δ are elastic stresses. For a two-span continuous bridge beam with equal span lengths L, with a prismatic section and with settlement at only one abutment, the greatest stress due to settlement occurs at the pier. The stress due to settlement σ_δ is computed as

$$\sigma_\delta = \frac{3}{4}E\frac{\delta}{L}\frac{d}{L} \qquad \textbf{Eq. 7-20}$$

where E is the elastic modulus, δ is the settlement of the abutment, and d is the depth of the beam. The expression includes the angular distortion term δ/L, and also the depth to span ratio. The use of a narrow range of depth to span ratios in US bridges leads to a single limit on angular distortion as proposed by Moulton [et al. 1985] and by Duncan and Tan [1991]. Eq. 7-17 yields the limit on δ/L as a function of σ_δ. These are shown in Table 7-4, for d/L equal to 20. Limits obtained by Moulton using elastic analysis are shown in Table 7-5.

σ_δ (ksi)	δ/L
1	0.0009
2	0.0018
3	0.0028
4	0.0037

Table 7-4 Examples of Elastic Limits on Angular Distortion

Span	Two-Span Bridges Settlement of One Abutment		Four-Span Bridges Settlement of First Pier	
ft	δ (in)	δ/l	δ (in)	δ/l
30	1.13	0.00310	0.90	0.00250
50	0.44	0.00074	0.48	0.00081
100	1.50	0.00125	1.04	0.00087
150	1.14	0.00063	0.35	0.00020
200	0	0	1.20	0.00050
250	0.43	0.00014	0	0

Table 7-5 Settlement Limits for Steel Bridges
Elastic Analysis (Mouton et al. 1985)

Moulton's results for elastic analysis of bridge beams are compared to observed angular distortion in bridges in Table 7-6. The limits proposed by Moulton are 60% to 80% less than the mean settlements that were observed to be damaging to bridges.

Category	Angular Distortion a/l		
	Observed Mean Tolerable	Observed Mean Intolerable	Proposed Limit
All Bridges	0.0025	0.0161	
Simple Bridges	0.0031	0.0241	0.005
Continuous Bridges	0.0022	0.0129	0.004
Concrete Bridges	0.0024	0.0232	
Steel Bridges	0.0026	0.0138	

Table 7-6 Field Data from Moulton [et al. 1985]

Elastic analysis leads to limits on angular distortion of bridges that are much lower than the distortions that real bridges are observed to tolerate without distress. The stark disparity between analysis and field observation apparently led Moulton to rely on field data alone to propose limits on angular distortion.

The inelastic response of bridge beams is important. Moulton recognized this, and examined the effect of creep in prestressed concrete bridges. Table 7-7 lists ratios of Mouton's results for inelastic and elastic analyses. For gradual settlement of supports, the inelastic capacity of prestressed concrete bridge beams can be more than 300% the capacity indicated by elastic analysis alone.

Two-Span Continuous P/S Girders

Span (ft)	Relative Settlement Tolerance
75	1.6
100	3.1
125	3.1

Table 7-7 P/S Girders, Increased Settlement Tolerance due to Creep (Moulton et al. 1985)

Moulton's findings on the use of elastic analysis for the study of tolerable settlements in bridges can be summarized as follows

- The elastic tolerance for settlement in bridge beams is determined by the level of additional stress that the beam can carry. For steel bridge beams, the tolerance for settlements is stated as a limit on angular distortion, *a/l* that is approximately 0.001. Steel beams that are well matched to their load demands (beams with little excess strength) have little and possibly no tolerance for differential settlements by an elastic criterion.
- For prestressed concrete bridge beams, there is a similar stress-derived limit on sudden settlements, that is, settlements where creep in the concrete is not considered. If creep is considered then the tolerance for settlements may increase by a factor of three.
- For prestressed concrete beams there is no simple statement of tolerance for settlements expressed as angular distortion. Stress due to settlements depends on the order of construction, on the excess strength of the beam, and on the occurrence and rate of creep in concrete.
- Observed settlements in bridges and damage in bridges due to settlement indicate that bridges in service tolerate greater differential settlements than elastic analyses indicate. The range of tolerable to intolerable angular distortion observed in bridges in service is from 250% to 1400% greater than elastic analysis results. Mouton's recommendation on tolerable angular distortion is 500% greater than can be justified on the basis of elastic analysis.

