Bridge Approach Slab Effectiveness

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Research Project GC 8286, Task 35
Bridge Approach Slab Effectiveness

BRIDGE APPROACH SLAB EFFECTIVENESS

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16. ABSTRACT

Settlement of roadway pavement surfaces in the vicinity of highway bridge abutments often leads to abrupt grade differences at the abutments. These grade differences subject travelling vehicles to a "bump" which leads to driver discomfort and potentially unsafe driving conditions, causes vehicle wear and damages sensitive cargo, subjects the bridge structure to repeated impact loads, and requires costly and repeated maintenance work that usually impedes the flow of traffic. To eliminate the bump at the end of the bridge, WSDOT and other agencies often install an approach slab with one end supported on the bridge and the other on the soil at some distance from the end of the bridge. Approach slabs ar often, but not always, effective in improving vehicle ride characteristics at bridge approaches subject to settlement.

A review of previous research indicated numerous potential causes of bridge approach distress, indicating that bridge approach settlement is largely a site-specific problem. A field investigation of nine distressed bridge approaches confirmed this observation. Recommendations for bridge approach design are presented.

The objectives of the research described in this report were to evaluate the effectiveness of bridge approach slabs, to identify site conditions for which approach slabs should and should not be used, and to present recommendations for the use of approach slabs.

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SUMMARY

Settlement of roadway pavement surfaces near highway bridge abutments often leads to abrupt grade differences at the abutments. These grade differences subject vehicles to a "bump," which leads to driver discomfort and potentially unsafe driving conditions, causes vehicle wear, damages sensitive cargo, subjects the bridge structure to repeated impact loads, and requires costly and repeated maintenance work that usually impedes the flow of traffic. To eliminate the bump at the end of the bridge, WSDOT and other agencies often install an approach slab with one end supported on the bridge and the other on the soil at some distance from the end of the bridge. Approach slabs are often, but not always, effective in improving vehicle ride at bridge approaches subject to settlement.

However, approach slabs are expensive, and in some cases they have been eliminated to reduce design and construction costs, particularly when bridge approach settlement has not been expected to occur. In most of these cases, performance has been as good as would have been expected with approach slabs, confirming the design decision to eliminate the slabs. In some cases, though, approach settlement has unexpectedly occurred, creating bumps that have required periodic maintenance.

The objectives of the research described in this report were to evaluate the effectiveness of bridge approach slabs, to identify site conditions for which approach slabs should and should not be used, and to develop an improved design process for the use of approach slabs.

A review of previous research on bridge approach settlement showed that the problem has been recognized and investigated to some extent by a large number of highway departments across the United States. Previous researchers have been unable to identify a predominant cause or mode of bridge approach distress, indicating that bridge approach settlement is largely a site-specific problem. Although the research does provide guidance

on soil conditions for which approach slabs are generally advisable, it does not define conditions for which they can be eliminated.

This report reviews previous research on the topic of bridge approach settlement. It summarizes measures that can be used to reduce the occurrence of undesirable approach settlement and unsatisfactory vehicle ride characteristics. It also presents and discusses the results of a survey of WSDOT personnel familiar with various aspects of the bridge approach settlement problem. Finally, the report presents the results of a field investigation of nine bridges for which unexpected approach settlement has resulted in the need for regular maintenance.

This research showed that there are certain soil conditions for which the use of approach slabs is advisable, but also conditions under which they may be eliminated. Elimination of approach slabs will require modification of a common abutment detail found in the majority of the WSDOT bridges in which unexpected approach problems have been observed. Recommendations for a modified abutment detail are presented. Use of the modified abutment detail should allow the satisfactory performance of bridge approaches constructed without approach slabs under many of the abutment soil conditions WSDOT bridge designers encounter. The elimination of these approach slabs may result in substantial cost savings in the design and construction of these bridges.

INTRODUCTION AND RESEARCH APPROACH

Bridge approach settlement often leads to abrupt differences in grade at the abutments of bridges. The resulting bump at the end of the bridge leads to uncomfortable and potentially unsafe driving conditions, increased vehicle wear, damage of sensitive transported goods such as fruits and vegetables, repeated impact loads on the bridge structure itself, and costly and repeated maintenance work that impedes traffic flow.

To eliminate or reduce the bump felt by traffic, approach slabs are often used at the ends of bridges. Approach slabs are structural slabs supported at one end on the bridge abutment and at the other end on the soil at some distance from the bridge. Approach slabs are often very effective in providing a relatively smooth transition in grade between the bridge and the approach pavement, but they are also quite costly. In some cases, the use of bridge approach slabs has not improved the performance of the bridge approach; occasionally it has merely moved the bump from the end of the bridge to the end of the approach slab. As a result, the use of approach slabs has become a subject of some controversy within highway departments across the United States. Currently, there is no uniform, rational procedure for evaluating the potential effectiveness of approach slabs, and no clear policy for the use or non-use of approach slabs at a given bridge site.

The purposes of the research investigation described in this report were to evaluate the effectiveness of bridge approach slabs, identify site conditions for which approach slabs should and should not be used, and develop an improved design process for the use of approach slabs. The approach taken in this investigation was designed to accomplish these objectives in the most efficient way possible.

First, previous research on the problem of bridge approach settlement was comprehensively reviewed to evaluate current knowledge on the subject. The literature review identified a significant number of investigations into the causes of bridge approach settlement and reports on the effectiveness of various measures of bridge approach settlement mitigation. The previous research was studied and summarized.

Previous investigations have produced wide differences in opinions regarding bridge approach design among highway department personnel across the country. Discussions with WSDOT personnel indicated significant disagreement within that organization regarding the causes of bridge approach problems and the proper design measures necessary to mitigate such problems. To gain a broad perspective on the extent of bridge approach problems and their causes in Washington, a survey was designed and distributed to WSDOT design, construction, and maintenance personnel throughout the state. Survey respondents were asked, among other things, to identify specific examples of bridges with good performance and with poor performance for more detailed study by the researchers.

To identify subsurface conditions for which bridge approach slabs could be eliminated, a field investigation program was undertaken. On the basis of the results of the literature review, the bridge approach survey, and discussions with WSDOT personnel, a number of bridges were identified for possible field investigation. These were bridges for which approach slabs were not used and for which unexpected approach settlement had developed. Problems associated with such bridges appear to be at the center of the controversy regarding the use or non-use of approach slabs; their performance has even led some to suggest that approach slabs be required on all bridges. The field investigation was intended to identify the causes of the poor performance of these bridges in the hopes that, if measures for eliminating the causes could be found, approach slabs could be eliminated for future bridges in similar conditions.

FINDINGS

The findings of the research investigation are divided into three main categories, each corresponding to a subtask of the project. In this chapter of the report, the results of the literature review are described first, followed by the results of the survey distributed to WSDOT design, construction, and maintenance personnel, and the results of the detailed field investigation of problems associated with a number of WSDOT bridges identified by survey respondents.

LITERATURE REVIEW

To evaluate current knowledge of bridge approach problems, a comprehensive literature review was undertaken. The review, described in the following sections, was based on both manual and computerized searches of the geotechnical and transportation literature.

Southern California (1959) Investigation

Jones (1959) described an investigation of four highway systems in the Los Angeles, California, area. The objective of the investigation was to identify possible relationships between bridge approach settlement and soil conditions. Available information regarding construction and approach characteristics were correlated to establish conclusions and recommendations.

The frequency of pavement patching was used as evidence of differential settlement for the purposes of this investigation. Soil conditions along the Hollywood Freeway were described as clay-filled depressions, and 50 percent of the bridge approaches were patched along this section of highway. Predominantly silts and soft clays made up the foundation soils along the Harbor Freeway, for which patching was observed at approximately 70 percent of the bridge approaches. Another section of highway, the Santa Ana Freeway, required 20 percent of its approaches to be patched to ensure acceptable driving conditions. A wide range of granular to cohesive materials was encountered in subsurface investigations of the foundation soils along the portion of the Santa Ana Freeway studied in the investigation. The final section of highway, the San Bernardino Freeway, was situated in an area underlain by coarse granular material. No approach patching had been needed in this section.

The key causes of approach settlement identified by Jones were compression of the approach embankment itself and consolidation of compressible foundation soils underlying the approach embankment. Additional settlement problems were construction related. Lack of sufficient compaction and excessively rapid construction were also identified as possible construction-related causes of approach settlement.

Jones observed that differences in patching frequencies were relatively small between structures on piles (40 percent) and structures on spread footings (38 percent). Differential settlements usually did not exceed 6 inches. More approach patching was observed for closed abutment bridges, primarily because their approach fills were placed after the construction of the bridges and abutments.

On the basis of the observations of bridge approach performance in California, several steps to reduce approach settlement were suggested. If economically feasible, removal and replacement of incompetent soils was one option. Early construction in conjunction with a preload fill to enable preliminary consolidation to occur was another option. The use of high quality backfill was recommended. Open end abutment construction was recognized as a way to improve access for compaction equipment near the abutment, and also to allow the approach embankment to be constructed before bridge construction. A 30-foot approach slab was suggested to "bridge" inaccessible materials adjacent to the abutment. In cases where bituminous paving was used, Jones recommended that it extend across the bridge deck. Lengthening of the bridge to reduce the embankment height was suggested. In other cases, the use of simply supported end spans on abutments without piles was suggested when approach settlement was determined to have stopped.

Ontario (1968) Investigation

In a study conducted in Canada (Stermac et al, 1968), seven bridge sites were investigated to understand the conditions that had caused movement of bridge abutments founded on end-bearing piles. Three of these structures had required significant rehabilitation as a result of lateral and vertical movement of the embankment/foundation system.

A review of the exploration, construction, and field monitoring records at each bridge site was conducted. The records included subsurface cross-sections, shear strength of subsurface clay, pile data, embankment height, and observed settlements at each site (both 900 and 1,900 days after construction). Settlement was measured with settlement plates placed under the embankment before construction. Field surveys of bridge conditions were also performed.

The designers originally realized that settlement of the approach embankment would occur because of the underlying sensitive, compressible Leda clays. However, abutment movements were not anticipated, since the abutments were founded on end-bearing piles. In conventional abutment design, the resultant force on a bridge abutment is inclined in a direction towards the structure. In each of the structures investigated, however, the abutments moved in a direction opposite to that assumed. Abutment movement away from the bridge had not occurred in other department of highway structures.

Of the seven bridge sites, two 22- to 25-foot high embankments (Brookdale, Hwy. 2) had settled 2.8 feet within 900 days after construction. Borelogs indicated that "very soft to very stiff clay" was present to depths of approximately 35 feet below the natural ground surface. The magnitude of settlement and lateral movement was large enough to require repositioning of rocker bearings at the abutments. After 1,900 days, settlements of 3.3 feet (Brookdale) and 3.8 feet (Hwy.2) were recorded.

Settlements at the other sites were of smaller magnitude than those observed at the Brookdale and Hwy. 2 bridge sites. The magnitudes and directions of the abutment movements at the various bridge sites were found to be dependent on the thicknesses of the underlying compressible layers.

The abutments that moved away from the structure were subjected to a resultant force acting in the direction opposite to that assumed in conventional design. The lateral forces arose from the consolidation of the subsoil under the weight of the embankment. Lateral movement of the embankment and foundation soils away from the structure created horizontal thrust on the abutment and supporting piles. This thrust allowed the abutment and piles to be displaced laterally, since the steel H-piles used to support these abutments provided very little resistance to horizontal forces. The investigators also noted that maintenance procedures, such as fills and asphalt overlays, only aggravated settlement and tilting.