INELASTIC ANALYSIS OF TOLERABLE SETTLEMENT FOR STEEL BEAMS

Settlement can cause increased stresses in steel bridge beams and may cause plastic rotations. Plastic rotations are acceptable if they are one-time, one-direction events. This is evident in the 1991 AASHTO guide specifications for the design of steel bridge breams using a plastic analysis approach [*Guide* 1991]. In the AASHTO approach, plastic rotations are the mechanism of a redistribution of bending moments along the length of a bridge beam. AASHTO provides for the reduction of negative moments at the supports of continuous beams and a corresponding increase of positive moments in spans. The design method is restricted to steel beam that are compact and that are adequately braced.

Plastic capacity in steel is both a mechanism for redistribution of bending moments and a mechanism for deflection. Redistribution and deflection are two aspects of flexural yielding in steel sections. For the strength design of steel bridge beams, plastic rotations must result in a set of bending moments under design loads that do not exceed the strength of the beam. The effective strength of a beam after redistribution is a function of the slenderness of web and flange plates. Redistribution must not cause undue deflections. For strength design, deflections are a by-product of plastic rotations and moment redistribution.

A complementary procedure can be proposed for the design of continuous steel bridge beams to tolerate settlements of supports. In a direct approach to design for settlements, steel bridge beams are sized to resist all loads elastically. The full amount of the plastic rotation capacity is available for the tolerance of settlements of supports. For bridge beams that carry all loads elastically, the tolerance for settlement of the beam is a function only of the plastic rotation capacity of the beam. Rotation capacity in compact steel beams can be large.

An inelastic approach to the design of bridge beams for settlement recognizes the potential for yielding and for plastic rotation of beams. The relation between settlements and plastic rotation is

$$\delta = \theta_\delta \times L \qquad \text{Eq. 7-21}$$

where δ is the settlement, θ_δ is the plastic rotation of the bridge beam and L is the span length. The rotation θ_δ that is available depends on the form of the steel beam and on the demand for rotation capacity for other purpose such as redistribution of loads in the beam. The limit state for plastic rotations for the beam can be written as

$$\phi\theta_n \geq \theta_d + \theta_M \qquad \text{Eq. 7-22}$$

where θ_n is the rotation of the steel beam at failure, ϕ is a reduction factor, and θ_M is the rotation required to distribute moments in the beam. The term θ_M is zero if the beam carries loads elastically, and no redistribution of moments is required. Combining Eq. 7-21 and Eq. 7-22, the tolerance for settlements of a continuous steel bridge beam is

$$\delta \leq (\phi\theta_n - \theta_M) \times L \qquad \text{Eq. 7-23}$$

ROTATION CAPACITY OF STEEL BEAMS

Steel bridge beams have a capacity for plastic rotation that is a function of the slendernesses of the flange and web plates, and of the spacing of lateral bracing of the compression flange. A steel beam may have no plastic rotation capacity if its parts are too slender or if bracing is not adequate. Plastic rotation capacity, and the allowance of plastic rotations in civil structures is not new. Plastic approaches to design of steel beams and frames appeared in the 1950s and 1960s. The American Society of Civil Engineers a manual on plastic design in 1961 and the American Institute of Steel Construction published a guide to plastic design of frames in 1968. AASHTO recognizes (and allows) a plastic design approach for steel bridge beams (Standard 1994), and inelastic procedures for load rating using a plastic analysis approach are reported in NCHRP — 1992.

An adequate capacity for plastic rotation is assured by the imposition of slenderness limits on the beam cross section and on the conditions of bracing as

$$\theta_n = \theta_n(\lambda_f, \lambda_w, \lambda_L) \qquad \text{Eq. 7-24}$$

where λ_f is the slenderness ratio for the flange, λ_w is the slenderness ratio for the web, and λ_L is the lateral slenderness of the beam that is a function of both the beam cross section and the distance between points of bracing.

The common definition of the rotation capacity of steel beams is [*Commentary* 1961]

$$Ru = \frac{\theta p}{\theta y} = \frac{\theta u - \theta y}{\theta y} = \frac{\theta u}{\theta y} - 1 \qquad \text{Eq. 7-25}$$

where R_u is the rotation capacity, θ_p is the plastic rotation and θ_y is the elastic rotation, and θ_u is the ultimate rotation capacity of the beam.

NONCOMPOSITE BEAMS

Kemp and Dekker [1991] investigated the ability of compact and noncompact steel beams to redistribute moments in regions of linear moment gradient, such as in regions adjacent to internal supports in continuous beams. Based on this investigation they proposed a prediction for rotation capacity of steel beams.

$$\lambda_e = K_f K_w \left(\frac{L_i}{r_y \varepsilon} \right) \qquad \textbf{Eq. 7-26}$$

where

$$K_f = \left(\frac{b_f}{20 t_f \varepsilon} \right)$$

$$K_w = \left[460 - \left(\frac{L_i}{r_y \varepsilon} \right) \right] \frac{\sqrt{K_{w2}}}{400} \qquad \text{for } (\alpha h_w)/(t_w \varepsilon) < 33$$

$$K_{w2} = \left(\frac{\alpha h_w}{33 t_w \varepsilon} \right) \qquad \text{for } 33 < (\alpha h_w)/(t_w \varepsilon) < 40$$

$$\varepsilon = \sqrt{\frac{34.08}{F_y}} \qquad F_y \text{ ksi}$$

where L_i is the length from the point of maximum negative moment to the point of zero moment, r_y is the radius of gyration about the minor axis, F_y is steel yield stress in ksi, ε is the ratio of nominal to actual yield stress, b_f is the width of the flange, t_f is flange thickness, h_w is web depth, t_w is web thickness and α is the ratio of the web depth in compression. Using effective lateral slenderness λ_e, Kemp and Dekker proposed a relation for ultimate rotation capacity.

$$Ru2\alpha = 3\left(\frac{60}{\lambda e} \right)^{1.5} \qquad \textbf{Eq. 7-27}$$

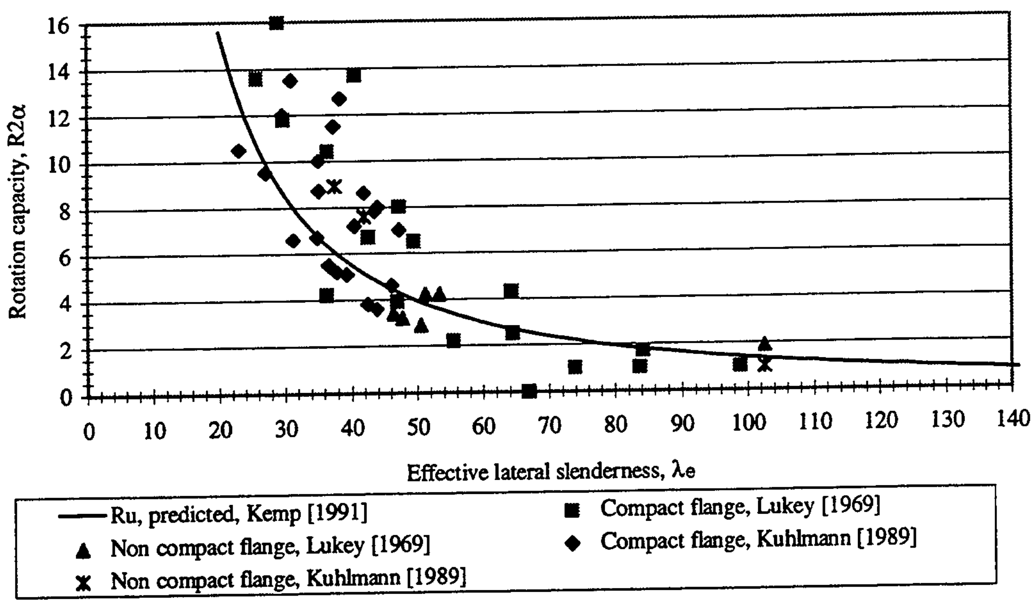


Figure 7-6 Kemp and Dekker Model Compared to Test Data

Kemp and Dekker developed their model using data from forty-four tests of rotation capacity of steel beams, include tests by Lukey and Adams [1969], by Kemp [1985] and by Kuhlmann [1989]. All the beams had flange ratios, $b_f/(2t_f\varepsilon)$, less than 11.05 and web ratios, $\alpha h_w/(t_w\varepsilon)$, less than 40. A comparison of tests and the Kemp and Dekker model is plotted in Figure 7-6. The Kemp and Dekker model offers a reasonable and often conservative estimate of rotation capacity.