Mitigating measures were proposed for other localities with comparable subsoil and loading conditions. Battered piles driven in a direction away from the bridge were suggested to resist lateral forces acting on the abutments. These battered piles (though piles battered in both directions longitudinally would have been preferable) were recommended for support of parallel wingwalls as well. Staged construction with longer waiting periods before bridge construction could have eliminated a great deal of post construction settlement.

Kentucky (1969) Investigations

Hopkins and Deen (1969) summarized approach settlement and various geotechnical conditions at a number of bridge sites in Kentucky. A survey of existing bridge approaches was conducted in 1964 and 1968 in which approach settlement was classified into three groups: Group 1 settlement-no maintenance necessary and no approach fault noticeable; Group 2 settlement-no maintenance performed, but an approach fault was observed; Group 3 settlement-maintenance performed on the approach. Additional information was obtained by visually inspecting and recording the age of each approach.

In the first part of the survey, conducted in 1964, the investigators indicated that approach embankment settlement appeared to be related to abutment type, geological conditions, and soil conditions. By the time of the second part of the survey, which was conducted in 1968, the relationships developed in the first stage could not be used as factors for predicting bridge approach settlement.

The investigators compared frequencies of repaired approaches for asphalt concrete pavement and portland cement concrete pavement and observed that differences were almost negligible. Apparently, at least for a short period of time, the rigidity of portland cement concrete pavement reduced the occurrence of the approach fault by bridging the presumed depression behind the abutment. The patched areas generally appeared within 100 feet of the ends of the bridges.

The investigators originally anticipated that approach settlement would be related to abutment type; however, many defective bridge approaches were observed with all types of abutments. Embankments located in valleys of major streams exhibited a greater percentage of Group 3 settlement. At a number of bridge sites, special fill was placed around the abutments and to distances approximately 20 to 60 feet behind the abutments. The study showed that embankments constructed with the special fill exhibited greater approach faulting than those with standard fills. The investigators did not elaborate on the inferior performance of the special backfill. From the results of the two surveys, the researchers concluded that the design and construction procedures used at that time were not sufficient to guarantee smooth bridge approaches.

In another 1969 study sponsored by the Kentucky Department of Transportation (Hopkins and Scott, 1969), field and laboratory settlement investigations were conducted at five bridge sites. General geologic and soil conditions were investigated, as were construction methods for some of the sites.

Settlement was measured with mercury-filled settlement gages at four sites and with settlement plates at the remaining site. Undisturbed soil samples were obtained and laboratory consolidation tests were performed to evaluate the consolidation characteristics of the soils at the various sites.

The investigators discussed potential methods to alleviate settlement problems at bridge approaches. Among them, an early construction start followed by waiting periods before bridge construction was considered effective whenever possible. The use of vertical sand drains was encouraged for acceleration of settlements in soft clay deposits. The use of high quality embankment fill material was considered to be very important for approach embankment construction. Reduction of embankment size by extension of the length of the bridge structure was recommended for certain cases. Reinforced, concrete approach slabs covered with temporary pavement was also suggested.

The researchers felt that their procedures for predicting embankment foundation settlement were adequate, since predicted and observed ultimate settlements usually agreed closely. Predicted settlements were generally overestimated by about 10 percent by the analysis procedures employed in the study (Terzaghi's theory of consolidation). Settlement of the approach foundation was observed to contribute significantly to settlement of bridge approach embankments and, consequently, approach pavements. Both field and laboratory data showed that time-settlement characteristics of foundation soils varied greatly; the estimated time for primary consolidation ranged from a few days to approximately three years. Compression of the embankment settlement itself can also lead to settlement of approach pavement. Therefore, careful investigation of embankment and foundation settlement was considered necessary to reduce the approach pavement settlement.

Ohio Department of Transportation Investigation

Timmerman (1976) described the results of an investigation of bridge approach design and construction practices that was sponsored by the Ohio Department of Transportation (ODOT). The investigation produced a comprehensive study of bridge approach design and construction practices in Ohio. The first phase of the investigation comprised interviews with bridge engineers and a literature search. The second phase, which investigated conditions at existing bridges throughout the state of Ohio, was an assemblage of three separate surveys conducted in 1961, 1974, and 1975.

The literature search established the state of knowledge based on research by other investigators. Factors found to be significant in influencing bridge approach settlement included pavement thermal expansion (growth), creep-induced lateral soil movements, abutment type, and fill characteristics. Differential settlement of the bridge deck and approach slab were found to result from vertical, lateral, and longitudinal movements of the embankment and foundation soils. The construction sequence and embankment construction technique were observed to either reduce or increase post-construction settlement. Interviews with bridge engineers provided information on various other factors relating to bridge approach distress.

The second phase of the investigation began with a review of data from a 1961 ODOT survey of 135 bridge approaches. This survey included general descriptions of foundation and embankment soils, embankment height, abutment type, waiting periods, and any unusual observations of distress. In the second part of phase two, Stark County personnel collected information on 38 bridges within Stark County, Ohio. The bridge approach distress parameters measured during that survey were the settlements of the approach slabs relative to the abutments and rotational movements of the abutments. The third and final survey of phase two was conducted by ODOT personnel in 1975 and yielded data from 147 bridge approaches built between 1964 and 1974 with approach embankments greater than 15 feet and piled abutments. Approach embankment settlements were determined in relation to "as built" centerline profiles. Information obtained by the survey included age and type of approach embankment, abutment pile type, drainage systems, and soil classification and profiles. The data obtained in the survey were evaluated by correlation-regression analysis to determine which characteristics, if any, could be associated with satisfactory or unsatisfactory approach embankment performance.

The statistical analyses the investigators performed indicated that no meaningful correlation could be made between observed bridge approach performance and the various design-construction parameters. Although the data were nearly random, a few general observations were offered by the researchers. Abutment type was noted to have some effect on embankment performance. For the pile supported abutments, 46 percent of the approaches had differential settlements greater than 0.1 ft., compared with only 22 percent of the pedestal abutments. The researchers concluded that the more confined the approach fill by abutment type or wingwalls, the greater the approach slab settlement. In addition, differential settlements between the bridge abutments and the approach slabs were greater for pile supported abutments than they were for stub abutments with spread footings supported in the embankment. The largest abutment settlements occurred with cast-in-place, reinforced concrete piles supported by soil friction or a combination of friction and end-bearing on soil. Smaller settlements were observed at sites characterized by slightly plastic embankment fills and foundation soils than at sites where those soils were highly plastic. In most cases, initial settlements were completed before final paving. On the basis of the results of their studies, the investigators concluded that a maintenance program was necessary to guarantee adequate bridge approach performance, regardless of the design and construction techniques employed.

Federal Highway Administration (1982) Investigation

An investigation conducted by the Federal Highway Administration (FHWA) (DiMillio, 1982) focused on the performance of highway bridge abutments supported by spread footings on compacted fill in Washington state. The investigation involved inspections and foundation movement studies. Brief case studies of seven bridges founded on spread footings were discussed. The cost effectiveness of spread footings versus piled foundations was illustrated with several examples.

The objective of the investigation was to evaluate the performance of highway bridges supported on spread footings in compacted fill to determine whether they could be used instead of pile-supported abutments in certain soil conditions. The objectives were accomplished in a four-task procedure. The four tasks involved a file search, field inspections/interviews, observed movement survey, and, finally, a data analysis/cost comparison.

Bridges were selected for evaluation from the Washington State Department of Transportation (WSDOT) Materials Lab and Bridge Division files. The selection criteria, which required that at least one abutment was on a spread footing, generated a list of 148 bridges for the survey. Interviews with maintenance, construction, and design personnel, as well as visual inspections, were conducted. Information from the interviews and inspections, allowed the study of potential damage from the use of spread footings as a foundation system. Approach embankment settlement was measured by comparing present profiles with "as built" elevations.

Differential settlement of at least .5 inch was experienced by at least 80 percent of the abutments. More than 50 percent of the abutments experienced more than 1 inch of differential settlement, and approximately 20 percent had more than 2 inches of settlement. None of the bridges exhibited signs of distress under these settlements.

In one case study, a structure over the Evergreen Parkway (SR-101) near Olympia, Washington, was situated on a 23-foot high embankment. Peat was removed under several pier locations and replaced with a silty gravel. Basalt bedrock was encountered 50 feet below the ground surface. The abutment founded on spread footings in the approach embankment settled approximately 2 inches differentially from adjacent piers that were supported on piles. The Anderson Road bridge across Interstate 5 north of Seattle, Washington, had abutments placed on 24 feet of granular fill overlying a soil profile described as "erratic." The abutments settled less than 1 inch differentially from the center pier.

The Pacific Avenue (AR-19) structure, at a major interchange in Tacoma, Washington, had soil conditions of dense sand and gravel underlain by 4 feet of loose sandy gravels. Three abutments were placed on high fills, while the two remaining abutments were on natural ground. Differential settlement between adjacent piers was less than 1 inch. The Nalley Valley Bridge, a curved structure in Tacoma, Washington, consisted of five piers and two abutments, all of which were founded on spread footings. Very compact, gravelly silty sand resided under two pier locations, and medium compact sand deposits were under the remaining piers. The largest observed differential settlement was 1.25 inches.

A 165-foot-long, single span bridge carrying SR-4 over Mill Creek at the Columbia River was constructed with one spread footing abutment on rock and the other on a 28 foot-high embankment. The foundation soils were loose to dense layers of organic silty sands, with occasional zones of fibrous peat, clay, and silt, and the fill consisted of broken basalt. The abutment located on the embankment fill continued to settle after construction and eventually settled 15 inches. The bridge was in surprisingly good condition, with no bumps at either end.

A piled foundation was ruled out for the Columbia River Bridge at Olds, Washington, because the required pile lengths would have been in excess of 200 ft. The seven span bridge was constructed with simply-supported end spans and continuous intermediate spans. The abutments were supported on spread footings in the approach embankment fills. One abutment experienced 0.34 feet of settlement and was jacked twice. The other abutment did not need jacking.

The report of the investigation offered some suggested procedures for site preparation to minimize approach embankment settlement at sites underlain by inadequate soils. Removal and replacement or in situ stabilization of the undesirable soils were recommended. Waiting periods and surcharges usually were suggested for use when possible. To avoid large embankment settlements, good quality fill was recommended to be compacted to 95 percent maximum density. If settlement was predicted to continue after construction, abutments modified with jacking systems were recommended. Design changes such as simple spans or larger girders were also recommended to accommodate differential settlements.

The FHWA considered the performance of 148 bridges with spread footing-supported abutments in Washington state to be very good. Spread footings appeared to provide a satisfactory alternative to pile foundation systems, provided that acceptable conditions of embankment and foundation soils were met. No functional distress or safety problems existed in these bridges, and their conditions during the study ranged from good to very good, with differential settlement of 1-3 inches tolerated without serious distress. Cost analyses showed spread footings were 50-65 percent cheaper than piled foundations.

Kentucky (1985) Investigations

Hopkins (1985) presented the results of long-term monitoring of the performance of approach embankments and pavements at six sites in the state of Kentucky. This work represented a continuation of efforts from previous studies (1968,1969,1970,1973,1982). The purpose of the investigation was to evaluate the magnitude and nature of the long-term movements of approach embankments and pavements. Approach embankments at the bridge structures were monitored for nearly 20 years with a number of techniques, including optical surveys, visual inspections, mercury-filled settlement gages, slope inclinometers, photo-documentation, settlement platforms, and piezometers. The study period lasted from 1966 to 1985, though not all sites were fully observed.

Shelby-tube and split spoon sampling were performed, along with unconfined compression, consolidated undrained, and one-dimensional consolidation tests. These test results were used to calculate settlement and stability, which were fundamental aspects of the research initiative.

From the results of previous studies in Kentucky, six potential factors leading to differential settlement were identified. Primary consolidation of the embankment foundation was frequently observed to have occurred before placement of pavement. Secondary compression and shear strain of the foundation and embankment had appeared to be related to soil type. Improper compaction of the approach embankment was observed to have led to both short- and long-term embankment settlement. Material erosion around abutment and approach pavement was also observed to have created loss of support, leading to differential settlement. Lateral and vertical creep deformations of the bridge approach embankment and foundation soils were also observed to result in differential settlement.

A 20-foot high embankment at a bridge along the Lexington Relief Route was constructed of silty clays placed in 1-foot lifts. Foundation soils, also of silty clay composition, were roughly 12 feet thick and were deposited over limestone. Settlements on the order of 0.3 to 0.5 in. in the first year and a half after construction were recorded within 175 feet of entry and 75 feet of exit. The author designated this zone as a "settlement cradle." Overprediction of settlement by 20 percent was attributed to the overconsolidated nature of the foundation soils. A long-term factor of safety against stability failure of 2.53 was calculated from effective stress parameters. Secondary consolidation was not an important factor at this site.

The original design for the bridge approach embankments at the Eddy Creek site indicated that total settlements of approximately 4 feet should be anticipated. The 35-foot embankment forged a portion of Lake Barkley, and the bridge crossed the main channel, Eddy Creek. Soft, saturated, slightly organic silts and clays, including interbedded sand lenses, composed the top 40 feet of the soil profile. Underlying these soft soils was a gravel hardpan, which exhibited standard penetration values in excess of 100 blows per foot. The approach embankment was constructed of a sand and gravel fill encased in rip-rap. In three to five months from the start of construction, initial settlements of the southern approach embankment were close to 12 to 18 inches, while the settlement of the northern approach embankment was only about 7 inches. Secondary compression had been anticipated to range from 2.1 to 6.2 inches in the next 27 years after primary consolidation. The approach embankment pavements were constructed 8 years after initial construction, when secondary compression was determined to be 0.5 inch. Negligible settlement was observed 9 years after placement of the pavement.

Sedimentary bluffs on either side of Bull Fork Creek required 65- to 75-foot approach embankments for the passage of I-64. Borings indicated that basement shales were topped by 18 feet of fluvial deposits, primarily shale and sandstone gravels hosting pockets of fines ranging in size from silts to clays. This floodplain foundation supported a shale-sandstone embankment constructed with a clay core. Although most of the samples obtained during the subsurface investigation at the site were considerably disturbed, a long-term factor of safety was predicted to be 1.2, and the ultimate settlement was estimated at 1 foot. Settlements measured 3 years after approach embankment construction averaged 4.5 inches, with the maximum settlement observed at 50 feet from end of bridge. The settlement cradles at the site extended back some 300 feet from the abutment to a cut section. The investigators indicated that the majority of the observed settlement resulted from compression of the non-durable rock (shale) placed in 2- to 3-foot lifts during embankment construction. Fluctuations from predicted values occurred, but overall agreement was satisfactory. Secondary compression was essentially nonexistent. Extrapolations of observed settlement indicated that a maximum of 10.4 inches of settlement could be expected by 1994. Large lateral movements, which occurred in the top 40 feet of the embankment, ranged from 2.5 to 7 inches, while lower embankment movements were typically 1.5 to 2 inches. As of 1984, considerable settlement was still occurring, and one of the abutments was undergoing rotation.

Also investigated was a bridge in the Kentucky River Basin. The approach embankment on one side of the structure was 35- to 55-feet high and was founded on 8 to 10 feet of soft, saturated, sandy clay. On the other side, a 55- to 75-foot high approach embankment was constructed on layered deposits of clay, sandy clay, and loose and very fine interbedded sand lenses deposited to considerable depth. These deposits had plasticity indices that ranged from 0 to 14 for all layers. Both approach embankments were constructed of a silty clay and had an effective angle of internal friction equal to 24.8 and effective cohesion equal to 250 psf. Calculations of short-term stability indicated factors of safety equal to 0.82 to 1.04, which suggested that staged embankment construction would be appropriate. Long-term factors of safety were 1.37 and 1.52 longitudinally at top and transversely along the side of embankment, respectively. Ultimate settlements of foundation soils in the taller embankment were estimated at 4 to 6 feet, assuming a soil profile consisting entirely of clay. Sand lenses, known to exist at the site, would act as drainage boundaries and would reduce ultimate settlements by 2 feet. The taller embankment was predicted to settle 7 inches by 1994, and exit embankments on the same side of the river were projected to settle 4 inches by 1994. Secondary compression of the foundation soil for the 35- to 55-foot-high approach embankment was expected to account for 3.1 inches, while 0.9 inch was expected because of secondary compression and shear strain of the embankment. Initial and primary settlements were sufficiently completed in the 2 years between the start of construction and paving. Secondary consolidation, on the other hand, was measured at 2.7 to 3.7 inches in the 16 years following completion (1968 - 1984). Secondary compression was predicted to be 4.3 inches, with 2.7 inches due to secondary compression and shear strain of the embankment. Actual lateral movements at this site for both embankments were less than one inch for a 14-year period.

The Slate Creek structure had 40- and 50-foot high embankments on its west and east ends, respectively. Both embankments were composed of a local, non-durable, greenish shale compacted in 2- to 3-foot lifts. The eastern foundation soils were classified as a low plasticity silt. Settlement was predicted to be 8 inches at the east end. Recordings taken 2.8 years after construction indicated that settlements of 3 to 4.7 inches had occurred at a point 30 feet from the edge of the bridge. The width of the settlement cradle reached from 75 to 200 ft. A 9-year observation period showed that only 1.2 inches of lateral movement occurred, and stability was moderate. On the basis of observed settlement behavior, the approach was projected to undergo final settlements of 8 to 14 inches by

1994. Unfortunately, the approach pavements were mudjacked and patched on numerous occasions.

The final site described by the investigators was situated in the eastern coal region of Kentucky. The southern abutment was founded on rock, but the northern abutment was founded on spread footings on a 52-foot high embankment overlying an alluvial foundation soil deposit. The foundation soil consisted of a 20-foot-thick silty-sand (SM) layer overlain by an 8-foot-thick layer of silty clay (ML-CL). The predicted settlement at the northern abutment area was 1.9 inches, with a long-term factor of safety against stability failure of 2.05 calculated for the embankment. Average actual settlements of 4 inches were observed in the first 8.8 years after construction. On this basis, an average ultimate settlement of 4.4 inches to 5.7 inches was projected. The observed settlement occurred almost instantaneously because of fine-grained sands in the foundation. Any further settlements of this approach were expected to be caused by secondary shear strain, or creep, of the embankment soils.

From the knowledge obtained by studying the six sites, the investigator developed a plan to mitigate approach embankment settlement problems. As a preliminary measure, a detailed subsurface and geotechnical report was recommended for every project. When approach embankments were determined to be located on compressible foundation soils, a number of mitigation techniques were suggested. The researchers recommended that a surcharge fill of length equal to approximately five times the sum of the height of the embankment and depth of the foundation be used to preconsolidate foundation soils. In addition, a detailed settlement analysis was recommended for evaluation of settlement due to secondary consolidation. The report noted that foundation soils with a coefficient of secondary compression as low as 0.007 were capable of producing significant settlements over long periods of time. The use of embankment materials exhibiting low secondary compression and creep behavior was recommended. Empirical methods for estimating rates of primary settlement of foundations and predicting settlements of bridge approach embankments were proposed.

Other mitigating measures included the use of wick or sand drains acting alone or in combination with a surcharge, removal and replacement of undesirable material, and the use of lightweight fills.

The investigators recommended that approach embankments be compacted to 98 percent of maximum dry density and within 2 percent of optimum moisture content. For durable rock, the maximum recommended compacted lift thickness was 2 feet. For durable shales with a slake-durability index (as determined from KM-64-513(79), 27) greater than 95 percent, the recommended compacted lift thickness was no greater than 1.5 feet. For nondurable shales with a slake durability of less than 95 percent but not less than 60 percent, the loose lift thickness was no greater than 12 inches. For nondurable shales with a slake durability index of less than 60 percent, the maximum recommended loose lift thickness was 8 inches.

Drainage considerations were also addressed. Select granular backfill enclosed in a geotextile filter fabric was recommended for use behind, under, and in front of the abutment. The use of perforated pipe installed in the select backfill and drained outside the limits of the embankment with adequate slope protection was recommended. In weathered shales, removal of weathered rock and construction of benches with drains in the unweathered shale was recommended.

Hopkins summarized observations made during the investigation. Initial and primary consolidation in all cases was completed before placement of the bridge approach pavements. The observed rate of primary consolidation was usually much faster than the predicted rate (by four times on the average), though total estimated settlements were in reasonable agreement with observed total settlements. Settlement cradles ranged from 70 to 300 feet, with a maximum approach settlement of generally 15 to 150 feet (averaging 50-60 feet) from the abutment.

Secondary compression of approach embankments and foundations was a major factor leading to settlement of the approach embankment. Lateral movements of approach embankments caused by shear strain may have led to settlement of approach pavements. Other lateral movements were detected for foundations that sloped toward the ends of bridges. Generally, if the embankment had a large factor of safety (F.S.=1.50), the settlement of the approach pavement was smaller than at those sites where the factors of safety were lower.

Poor compaction and a specified lift thickness of 2-3 feet was considered a chief flaw in the construction of some embankments. This was especially apparent when poor compaction was coupled with the use of nondurable rocks. The settlement of approach pavements was aggravated by erosion of material around the abutment. Use of reinforced concrete bridge approach slabs did not eliminate differential settlement between the bridge deck and the approach pavement, though the slates did improve vehicle ride characteristics.

California (1985) Investigation

In an investigation sponsored by Caltrans (Stewart, 1985), approach slab parameters and abutment types were compared in an attempt to rationalize the rough transition often observed at bridge ends in California. Experimental approach slabs were analyzed, and a new approach slab design concept was developed. The report presented statistical data and recommendations.

In a 1973 survey, every fifth structure along an 1,800 mile loop with 410 bridge structures and 820 approaches was inspected. The investigation concentrated on three areas. The first area concerned approach conditions. The observed approach conditions included major spalls, pop-outs, cracking, buckling, repair work (surface patching or mudjacking), and vertical displacements at the cold joint. The second area of emphasis was roughness rating which was measured by a vehicular mounted, strip-chart recorder developed by Caltrans. A roughness rating of greater than or equal to 12.0 suggested that repair was needed. Approach slab parameters identifying location (entrance or exit), pavement type (AC or PCC), age, skew, fill height, and original ground type were also obtained. The last area of interest was related to bridge conditions. Noted bridge conditions were cracks and/or deformation of the abutment backwall, deck, girders, or wingwalls and any considerable erosion.

Of the bridges inspected in 1973, 63 percent were resurveyed in 1984. The main objective of this repeated survey was to study patching requirements at these bridges during the period between 1973 and 1984.

As part of the investigation, 60 test approach slabs were constructed and observed. Thirty-four were of special design, while the remainder were built according to standard specifications. The data collection extended over 10 years (1972-82) and used monitoring techniques such as vertical control at grade points and settlement devices, and visual inspections.