Composite beams and the Kemp and Dekker Model

The model proposed by Kemp and Dekker [1991] is intended for noncomposite steel beams. Rotation capacity of composite steel beams will differ because of the change in web slenderness. In positive bending of composite sections, the major portion of the web is in tension, and can not buckle. For positive bending, $\alpha < 0.5$, the web is less slender compared to a noncomposite beam, and the rotation capacity of the section is increased. For negative bending in composite sections the situation in reversed. The major portion of the web is in compression, α is greater than 0.5, the web is more slender, and the rotational capacity of the section is reduced.

There are few tests of the rotation capacity of steel-concrete composite beams. More often, the rotational capacity of composite steel beams has been studied using steel beams that are made asymmetric by the addition of steel cover plates. The resulting shift in the neutral axis in the web is equivalent to the conditions in composite steel sections. Tests of steel-concrete composite beams and beams with cover plates are reported by Climenhaga and Johnson [1972] and by Hope-Gill and Johnson [1976].

A comparison of the Kemp and Dekker model with tests of composite beams and of beams with cover plates are plotted in Figure 7-7. All of the steel-concrete composite beams, except for one, have tested rotation capacities greater than those predicted by Kemp and Dekker. The one composite beam that did not perform better than the Kemp and Dekker model had a web slenderness, $K_W > 40$. For the steel beams with cover plates, the same trend is observed. All predicted rotational capacities are lower than observed capacities except for beams with a web slenderness, $K_W > 40$. For steel beams with $K_W > 40$, there is nearly zero rotational capacity.

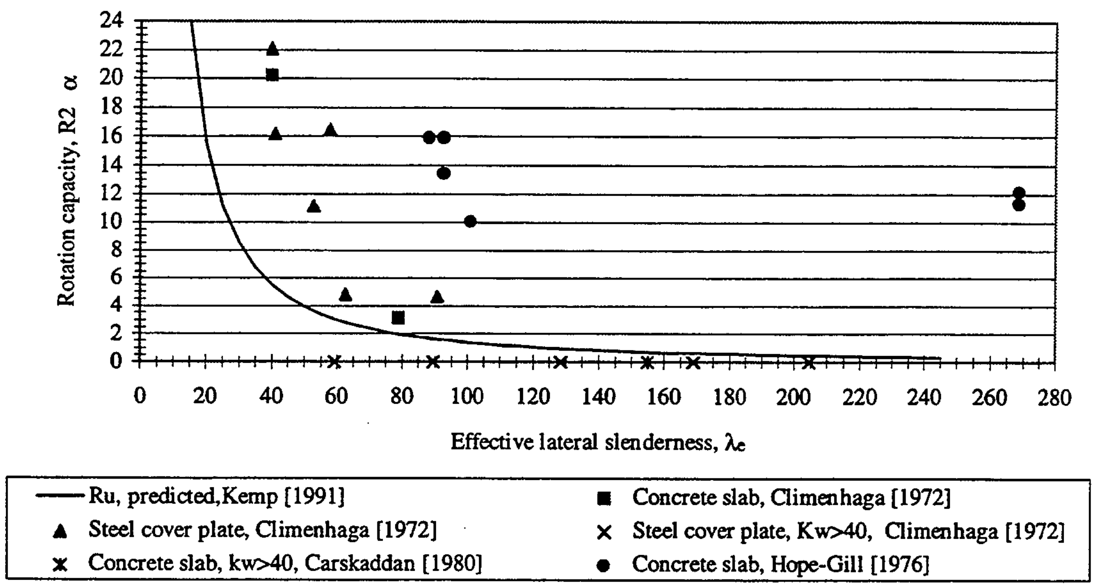


Figure 7-7 Kemp and Dekker Model Compared to Tests of Composite Beams

ROTATION LIMITS ASSOCIATED WITH LOCAL FLANGE BUCKLING

In tests reported by Lukey and Adams [1969], local flange buckling preceded local web buckling and lateral torsional buckling. On average, the plastic rotation at the onset of local flange buckling was about 30 percent of the ultimate rotational capacity. Kuhlmann [1989] also observed that local flange buckling preceded other buckling modes.

A limit on tolerable rotation capacities, R_T, is set slightly less than the rotation at the onset of local flange buckling. Using 20 percent of the ultimate rotation capacity predicted by the Kemp and Dekker model, a lower bound on observed rotations at the point of local flange buckling is obtained (Figure 7-8). This lower bound is valid when the web slenderness, K_w, does not exceed 40.