Additional experimental slabs, built in 1956-57, were evaluated. Of these slabs, 56 were of standard size (10.5 feet long, 9 inches thick), and 24 were of standard configuration except that the length was extended to 24-37 feet. Fifty of the 56 had variations in reinforcement and were 1-foot thick. Generally the soil conditions were similar. The performance of the approach slabs was based on the post-construction maintenance activities required for each slab.

A 1983 Caltrans task force proposed a variety of possible factors contributing to the settlement of the approach pavements. They considered consolidation of foundation and embankment soils to be an extremely important factor in approach settlement. Embankments constructed with poor quality materials and a lack of compaction were considered candidates for settlement. Settlement that occurred in the zone adjacent to the abutment required an approach slab of sufficient thickness to "bridge" that section of embankment. Longitudinal movement was also known to occur along the axis of the bridge in diaphragm type abutments. This movement resulted in bumps at the end of the approach slab.

The report contained a significant quantity of statistical data that related approach/bridge parameters to maintenance needs. The data were correlated to see whether any one, or any combination of parameters, was the sole contributor to rough approaches. Evidence from measurements of approach pavement/slab movement distinguished settlements of .02 ft. to .22 ft. The settlement devices installed into the slab were designed to measure slab-embankment separation and made recordings of 0.005 ft. to 0.195 ft..

A new approach slab was designed for larger volumes of traffic (>5000 ADT). The modified approach included a preformed, permeable, filter fabric placed along the abutment backwall and connected to pipes that drained through the wingwalls. A 6-inch permeable base interfaced the filter fabric and approach slab. A rubber water stop was placed between the approach slab and pavement sleepers. Specifications also required select material placed at 95 percent compaction within 150 ft. of the bridge ends.

The researchers felt that a maintenance summary should be logged for each bridge to determine future maintenance frequencies and costs. New approach slab design should be monitored with "rail" scribes. The investigators also recommended researching the effectiveness of different lengths of slab on a common fill, a 6-inch paving notch, and a sleeper slab that rests on a 10-foot length of highway pavement.

Of all the parameters discussed earlier, the ones determined to have some effect on the need for maintenance were age and geographical region. Age was mentioned mainly because older construction tended to have less rigorous specifications. The mudflat (geographic) region would generally be more susceptible to settlement because of softer soils. The settlement period was not a good parameter since almost every site was different. No conclusion was reached for average daily traffic (ADT).

The experimental approach slabs showed that slab length did not have a significant effect on the overall settlement. However, the shorter slab (10-15 ft.) was more cost effective. Separation occurred between the embankment and the slab in 92 percent of the cases. No recommendations were considered warranted for a change in the design of truck lanes, yet an increase in maintenance efforts and paving notch failure for truck lanes was noted, especially in sections with bituminous paving. The bridge engineer's survey confirmed that bituminous paving required more maintenance than portland cement concrete, while it dismissed the fact that specific abutment type also affected approach settlement. The 1956 experimental slabs functioned unsatisfactorily.

Kentucky (1985) Investigation

The Kentucky Transportation Research Program conducted an investigation (Allen, 1985) in which a questionnaire concerning bridge approach performance was distributed to state highway departments throughout the country. The intent of the questionnaire was to determine the extent of approach embankment settlement problems in the various states and to identify potential mitigating measures attempted by states that had experienced approach embankment settlement problems.

The questionnaire was composed of the following seven questions regarding approach embankment characteristics:

1. Is settlement of bridge approaches a major problem in your state?
2. Do you use some form of reinforced approach slab? If so, are they successful?
3. Also, if reinforced slabs are used, how long are they?
4. Are integral end bents used in your state? If so, have they performed well?
5. Are special procedures used when backfilling around the end bent? What are these procedures?
6. Are abutments on spread footings used in your state? If so, are they successful?
7. Are there any other methods that your state uses to minimize this problem?

Standard drawings and specifications for approach slabs and related earthwork were also requested from each state.

The results of the survey indicated that most of the northeast and north central states experienced little or no approach embankment settlement problems. The survey indicated that nearly all states have successfully used reinforced approach slabs to reduce differential settlement. Oklahoma, Wisconsin, and Montana had experienced difficulties with approach slabs, while Maryland did not use them. Approach slab lengths reported by the survey respondents ranged from 10 feet to 120 feet, with an average length of 40 feet. As for abutment design, though only 18 of the responding states indicated that integral end bents were commonly used, those states generally reported successful performance. The use of spread footings was common, as only eight respondents indicated that they did not use spread footings. All responding states except Ohio indicated that abutments on spread footings were at least partially successful in reducing approach settlement.

The survey respondents provided considerable descriptions of methods to minimize approach embankment. A foundation study was recommended as a preliminary measure. Many states suggested removal and replacement of incompetent soils if economical, or the use of wickdrains, preloads and waiting periods incorporated with select fills placed at high levels of compaction to resolve approach pavement settlement. Approach slabs of various lengths, wide expansion joints, drainage provisions, lightweight fills, jackable abutments, and temporary overlays were also suggested as potential solutions.

Some states described unique methods for minimizing bridge approach settlement problems. One state reported the use of three-span, continuous steel girder end spans, cantilevered to avoid abutment supports. Another state described the use of a bituminous concrete berm placed against the backwall and over the approach slab to minimize the abrupt change between the gravel fill over the slab and the concrete backwall.

The survey indicated that bridge approach settlement is a widespread problem throughout the United States. Most states used some form of reinforced approach slabs and had been generally satisfied with their performance. Relatively few states routinely used integral end bent abutments, though those that did felt they performed well. With the exception of eight responding states, all other respondents had used spread footing foundations on abutments. The survey concluded that the performance of spread footings as a foundation system had produced mixed results.

Oklahoma (1986) Investigation

University of Oklahoma researchers (Laguros et al, 1986), on behalf of the Oklahoma Department of Transportation, conducted an investigation into the causes of approach pavement settlement. This report described the first phase of a multi-phase project and presented the results of a literature search and questionnaire.

The objective of the first phase of research was to establish the existing knowledge of approach slab settlement. The goals were accomplished with a detailed literature search, a questionnaire, and a synthesis of the pertinent findings.

The literature review was conducted by computer searches at the Highway Research Information Service and at the University of Oklahoma (DIALOG, ORBIT, AND BRS systems). The results of the literature search provided numerous references on approach slab and embankment design, construction, and related case studies. The reports that were investigated suggested a number of approach slab/embankment problems and solutions. Factors found to be important in influencing bridge approach settlement included settlement of the embankment and/or embankment foundation, erosion of soil near the abutment, creep, and secondary compression. Construction problems such as improper compaction of the embankment and an improper sequence of construction events were shown to be responsible for approach pavement settlements. Wicke (1982) described a situation in which expansion of concrete pavements with temperature exerted forces on the abutment, creating a gap in the area adjacent to the abutment. The literature review also concluded that traffic loading contributed to settlement of approach pavements.

The literature review found many suggested solutions for alleviating approach settlement. Waiting periods and sand drains with or without surcharges had helped to preconsolidate soft soil deposits. Removal and replacement of undesirable material or light weight fills, lime treated bases, and pulverized fuel ash had also been used in embankment construction to reduce settlement. Reinforcing fabric had been used to distribute embankment loading and reduce differential settlement and the potential for failure of soft soil during construction. Most of the state officials stated that reinforced concrete approach slabs, some with longitudinal camber, had performed satisfactorily. Other design techniques included an approach supported by piles, extension of the structure, and the use of temporary pavement.

A questionnaire was sent to state highway offices and Corps of Engineer headquarters throughout the country. The focus of the questionnaire was to determine the status of the settlement problem, possible causes, and satisfactory solutions.

The literature search and questionnaire produced mixed results. A potential solution for approach embankment settlement problems used by one department was often not considered by others. The lack of consistent failure criteria at different sites made interpretation of the questionnaire results even more difficult. Because of the inconsistent and inconclusive results, the researchers presented numerous facts and observations.

The literature search and questionnaire did indicate that approach settlement was a widespread problem. Historically, most research on settlement problems had focused on specific approach parameters (drainage, backfill, pavement type). Few if any investigators had encompassed all aspects of the approach settlement problem. The economics of the approach settlement problem had not been investigated.

The investigators concluded that approach pavement settlement cannot be generalized for all sites. Settlement of the embankment and/or foundation, and construction specifications and procedures were considered to be the most significant causes of approach pavement settlement. Although they might influence the short-term rate of settlement, pavement type and traffic direction did not appear to be important factors. Erosion of soil from the abutment and embankment slopes appeared to contribute to the settlement problem. The effectiveness of stabilization of embankment material had not been sufficiently studied by most highway departments. Use of select material for the embankment might alleviate the problem in some areas. A waiting period might not be effective for many sites where secondary consolidation was significant.

Settlement was observed to be related to abutment type, with stub type abutments providing smoother transitions. Approach slabs often did not prevent the problem but only shifted the location of the bump to the end of the approach slab. Pile supported approach embankments had been used in extreme cases to provide smooth transitions, but their cost was usually prohibitive.

Kentucky (1986) Investigation

Hopkins (1986) investigated the accuracy of different methods of slope stability analysis for embankments. The Kentucky Department of Transportation had observed a number of slope failures at sites for which the factor of safety against slope failure had been calculated as greater than 1. Also, previous investigations had indicated that creep deformations leading to bridge approach settlements could be related to the factor of safety against slope failure. Review of a number of documented slope and embankment failures indicated that standard methods of slope stability analysis often overestimated the factor of safety. These methods, as well as a new model and program proposed by the author, were compared to develop a proposed method of design.

The loading induced by the construction of an embankment on a clay foundation was considered for short-term and long-term conditions. The short-term period consisted of the construction phase, during which the clay foundation soils were loaded under undrained conditions. The end of construction was considered to mark the end of short-term loading. The design of the embankment was considered to be governed by this end-of-construction stage, since pore pressures were reported at their maximum, and therefore, the factor of safety was at its most critical level. Short-term stability was calculated from total stresses and the undrained shear strength. Generally, the undrained shear strength was obtained from unconsolidated-undrained triaxial tests, unconfined compression tests, field vane shear tests, or cone penetration tests. The report noted that when pore pressures were measured, short-term stability could be analyzed with an effective stress analysis. The long-term period assumed dissipation of pore pressures over time, and stability was analyzed from effective stress parameters.

The investigators attempted to define potential difficulties associated with existing slope stability analyses. Methods of slope stability analysis examined included those developed by Bishop, Janbu, Morgenstern and Price, and Spencer. The investigators used the various analytical methods to calculate factors of safety for sites at which failure involving failed embankments, footings, load tests, excavated slopes, and natural slopes had occurred. Seventy-five case histories were investigated with total stress analysis, while effective stress analysis was used for 15 additional case studies. Shear strengths and foundation soil properties (w, LL, PI, LI), along with the calculated factors of safety for each case, were compared and summarized. The results were summarized on plots of LI versus F.S., PI versus F.S. and F.S. versus frequency of embankment failure. Several linear regression analyses were performed on the data to establish empirical formulas for correcting the undrained shear strengths to be used in stability analyses. The investigators suggested that the empirical formulas for drained strength correction be used cautiously. No corrections of laboratory effective stress parameters were recommended.

The investigators presented recommendations for selecting a design factor of safety and appropriate methods of slope stability analysis. Index properties of soils were used as guidelines to establish whether total stress or effective stress analysis would provide a more appropriate indication of slope stability. The guidelines suggested that a design factor of safety as low as 1.3 (total stress) could be used without correction of undrained strength if LI was greater than 0.36 and PI was less than 40 percent. The investigators recommended that when PI was greater than 40 percent and LI was greater than 0.36, the shear strengths be corrected and a design factor of safety of 1.3 be used. For a LI that was less than 0.36, both total and effective stress analyses would yield factors of safety that were too large. The laboratory shear strengths should be corrected and an effective stress analysis used.

Wyoming Investigation

The Wyoming Highway Department (Edgar et al, 1987) had successfully incorporated geotextiles into embankment construction for several years. The steep side slopes often used for approach embankments can cause lateral deformations to occur along the sides and end of the embankment. The resulting high shear stresses can cause horizontal movement of the approach embankment toward the bridge, as evidenced by observed closure of expansion joints. The Wyoming Highway Department developed a design concept that created a free standing geotextile wall behind an abutment. Consequently, the bridge abutment was not subjected to lateral earth pressures.

The geotextile reinforced approach embankment was used to limit lateral deformations and consequently reduce approach settlement. The construction procedure used by the Wyoming Highway Department was fairly simple. Several layers of geotextiles were installed in the embankment to a distance of 23 to 36 feet from the back of the abutment wall. A 9- to 10-inch lift of soil was placed and compacted on each layer of fabric, and the geotextile was then draped back over the surface of the compacted soil. The lapped edge extended 1 meter from the wall into the embankment and was buried under the next lift. The fabric used in the Wyoming specification was a Mirafi 1000 HP.

Edgar et al (1987) undertook a laboratory testing program to evaluate the horizontal stress reduction provided by the geotextiles. This reduction was expected to vary with deformation of the exposed face of the geotextile wall. The testing was performed in a stiffened box with one side and the top open and one movable side wall. The box was 30 inches tall, 36 inches wide, and 48 inches long. The sample was prepared in the box by fastening a piece of geotextile to the floor of the box. The soil, compacted in 6 -to 8-inch lifts, was wrapped in the geotextile fabric and subsequently fastened to the backwall. Surcharge loading was provided by a 24.5 kN MTS hydraulic actuator through a top panel.

Each test involved applying surcharge pressure to the sample and recording the vertical and lateral deformation of the reinforced sample.

In test type I, K_0 conditions were established after the application of surcharge pressures, and the side wall was subsequently moved away from the sample under constant vertical pressure. Measurements of lateral load and deformation were recorded during side wall movement. Test type II differed from test type I, in that the wall was moved away before the surcharge pressure was applied. The test type III was the same as test type II except a styrofoam panel was placed between the movable wall and the geotextile to evaluate the effects of the styrofoam forming system used in standard Wyoming Highway Department design practice.

The testing program enabled the researchers to identify a number of observations concerning the effectiveness of the geotextile reinforced soil in resisting deformations under the applied loading. The investigators observed that lateral deformations of 3 to 6 inches were required for the geotextile to fully support the soil. Total lateral deformations, due to unloading the fabric face under conditions of zero vertical pressure, were highly dependent on the initial tension imposed on the geotextile during construction. Once released, however, large vertical pressures cause little subsequent deformation. Inserts of common plastic foams did not appear to provide a viable alternative for reducing lateral stress behind abutments.

Colorado Department of Highways Study

The Colorado Department of Highways (CDOH) (Ardani, 1987) undertook a research program to identify factors or groups of factors responsible for pavement settlement at bridge approaches and to suggest measures for mitigating problems caused by such settlements.

The researchers were provided with a list of over 100 CDOH bridges with "moderate to severe" approach settlement problems. Each of these bridges was visually inspected, and the ten bridges whose problems were judged the worst were selected for more detailed study. These bridges were located in various CDOH districts and were in different geotechnical conditions.

Drilling, sampling, and in situ testing were performed at the approaches to each of the ten bridges. Drilling was apparently accomplished with hollow stem auger equipment, sampling with thin-walled sampling tubes, and in situ testing by the Standard Penetration Test (SPT).

The site conditions, including visual descriptions of the nature and amounts of approach settlement, were described for each of the selected sites. Detailed descriptions of the field and laboratory studies at each site were also presented.

A number of causes of settlement were inferred from observations and field and laboratory studies at the bridge locations. Consolidation of approach embankment and foundation soils was the most serious problem associated with pavement settlement. Pavement settlement was also significant where compaction of backfill and embankment material was deficient. Poor drainage enabled erosion at the abutment face.

One 22-foot-high embankment exhibited settlement of approximately 3 inches, which caused a wing wall to break. The embankment materials, which were placed on a foundation of sandy material, were 93 percent fines, while the other 7 percent passed the #40 sieve. The plasticity index of the embankment material was 18, and the SPT resistance was as low as 4 at a depth of 10 feet. Consolidation of the foundation soils was considered unlikely, hence approach settlement of this structure was attributed to consolidation of the embankment soils.

Another set of structures exhibited evidence of 3/4 inch to 1-1/2 inches in settlement of embankments constructed of cohesionless (gravel, sand, and silt) soil overlying foundations of dense sandy gravel with cobbles. The amount of settlement correlated with zones of low SPT blow count (N = 4 - 6) in the backfill and embankments; consequently, the approach settlement was attributed to poor compaction.

One structure was constructed with poorly compacted clay fill that had undergone significant erosion near the abutments. The embankment materials were so poorly compacted in the upper 10 feet that one SPT blow caused 2 feet of penetration, and the penetrometer could be advanced by hand. Water apparently entered the subgrade soils through cracks at the edge of the abutment and caused internal erosion and the possible formation of voids beneath the pavement section. The researchers attributed the observed settlement at this site to erosion. Two other structures were constructed of AASHTO A-7 and A-6 clays with high plasticity indices. Obvious evidence of erosion was observed at these sites, as were low SPT resistances (5 to 6) at depths of 5 to 10 feet. A total settlement of about 1.5 inches was caused by approximately 0.6 inch of foundation settlement, while the remainder was caused by consolidation and/or erosion of the embankment soils.

Two other structures were constructed in an area underlain by expansive soils that were hydrocompacted with prewetting before construction. Settlements of 2 to 3 inches were observed at the approaches, while up to 8 inches of settlement were observed at the wing walls. Standard penetration resistances were highly irregular, with blow counts as low as about 5 in portions of the structural backfill and embankment fill. These zones had locally high water contents, indicating poor drainage in these areas. The large settlements observed at these sites were attributed primarily to settlement of the foundation soils, with poor compaction of the backfill soils and poor drainage listed as secondary causes.

The researchers recommended a number of measures to alleviate the problem of approach settlement. These included treatment of any compressible foundation before construction (delaying construction, preloading, sand drains, wick drains), use of well-graded structural backfill behind abutments, and provision of adequate drainage systems to prevent erosion of the soil at the abutments.

The researchers also recommend further study on the effectiveness of geofabrics in reducing bridge approach settlements. Such a study was about to be initiated when the report was published.

The researchers concluded that foundation settlement due to consolidation was a major cause of observed approach settlement. Immediate settlement was not considered to be a significant problem. Consolidation within the approach embankment fill was also considered to be a problem, particularly where compressible materials were used in the embankment fills. Secondary compression was also considered an important cause of settlement of embankment fills. Poor compaction in areas of restricted access was considered significant, as was erosion due to infiltration of water behind abutment backwalls.

Maryland Investigation

The University of Maryland (Wolde-Tinsae, et al, 1987), on behalf of the Maryland Department of Transportation, conducted a study comprising a survey to state highway departments throughout the U.S. and overseas, and a literature review. The objective of the study was to identify the factors that create rough driving conditions, excessive maintenance, and structural problems at bridge approaches. Recommendations were also presented to mitigate the factors associated with poor approach performance.

The literature review was compiled from a computer search of the TRIS (Transportation Research Information Service), NTIS (National Technical Information Service), COMPENDEX (database, produced by "Engineering Information, Inc."), and FEDRIP (Federal Research in Progress) databases. The major causes of bridge approach problems found in the literature review were discussed. Most of the approach problems were associated with differential settlement between the highway pavement and bridge deck, rotation and/or lateral movement of the abutment, and poor design of structural components. The review covered the pertinent details of design, construction, and special treatments that helped reduce the rough riding transitions often experienced at bridges.

Differential settlement at bridge approaches was most commonly attributed to consolidation of embankment foundation soils. The factors found to affect the rate of consolidation included the degree of preconsolidation, soil properties, layer thickness, embankment dimensions, surcharges, length of drainage paths, and rate of construction of embankments. Settlement was defined by three phases: initial settlement, primary consolidation, and secondary consolidation. Another cause of differential settlement of bridge approaches was volume change of the approach embankment. Volume change within the embankment resulted from rearrangement of soil particles, shrinkage, swelling, or ice and frost action.

Lateral movement of approach embankments was identified as a cause of differential settlement at bridge approaches, particularly when the embankments were located on sloping ground, and a high water table, weak foundation materials, and poor compaction were present. Subsurface erosion adjacent to abutments was also found to contribute to the development of approach faults.

Rough transitions across bridge approaches were related to longitudinal, rotational, or vertical bridge abutment movements. Abutment movements were caused by slope failure, seepage, thermal forces, and foundation settlement. Embankment instability caused large lateral earth pressures to be exerted on abutments and supports, which caused piles to bend, wells to tip and crack, and caused damage to abutments and expansion joints. Seepage was considered to be a cause of abutment movement because of an overall reduction in soil resistance and an increase in lateral earth pressure on abutment backwalls. Seepage was also found to be detrimental in soils susceptible to piping. Lateral abutment movements were also associated with thermal expansion of the superstructure and highway pavement. The investigators believed that expansion joints were often not functioning properly because of an influx of debris. Large lateral pressures were then exerted against the abutment and supports because of the inability of the expansion joint to accommodate thermal movements. Rotations were detected in abutments that had settled because of the consolidation of underlying layers of variable thickness and also because of negative skin friction on deep foundation elements.

A number of mitigating measures for reducing bridge approach settlement were identified in the literature search. Most were for treatment of soft soil conditions and included densification, preconsolidation surcharges in conjunction with waiting periods, and wick/sand drains. The use of removal and replacement or lightweight fills was also suggested to help reduce approach settlements.

The investigators felt that most highway departments constructed approach embankments that produced only minor settlements because of the compression of the embankment materials themselves. Hence, embankment settlement was considered to be small in comparison to consolidation settlement of compressible foundation soils. Parameters known to affect embankment compression were gradation, plasticity, water content, and degree of compaction. Strict construction specifications enforced by trained inspectors typically minimized such problems. The investigators noted that cohesionless soils may have a high potential for volume change caused by traffic induced vibrations. Expansive soils were known to be problematic for several state highway departments, and appropriate solutions were suggested by the investigators. Among them was placing an open-graded fill without compaction behind the abutment to minimize the effect of soil expansion. Prewetting and covering with geomembranes were also proposed for controlling the moisture content of the expansive soil. Free draining materials were suggested for use in the upper zone of the backfill in areas susceptible to freezing. Provisions for draining the entire embankment, approach slab, and abutment system was strongly encouraged.

The type of abutment foundation was shown to have an important influence on the development of irregular approach surfaces. Significant differential settlement was more common for pile supported abutments than for abutments founded on footings. The investigators recognized the difficulty of achieving proper compaction in confined areas and stated that the more confined the approach fill was by abutment type or wingwalls, the greater was the observed approach slab settlement. Stub or perched abutments generally provided the best performance, since the embankment was constructed before the abutment. The use of drained benches was also suggested.

Approach slabs were used by almost all state highway agencies, yet no standard size, shape, reinforcement, connection/end support or sleeper arrangement was identified. The investigators also recommended that the "joint treatment between the approach slab and the abutment should be able to transfer traffic loads from the approach slab and the abutment, to prevent surface water from entering, and permit expansion as necessary to prevent damage." Some special designs such as cellular abutments, cantilever end spans, and geotextile reinforced embankments were recognized as potential solutions to reduce bridge approach defects.