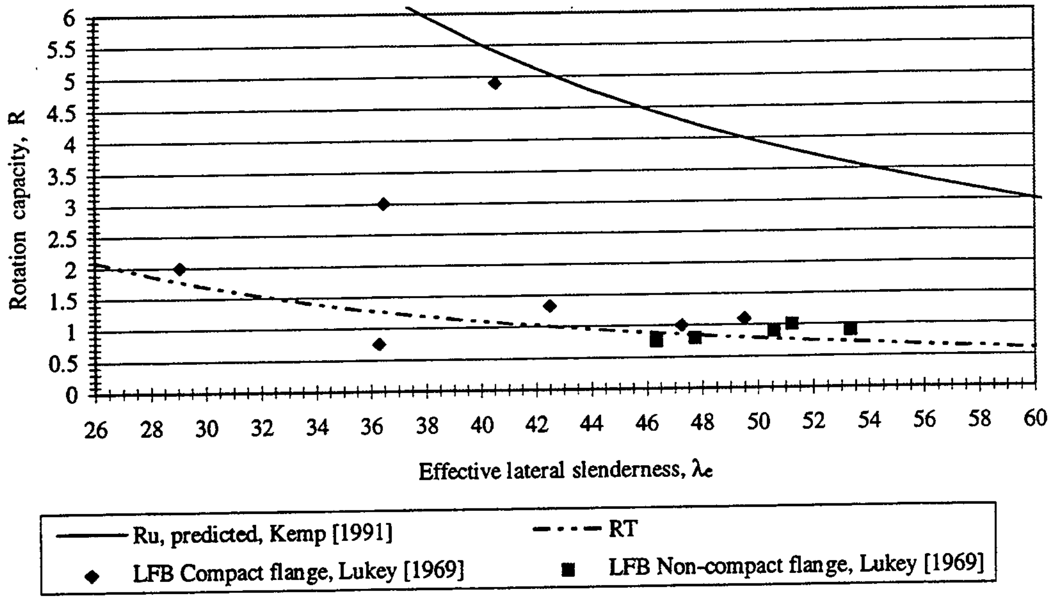


Figure 7-8 Local Flange Buckling in Noncomposite Steel Beams

Tests of composite beams in negative bending indicate that local flange buckling occurs before other buckling modes. In Figure 7-9, rotations in tests at the onset of local flange buckling are compared to 20% Ru from the Kemp and Dekker model. The (reduced) rotation capacity of the model is conservative for all beams with adequate web slenderness.