The investigators recommended a phase II study that would encompass research on details and methods of construction for surface and subsurface drainage systems. The second study would also analytically investigate the type of soil and degree of compaction to achieve the best possible results. Guidelines for the use of geotextiles in the construction of approach embankments were to be established. Finally, a parametric finite element study would be used to identify the most appropriate reinforced concrete approach slab design. The phase II study is apparently now in progress.

NCHRP (1989) Synthesis

The motivation for this report was to update the previous NCHRP synthesis on bridge embankment construction (NCHRP, 1969) by recognizing new materials and techniques that had emerged over the last 20 years. The updated synthesis covered embankment/foundation conditions, approach embankment design/construction, abutment design/construction, and approach slabs. A synopsis of bridge approach maintenance and rehabilitation procedures was presented, along with case studies describing the implementation of some new practices.

The synthesis emphasized the need for interaction among design, construction, maintenance, pavement, and geotechnical engineers to ensure economical and safe bridge approach systems. New innovations in geotechnical design and construction methods have provided engineers with greater versatility in reaching potential solutions to bridge approach settlement problems.

The results of the literature review again indicated that bridge approach embankment settlements result from a variety of factors. The most common causes, which can act in combination or individually, were consolidation of embankment and foundation soils, poor compaction of abutment backfill, erosion of soil at the abutment face, and poor drainage of the embankment fill. Although these factors were considered to be the most common causes of approach embankment settlement, other design errors were also included. Most commonly, marginally stable embankments with low design factors of safety against stability failure and inaccurate estimates of stress distribution beneath embankments were found in cases where large bridge approach embankments had settled.

An FHWA study of 21 shallow foundations on sand reported an average total settlement of 0.75 in., but less than 0.25 in. occurred after construction of the bridge deck. Measurable movements were detected in 75 percent of the abutments surveyed; the greatest frequency occurred in perched abutments constructed on either spread footing or piled foundations. Nevertheless, a significant portion of other abutments were reported to have moved. Eighty-one perched abutments constructed on spread footings with preloads or waiting periods produced an average settlement of 1.8 in. in comparison with a 7.5 in. average settlement for 60 cases without preloads or waiting periods.

The synthesis focused on new advances in design and construction. The following methods were presented as options, but not necessary practical solutions. Ground improvement methods such as dynamic compaction, vibrocompaction and consolidation surcharges, ground anchors, geosynthetics, and mechanically stabilized walls and abutments can be used to reduce movements. In some situations, removal and replacement of incompetent soils, embankment piles, or stone columns have been shown to increase the bearing capacity of an embankment. Lightweight fill was suggested for maintenance and rehabilitation work. Standard specifications and procedures that require end slope protection, provisions for drainage, and strict density specifications for select, previous backfill placed in 6-inch lifts have been associated with reduced bridge approach settlements. Other schemes incorporating field monitoring and jackable abutments with temporary asphalt overlays and sleeper slabs have also been effective.

Wyoming (1989) Investigation

In a joint effort, the University of Wyoming and the Wyoming Highway Department (WHD) conducted laboratory and field studies to determine effective methods of constructing geotextile reinforced embankments (Edgar, et al, 1989). Four embankments were reconstructed for a set of structures on Interstate 80 approximately 30 miles east of Cheyenne, Wyoming. The WHD considered geotextile reinforced embankments to be a potential way to alleviate expansion joint closures and to reduce settlement of approach slabs. Differential settlement was assumed to be caused by excessive embankment deformation. Damage to expansion devices was attributed to high lateral soil pressures acting on bridge abutments. Both of these problems were controlled by a "voided" embankment reinforced by geotextiles.

The construction technique required a small void space between the abutment wall and the geotextile wrapped backfill during construction. This void space allowed the fill to undergo a small deformation under loading that allowed minimum active earth pressures to develop.

The researchers analyzed several types of techniques for providing a void space between the geotextile reinforced embankment and the bridge abutment. The objective was to create a form against which an embankment could be constructed and which would later collapse to provide the required void space. The use of styrofoam was unsuccessful because no deformation occurred from embankment pressures. Blocks of ice were hypothesized for creation of void space but were deemed impractical because of the temperature and time required for construction. Plywood slipforms and collapsible cardboard cells were investigated in this study.

Another set of tests was performed to model the reinforced embankment with the apparatus developed for the previously described research. In tests in which the wall was allowed to move under various vertical surcharge pressures, a 90 percent reduction in the lateral pressure occurred with only 0.15 inches of wall movement for a surcharge pressure of 500 psf. For a surcharge pressure of 1,500 psf, a 90 percent reduction was observed with 0.4 to 0.45 inches of wall movement.

Four instrumented embankments were constructed with various design differences. One embankment was constructed without and the others with woven geotextiles reinforcement. One reinforced embankment was constructed with 2 inches of corrugated cardboard to form the void space between the abutment and the embankment. Another geotextile reinforced embankment utilized plywood slipforms to create a 6-inch void space, and the third reinforced embankment had no provisions for void space.

The investigators summarized the significant findings based on measurements made during the field observation, which started in May 1987 and extended through July 1988. The most notable findings related to the embankment pressure distributions that were applied to abutment backwalls. The embankment constructed with the collapsible cardboard cell showed lower lateral earth pressures (near zero) than the embankment constructed directly against the abutment. Lateral earth pressures decreased from their initial values in August 1987 to February 1988 and returned to the initial values, or in some cases exceeded the initial values, by July 1988. Voided embankments also showed higher lateral pressure cell readings in the top cells (because of the notch at the corbel) than any of the cells located at further depths in the embankment.

Other important conclusions described embankment movements. Smaller lateral movements occurred under the roadway surface than in the side slopes. Unreinforced embankments settled more than the reinforced embankments. Apparently, heavily reinforced concrete approach slabs were necessary to span the voids caused by the differential settlement in the embankments.

The investigators recommended the use of cardboard to create a void between the reinforced embankment and abutment. It was imperative that the cardboard be kept dry before construction. Plywood slipforms proved difficult to remove and needed redesigning. The investigators also suggested wrapping the sides of the embankment and laying the fabric parallel to the centerline of the roadway. Since the geotextile reinforced embankment is relatively new, extended research was recommended for evaluation of long-term behavior.

Nebraska (1989) Investigation

In a report sponsored by the Nebraska Department of Roads (NDR), Tadros, et al. (1989) presented the results of a study that identified factors contributing to bridge approach settlement and evaluated solutions designed to reduce rough transitions across bridge ends. The data were compiled from a literature review, a survey to highway agencies in the 50 states and the Canadian provinces, and inspections of bridges in eastern Nebraska. On the basis of the pertinent findings of the study, recommendations were suggested for properly implementing design, construction, and maintenance procedures to provide for smooth bridge transitions.

A previous NDR study (Chenney, 1975) concluded that approach slab "faulting" was caused by differential settlement between the bridge deck and the adjacent pavement. Differential settlement was attributed to consolidation of foundation soils, consolidation of embankment materials, or displacement of embankment materials under the paving surface. Although embankments were constructed long before final paving, a 25-foot section was typically excavated to depths of 4 to 6 feet to facilitate abutment construction. This was the zone in which embankment settlement was considered to have occurred. Additional study, which measured settlement 3 to 4 years subsequent to final paving, inferred that consolidation of the foundation soils, rather than displacement of embankment material under the pavement, was the primary cause of approach settlement. Lower settlements were observed at the abutment than at greater distances from the bridge.

In another study (Dunn et al, 1983) reviewed as part of the literature search, Wisconsin state maintenance personnel noticed an increase in the amount of maintenance required on unreinforced concrete approach slabs. A survey of 200 approaches determined that flexible approaches were rated as poor 70 percent of the time, while 93 percent of the reinforced approaches were rated as good. The survey also found that most reinforced approach slabs provided good performance through at least 8 years, while non-reinforced slabs were in poor condition within 2 years after construction.

An Ohio study (Grover, 1978) disclosed that over 80 percent of abutments statewide underwent 2.5 inches of total settlement, and 10 percent had settled over 4 inches. Most of these abutments, whose settlement was considered to be intolerable, were founded on spread footings. The report indicated that differential settlement became more prevalent after bridge designs were switched to piled foundations.

Some general trends were established from a 14 question survey to the 50 states and the Canadian provinces. Fifty-two responses representing 36 states, the district of Columbia, and three Canadian provinces were received. The survey showed that the settlement of bridge approaches was a widespread problem. Most survey respondents identified high embankments and high volume roads as particularly susceptible to bridge approach settlement problems. Approximately one-third of the respondents had monitored embankment/slab settlement. Overall, 19 percent of abutments appeared to be placed on spread footings, and 81 percent on deep foundations. Some 88 percent of highway departments reported the use of reinforced concrete approach slabs to control approach settlement. Roughly one-third of those states also used a sleeper slab. Asphalt overlays appeared to be routinely used for maintenance of rough approaches, but more than half of the respondents reported the use of slab jacking. Repeated maintenance was required every 3 years or less for most of the respondents.

The investigators also inspected 53 Nebraska bridges. Approximately 79 percent had undergone some form of maintenance work to remediate the settlement problem. The investigators also attempted to establish a criterion for distinguishing tolerable bridge transitions. Factors observed to affect tolerable settlement included angular variance, speed of vehicle, direction of traffic, and human perception. A limited number of measurements were made, but a useful correlation of measured factors was not achieved. Further research was suggested.

On the basis of the information obtained from the literature review, survey, and bridge inspections, the report discussed a number of potential causes of bridge approach settlement and their remedies. The main cause of approach settlement was determined to be differential settlement between two dissimilar structural systems. Typically, the embankment was free to settle, while the abutment supported on deep foundations did not move. Foundation consolidation was the primary explanation presented for this differential settlement. Embankment stability was also an important factor that influenced approach slab settlement, especially for embankments constructed on sloping ground. The investigators recommended that a factor of safety of at least 2 against stability failure be maintained to minimize approach slab settlement. Erosion and high lateral forces on the abutment backwall both aggravated the approach settlement problem.

Potential remedies to reduce the number of structures that experienced intolerable approach settlement were presented. Detailed geotechnical investigations, preconsolidation surcharges, waiting periods, removal and replacement, wick drains, and benching were suggested. Other techniques such as the use of lightweight fills, stone columns, bridge lengthening, and cellular abutments were proposed. One technique, based on the assumption that settlement was inevitable, involved construction of the approach at a level higher than final grade.

The investigators discussed modifications to the typical approach slab design. One modification involved construction of the separate, but adjacent, approach slabs for each individual lane to allow traffic to be diverted to adjacent lanes during slab repair. The use of preformed grout holes in the approach slab was discussed by a handful of states, including Missouri. New methods of slab lifting were discussed, including the use of polyurethane foam used in building construction and physical jacking by hydraulic rams or mechanical jacks. The investigators suggested an elaborate pneumatic adjustable sleeper slab built below the connection of the two reinforced approach slabs. Inflatable tubes would adjust the settled grades of the slab. Removable precast pavement panels were also discussed as a means of mitigating serious approach slab settlement.

WSDOT SURVEY

To acquire additional information on bridge approach conditions in Washington, a survey was prepared and distributed to WSDOT design, construction, and maintenance personnel. The survey was designed to provide a broad perspective on the extent of bridge approach problems in Washington, and to determine the use and effectiveness of approach slabs. The survey also addressed causes of approach distress and measures used for mitigation of such distress. The survey also requested specific examples of bridges with and without approach slabs that had and had not experienced approach distress. A copy of the survey is included in Appendix I.