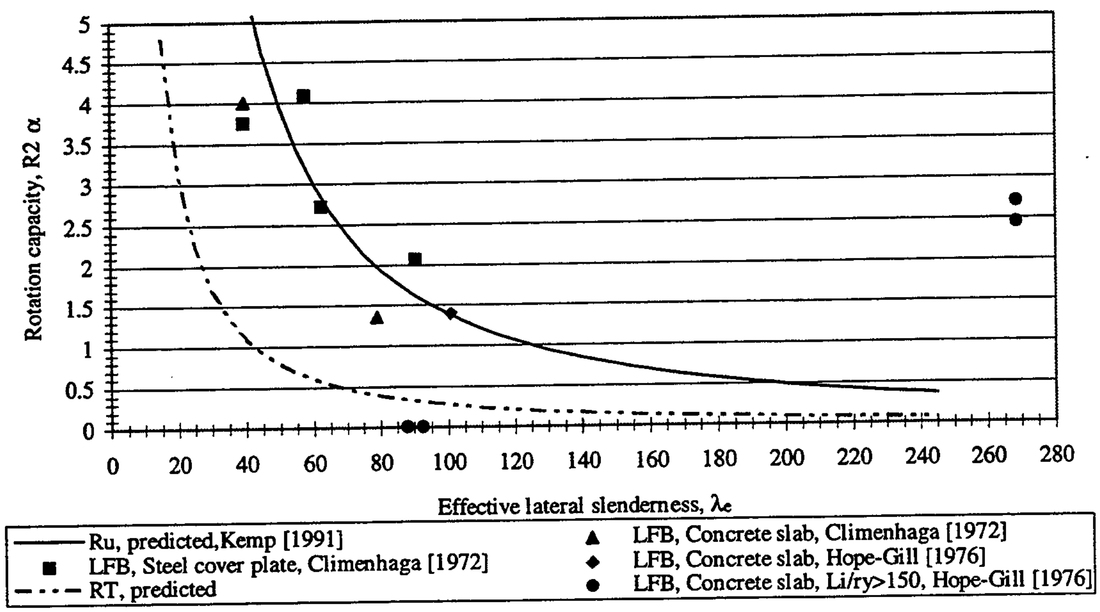


Figure 7-9 Local Flange Buckling in Composite Beams

DESIGN CAPACITY FOR INELASTIC ROTATIONS

The tolerable rotation capacity of noncomposite and of composite steel beams can be estimated as 20 percent of the ultimate rotation capacity if controlling slenderness ratios for webs, flanges and bracing are met. Tolerable rotation capacity R_T can be written as

$$R_T = 0.2\,Ru = 0.2\left(3\left(\frac{60}{\lambda e}\right)^{1.5}\frac{1}{2\alpha}\right) = \frac{3}{10\alpha}\left(\frac{60}{\lambda e}\right)^{1.5} \qquad \textbf{Eq. 7-28}$$

constraints $\quad K_w < 40$

$$b_f / 2t_f < \frac{65}{\sqrt{Fy}}$$

$$L_i / r_y < 150$$

SETTLEMENT CAPACITY OF STEEL BRIDGE BEAMS - EXAMPLES

The tolerance for settlement of steel bridge beams is computed for six rolled shapes. These are examples of the range of tolerance of settlement that is available from inelastic response. The rolled shapes are compact shapes with nominal depths from 30 inches to 36 inches. Both relatively heavy and relatively light sections are chosen at each depth. The beams are assumed to be used as two-span continuous beam bridges. The span to depth ratio is taken as 25. Beams are sized to carry all loads elastically. This leaves all of the plastic rotation capacity available to accommodate settlements.

Beams, properties of beams, and capacity for settlements are shown in Table 7-8. All beams are compact according to AASHTO criteria. All webs have slenderness ratios about half that required by AASHTO for compact sections. The tolerable rotation θ_T is shown. These inelastic values of θ_T are 5 to 10 times greater than the angular distortion values determined by elastic analysis. The tolerable differential deflections, *D*, are shown in the last column. For heavy beams, very large differential settlements may be tolerated. For lighter beams, the tolerance for differential settlements is less, but it is still a significant tolerance and it is larger than the limits on differential settlements developed from observations of bridges in service.

Beam	Span, ft	b_f/t_f	h_c/t_w	R_T	θ_T, rad	*D*, in
W 30 x 90	62	8.5	57.5	0.6	0.0038	2.9
W 30 x 148	62	4.4	41.5	2.4	0.0146	10.9
W 33 x 118	69	7.8	54.5	0.7	0.0045	3.8
W 33 x 169	69	4.7	44.7	2.0	0.0124	10.3
W 36 x 135	75	7.6	54.1	0.7	0.0044	4.0
W 36 x 256	75	3.5	33.8	3.7	0.0231	20.8

Table 7-8 Examples of Settlement Capacity of Steel Beams

SUMMARY

Limits on angular distortion, on differential settlements, and on total settlements for highway bridges have been proposed by various authors. Limits on angular distortion, first proposed by

Moulton [et al. 1985] and modified by Duncan and Tan [1991] are the most often cited and used today.

The methods for estimating differential settlement developed in Section 5 are used to establish limits on total settlement that a bridge can accommodate. Spatial correlation of settlements can be included as well. Spatial[correlation can have a large effect on limits on total settlement for short spans.

Criteria for tolerable settlements of bridges are empirical. Observations of bridges in service are the basis for accepted limits on angular distortion in bridges. These limits are associated with poor riding quality, with damage to pavements or with damage to joints and railings. Damage to bridge beams or loss of strength of superstructures due to settlement is rarely a problem.

Elastic analysis of bridge beams for tolerance to settlements yields very tight limits on settlement. The result of elastic analysis do not agree with observations of bridges in service. Inelastic analysis yields much better agreement.

The application of inelastic analysis of steel bridges beams to compute their tolerance to settlement is developed in this section. It is found that the inelastic analysis gives results that are consistent with observed performance of bridges.

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