In recognition of the widespread disagreement over causes of bridge approach problems and policies on the use of approach slabs in other states, the researchers wanted to obtain the opinions of a variety of WSDOT personnel who would have different perspectives on the issue. Accordingly, the survey was sent to design, construction, and maintenance personnel at both the state-wide and district levels. The respondents were promised anonymity and asked to be frank in providing their honest opinions.

The survey was sent with stamped, self-addressed envelopes to 78 potential respondents. Actual responses were received from 23 of these people. Responses were received from each of WSDOT's districts according to the distribution shown below:

District	1	2	3	4	5	6
Responses	6	2	3	2	7	3

The following paragraphs describe and discuss the responses to the various questions posed in the survey. Since respondents were advised to skip any questions with which they were uncomfortable or uninformed, the total number of responses to individual questions varied. The survey was not designed or intended as a scientific, statistical sample of any of the districts or WSDOT at large. Its primary purpose was to gain the benefit of the experience of WSDOT personnel with different backgrounds and different perspectives on the problem of bridge approach distress. The brief interpretation of the survey results was made in that spirit.

1. Is differential settlement between approach pavements and bridges a common problem in your district? Please rate from 1 (very common) to 5 (very unusual).

District	1	2	3	4	5	6
Average Rating	1.50	3.75	1.33	2.50	2.14	1.67

Responses to this question indicated that approach settlement problems occur commonly in all WSDOT districts except District 2. Interestingly, however, District 2 personnel were among the most active respondents to a follow-up survey requesting examples of bridges for detailed field investigation. Inconsistencies such as this serve to emphasize the difficulties of interpreting a small sample survey.

2. In your district what percentage of bridges use approach slabs? (Circle best answer)

	Percentage of Total Number of District's Responses				
District	0-20%	20-40%	40-60%	60-80%	80-100%
1	17	33	50	17	0
2	50	50	0	0	0
3	100	0	0	0	0
4	0	0	100	0	0
5	14	29	29	29	0
6	50	33	0	0	0

A relatively low rate of approach slab utilization was implied by the responses to this question. Interestingly, the apparent rate is lowest in District 3, where many bridges cross soft fluvial deposits known to produce large foundation soil settlements. The most significant rate of approach slab utilization was reported by District 5.

3. In your district, what percentage of bridge approaches need periodic maintenance to reduce differential settlement problems?

Bridges without approach slabs

	Percentage of Total Number of District's Responses				
District	**0-20%**	**20-40%**	**40-60%**	**60-80%**	**80-100%**
1	0	14	14	43	14
2	50	50	0	0	0
3	0	0	0	50	50
4	0	0	100	0	0
5	0	43	14	29	14
6	0	0	0	100	0

Bridges with approach slabs

	Percentage of Total Number of District's Responses				
District	**0-20%**	**20-40%**	**40-60%**	**60-80%**	**80-100%**
1	57	29	0	14	0
2	50	0	50	0	0
3	100	0	0	0	0
4	50	0	50	0	0
5	86	14	0	0	0
6	50	50	0	0	0

The perceived effectiveness of approach slabs was strongly illustrated by responses to this question. Approach maintenance requirements for bridges without approach slabs is needed on the majority of bridges in all districts except District 2. On the other hand, for bridges with approach slabs, reported maintenance requirements were much lower. The survey respondents obviously felt strongly that approach slabs have been effective in reducing bridge maintenance requirements.

4. On the average how long does it take for significant differential settlement to develop after completion of construction?

District	Percentage of Total Number of District's Responses				
	0-1 yr	1-2 yrs	2-3 yrs	3-5 yrs	other ____
1	0	14	71	14	0
2	0	100	0	0	0
3	0	33	33	33	0
4	0	50	0	50	0
5	14	50	33	17	0
6	0	33	33	33	0

The results of this question indicated that significant differential settlement requires an average of 1 to 5 years to develop. Because these are average periods, individual bridges can develop significant differential settlement within 1 year of construction or more than 5 years after construction.

5. On the average, how often is maintenance necessary for bridges which develop problems?

Bridges without approach slabs

District	Percentage of Total Number of District's Responses				
	0-1 yr	1-2 yrs	2-3 yrs	3-5 yrs	other ____
1	0	71	29	0	0
2	0	100	0	0	0
3	0	50	50	0	0
4	0	100	0	0	0
5	14	57	14	14	0
6	0	33	67	50	0

Bridges with approach slabs

District	Percentage of Total Number of District Responses				
	0-1 yr	1-2 yrs	2-3 yrs	3-5 yrs	other _____
1	0	0	0	57	43 (5-10, 15-20, 10-20 yrs)
2	0	33	0	0	67 (8-10 yrs)
3	0	50	0	50	0
4	0	100	0	0	0
5	14	0	0	14	72 (none, never, 10-20 yrs)
6	0	33	67	50	50 (10-12 yrs)

When approach slabs are not used, approach maintenance appears to be required at intervals of 1 to 3 years for all WSDOT districts. With the exception of District 4, each district reported that frequency of approach maintenance was reduced when approach slabs were used. Several respondents indicated quite large intervals between required maintenance when approach slabs were used.

6. How is maintenance typically performed?

• **Pavement overlays**	Listed by 21 respondents - common in all districts
• **Slab jacking**	Listed by two respondents as rarely used
• **Other - please specify:**	Other listed techniques were: • Full depth base repair • Hand patching or rotomill with hand patching • Cold patching at the immediate end of bridge • Expansion joint work • Corrected with bridge deck rehabilitation contract • Backfill

The use of AC overlays is by far the most common method used to repair distressed bridge approaches in all WSDOT districts. While other methods were listed, their use appears to be very infrequent and to be limited to special cases in which pavement overlays cannot be used.

7. The type of abutment foundation can influence the amount of differential settlement at bridge approaches. In your district what percentage of bridges with the following types of bridge foundations develop differential settlement problems?*

Shallow foundations (footings)

	Percentage of Total Number of District's Responses				
District	0-20%	20-40%	40-60%	60-80%	80-100%
1	33	33	0	33	0
2	50	0	100	0	0
3	100	0	0	0	0
4	0	0	100	0	0
5	0	33	33	33	0
6	0	0	50	0	50

Deep foundations (piles drilled shafts)

	Percentage of Total Number of District's Responses				
District	0-20%	20-40%	40-60%	60-80%	80-100%
1	0	50	25	25	0
2	100	0	0	0	0
3	0	0	0	100	0
4	0	0	100	0	0
5	0	33	33	33	0
6	100	0	0	0	0

*Nine respondents indicated that they did not know the foundation conditions at specific bridges well enough to respond to this question.

Somewhat higher frequencies of approach problems appear to develop with deep foundations than shallow foundation, according to the results of this question. However, the relatively large number of respondents who indicated that they did not know foundation conditions at specific bridges rendered the results of this question difficult to interpret.

8. There are a number of possible causes of differential settlement at bridge approaches. Based on your experience with bridges in your district rank the following possible causes from 1 (very likely) to 5 (very unlikely). Since there can be more than one cause of differential settlement at a site more than one can be rated as very likely.

Cause 1: Settlement of foundation soils beneath approach embankment:

District	1	2	3	4	5	6
Avg.Rating	1.6	4.0	1.0	1.0	1.7	1.3

Cause 2: Overall compression within approach embankment fill:

District	1	2	3	4	5	6
Avg. Rating	1.7	3.0	3.0	3.0	2.2	3.3

Cause 3: Local compression (within 3-4 ft of bump) of embankment fill:

a. Due to poor compaction:

District	1	2	3	4	5	6
Avg. Rating	1.4	4.0	5.0	3.0	1.7	3.0

b. Due to poor embankment fill soils (excessive fines, etc.)

District	1	2	3	4	5	6
Avg. Rating	3.0	3.0	5.0	3.5	3.5	3.0

c. Due to water infiltration

District	1	2	3	4	5	6
Avg. Rating	3.0	3.0	5.0	3.5	2.3	1.7

Cause 4: Distortion of pavement section (rutting, etc.):

District	1	2	3	4	5	6
Avg. Rating	3.0	4.0	3.5	5.0	3.8	3.0

Cause 5: Erosion of embankment soils

District	1	2	3	4	5	6
Avg. Rating	3.3	2.5	5.0	5.0	2.2	3.3

Comments

"Soils available locally and foundations almost always have caused problems here."

"Bridge cost per sq. ft. was so great by comparison to the fill, it is cheaper to repair the approach."

"All of these occur at times."

"On some old bridges soils settle and between piling."

"Within 3 to 4 feet or put seat on structures with no approach slab."

"Most problems appear to be density problems with backfilling of piers."

"We have few classic differential settlement situations. Our biggest bridge end problem comes from the 'modern' bridges without expansion joints at the abutments. As the bridges expand due to thermal action, the adjacent embankment, surfacing, and pavement is disturbed. As the bridge contracts an opening often remains. Sometimes the water gets into the opening and further erodes the materials. This is not to say that abutments and expansion joints are the solution. We have lots of expansion joint failures, too."

"On certain structures, such as a box girder, I believe that the structure itself cause a part of the settlement. When the bridge expands, it moves the fill. When it contracts, the traffic compresses the fill."

"None of above-compression at bridge embankments by expanding structures and movement of soils into the void."

"There is also the thermal effect of the bridge expanding and contracting, which may compress the approach embankments."

"I believe the bumps are a "design" problem rather than construction. Without approach slabs or expansion joints on end piers, no provision is made for movement of the bridge due to temperature changes."

The perceived causes of approach settlement reflected, to a large extent, the general geological and geotechnical conditions present in the various districts. Settlement of foundation soils was considered dominant in Districts 3 and 4 in which many river bridges cross deep deposits of soft, fine-grained soils, and important in Districts 1, 5, and 6. Conversely, foundation soil settlement was considered to be an unlikely cause of approach settlement in District 2. Compression within approach embankment fills was considered to be a moderately likely cause of approach settlement in all districts, but somewhat more likely in Districts 1 and 5. Interestingly, local compression due to inadequate compaction in the vicinity of abutments, one of the most common causes suggested by previous researchers, was considered to be a likely cause of approach settlement only in Districts 1 and 5. Districts 3 and 4 considered it to be unlikely; however, their perception of its importance may have been masked by the large foundation soil settlements that commonly occur in those districts. Settlements resulting from the use of poor quality soils adjacent to the abutment were considered unlikely to moderately likely by all districts, as was local compression due to water infiltration, except in Districts 5 and 6. Pavement distortion, or rutting, was also considered an unlikely cause of approach settlement. Erosion of embankment soils was considered a likely cause of approach settlement only in Districts 2 and 5.

One mechanism not listed in the survey but consistently mentioned by survey respondents was thermal bridge movement. Expansion of bridges during warm days causes lateral displacement of the soils adjacent to the abutment. Subsequent cooling and shrinkage of the bridge results in the formation of a gap between the soil and the abutment. Traffic loading then pushes the approach pavement down as the underlying soil moves laterally to fill the gap. This appears to be an important mechanism not identified in the literature search and not considered before preparation of the survey.

9. **In your opinion what measures could be taken to improve compaction in tight quarters adjacent to abutments approach slabs etc.?**

- "Using a fly ash modified, weak concrete, flowable fill for the embankment within 25 ft. of the abutment."
- "Preload- compact soils-excavate for bridge."
- "Closer specs on material, mandatory inspection tests, and presence of inspector."
- "Better inspection! We have excavated two to four feet on several approaches and found construction debris, wood pieces, tar paper, etc. More intense, quality inspection would mitigate some of the natural problems."
- "Use controlled density fills, i.e. lean/ fly ash concrete."
- "Greater emphasis on manual compaction."
- "Mechanical compaction with power whackers, pogo sticks, etc."
- "Use quality borrow in embankments, use approved compaction methods, have diligent inspectors."
- "More instruction to inspectors - additional compaction testing - always build embankments up to sub-grade before constructing abutments."
- "Closer inspection. Specify better quality material for embankments."
- "Specify a hoe-ram type compactor."
- "Better inspection-more water while filling, use proper materials."
- "Required granular material - have special bid item for compaction with more test and inspection."
- "Better attention to details of compaction and controlled density fill (cement/fly ash/aggregate)."
- "Use 4" lifts, 100 percent compaction, rigid testing of density and contractors methods, material on the dry side of maximum moisture, or crushed manufactured material."
- "Unless bridges are constructed differently, the problem will continue, or in other words, it doesn't really matter how we compact if this is not the real problem."
- "Be sure that the design of the backwall-catch basins, etc., does not preclude proper compaction."
- "Compaction can be obtained with hand equipment."
- "Insist that an area at least 100 ft. in length and full width be brought up as a unit rather than just a small area in the proximity of the abutment. We should utilize as large a piece of equipment as possible."
- "Treating this area differently from general embankments area. Have a special item with tighter specifications for this area. Use smaller equipment."

- "I-90 S. Fork IC MP 31 slabs bridged settlement until fill settled."
- "290/12N (one winter cycle)"
- "90/581, 290/15N, 27/27"
- "SR 90 east approach to floating bridge, SR 5 for 220th St. S.W. undercrossing."

13. Please list any bridges you are aware of where approach slabs were used but where their performance was poor.

- "I-5 M.P. 233I - Samish River Bridge."
- "SR 516 Green River Bridge."
- "405/35W MP 11 SR I-90 OC"
- "155/101 Contract remove rotted wood retainer from west end of Br. in 1989, functioning ok now."
- "26/110"
- "SR-101 Bernard Creek Br."
- "SR-101 Naselle River Bridge 101/24, SR-5 Kalama R. 5/112 E&W."
- "SR-5, Kalama R. Bridges M.P. 31.8"
- "SR 82 W.B. at Granger-There's a settlement problem- may not be a problem with the approach slab."
- "82/1N, 255 S, 7"
- "Most of our failures when approach slabs used or not with slab but expansion joint failure-several of these on I-82 and I-182."
- "Can't think of any that are poor. One problem in the past was large opening developing at the pavement seat joint. This has been corrected by anchors and compression seals in recent design.
- "None to my knowledge (maybe SB 405 over I-90 at Factoria interchange)."
- "East end of 90/540 S"
- "SR 167 bridges from Kent to Auburn."

14. In what percentage of bridges with approach slabs does the use of the approach slab simply move the bump from the end of the bridge to the end of the approach slab?

	Percentage of Total Number of District's Responses				
District	0-20%	20-40%	40-60%	60-80%	80-100%
1	83	17	0	0	0
2	100	0	0	0	0
3	50	0	0	0	0
4	50	50	0	0	0
5	6	14	0	0	0
6	100	0	0	0	0

The phenomenon of approach slabs moving the bump from the abutment to the end of the approach slab reported by investigators in other states clearly does not appear to be a significant occurrence in Washington.

15. In your opinion, what is the cause of bumps at the end of approach slabs?

- "Poor embankment construction and settlement in underlying soils."
- "Poor subgrade."
- "Compaction, materials, quality control on construction."
- "Same as next to bridge, poor compaction and inadequate support base width. Base spread is difficult to detect but obviously occurs when approaches settle in some cases."
- "Normally pavement wear/ rutting sometime are all settlement of grade."
- "Poor soil material (sand) and poor compaction."
- "Frost"
- "Supporting base consolidates or erodes and leading edge of slabs drop."
- "Deep foundations with low strength that can not support a fill without subsidence."
- "Settlement of the embankment."
- "Usually an alignment problem, paver take off causing a bump at the joint."

- "Bump only occurs if approach slab is not extended for enough to cover width of excavation."
- "Lack of compaction -water in embankment."
- "Thermal expansion of bridge causing end of approach slab to move and raise a bump of A.C. pavement. This is not a common situation. Snowplows often plane off the bump."
- "Low compaction as fill is built."
- "The bridge approach slab has been moved by bridge expansion- probable cause is migration of incompressibles into expansion joint or inadequate joint opening on multiple span bridges."
- "Poor construction techniques or excessive grade."
- "Dissimilar material-varied amounts of deflection and load transmissibility. Intrusion of moisture at the joint between P.C.C approach slabs and A.C.pavements."
- "Not planning ahead when paving."

Given that the responses to the previous question indicated that bumps at the end of approach slabs are not a significant occurrence in Washington, this question may have little applicability. However, it is interesting to note that lack of compaction in the tight quarters adjacent to the end of the approach slab was the most common explanation offered for the development of bumps at the end of approach slabs.

16. What factors do <u>you</u> think should be used to determine whether approach slabs should be used at a particular bridge site?

- "Approach slabs should be used on 100 percent of bridges."
- "Use on all bridges."
- "Base width of approach fill, caused by R/W constraints. General location for instance: If the roadway fill is constructed on a western Washington valley floor, all bridges should probably have approach slabs."
- "Should probably use approach slabs on all bridges or use controlled density fill for approach embankments."
- "Approach slabs should be used on all bridge sites."
- "Truck traffic, frost susceptible soils."
- "Height, slope, and stability of embankment."
- "Ability to maintain profile grade."
- "Subsurface soil conditions-any post tensioned bridge should have an approach slab."
- "Cost of construction on retrofit jobs. If they're doing deck repair they should always install slabs. All new construction on roads with 2500 ADT or more should have slabs."

- "District policy is to use approach slabs on all bridges. I believe the policy is a good one."
- "Should be used on all bridges."
- "Traffic Volume- type of bridge."
- "1. Traffic volume and speed 2. Type of bridge and size of bridge 3. Type of embankment material 4. Probability of ground settlement, below embankment or foundation. 5. Range of bridge end movement 6. Grade of approach roadway."
- "Type of bridge, amount of traffic, type of soil."
- "All bridges that are constructed to expand into bridge embankments must be built with an approach to mitigate bridge end problems. Exceptions would be very short spans. Best solution would be bridge approach embankment retaining wall construction with control of bridge expansion through expansion joints."
- "Type of material in approach fill volume of traffic."
- "I believe approach slabs should be used at all locations unless there is a parapet wall (exp. joint at the end of the structure)."
- "An approach slab should be used unless conditions can be proven to not require it. Looking at bridges in district 6, if your R-value for the fill is below +/- 60-70, use an approach slab. Personally, I think we should always use an approach slab."
- "Always use them."

A significant number of the survey respondents felt that approach slabs should be used without exception on all bridges.

17. Please list any other comments you might have regarding the use and effectiveness of bridge approach slabs.

- "Use on all bridges. This will save dollars and problems in the life span of any bridge."
- "Approach slabs gone bad are very difficult to work with. Erosion beneath them, cracking, settlement, all seem to compound the problem of maintenance and repair. Especially when availability of the system to ever-increasing traffic demands is becoming more vital. Working on problem approach slab is <u>not</u> a short term or quick-fix today, particularly if you are under heavy traffic."
- "In most cases approach slabs have required very little maintenance. Occasionally an approach slab will break up due to voids."
- "All other factors being equal, approach slabs are preferred; however, if there is settlement maintenance costs are greater with slabs."

- "They are effective only when the bridge length totally crosses the soft foundation. All bridges that have subsidence near the approach are too short. However, the cost to lengthen is far in excess of the cost to repair 5-6 times. Approach slabs are generally use less."
- "Have soils engineer make recommendation concerning settlement and the need for approach slabs in bridge soils report."
- "We've experienced cavernous washouts under two approach slabs due to poor bridge drain design and or performance. I suspect we have significant voids under almost all approach slabs due to settlement of fill and/or infiltration of water."
- "I think approach slabs help to transition between the bridge and grade. It is much easier and cheaper to install the approach slabs during bridge construction than to come back later and install the approach slab."
- "I think bridge approach slabs are very effective, but just as important are expansion joints and drains. We must get better design on expansion joints to keep moisture from entering embankments and drains must be located at correct location to keep water away from expansions and embankments."
- "We have had a number of past policies on use of slabs in this district in the past. Once it was "only where there was adjacent concrete pavement." Later, it was " all bridges on Interstate but only selected bridges elsewhere." Now it is "all bridges unless otherwise determined." We are retrofitting slabs on bridges where settlement has been a problem. Obviously, we in district 5 recognize the effectiveness and need for approach slabs, but often for reasons other than classical differential settlement."
- "I believe the DOT has not used enough bridge approach slabs."
- "Bridge approach slabs may mitigate problems, but in may cases may shift problem to the end of slab if expansion joint is not properly designed or maintained."
- "Additional data are readily available in the district files."
- "An approach slab allows full use of normal compaction equipment on the A.C.P pavement coming into it. You should not run vibratory compactors against the bridge seat. Poor pavement at bridge ends is not just a bump, it's numerous things, post compaction, shifting of fills, fill foundation settlement, rutting, etc, It cannot be easily dismissed."
- "Why shouldn't we provide approach slabs for all bridges?"
- "Make sure the approach slabs are on well compacted fill material, preload fill material if necessary before placing slab. Make sure drainage from bridge does not undermine slab."

FIELD INVESTIGATIONS

In an effort to obtain site-specific information on the nature and possible causes of bridge approach distress, a field investigation program was undertaken. The field investigation evaluated the cause(s) of bridge approach distress observed at sites for which no approach distress was expected. Such sites were considered to be those with all of the following characteristics:

a. no approach slabs,

b. poor approach performance (significant maintenance required),

c. low (or no) approach embankments (approaches in cut sections or sections with 5 feet or less fill thickness), and

d. incompressible foundation soils (foundations on rock or very stiff soil unlikely to consolidate and settle significantly).

An additional class of sites considered for field investigation comprised those in which thermal bridge movements appeared to play a significant role in causing localized roadway deformations.

Potential sites for detailed field investigation were identified through discussions with WSDOT geotechnical engineering personnel and through the use of a second questionnaire. The second questionnaire was sent to all respondents to the first questionnaire, and it asked them to identify specific bridges that they felt fit either of the types described above.

Below are described the bridges studied, parameters measured at each site, and the results of the field investigations, listed in the order they were performed.

Background Information

A total of 47 bridges were identified for potential detailed field investigation. Additional background information on these bridges was obtained by telephone interviews with local district personnel and by review of design and construction files in Olympia, Tumwater, and district offices. On the basis of the available background information and various logistical and scheduling constraints, nine bridges were selected for actual field investigation. The locations of the nine bridges are shown in Figure 1.

Field Work

The nature of the bridge approach settlement problem required that the field investigations be largely observational, and suggested that firm conclusions regarding specific causes of distress at the individual bridges could be difficult to draw. Nevertheless, an effort was made to measure a number of quantifiable parameters that could provide direct or indirect evidence in support of various hypotheses regarding the causes of distress.

Grain Size Distributions

Bulk samples were obtained for subsequent laboratory grain size analyses. The results of these analyses were compared with the grain size distributions of the soils during construction (if known) to determine whether certain particles, e.g. fines, had migrated during the life of the approach. The grain size characteristics were also compared with the physical location of the samples to evaluate whether segregation/migration of fine particles had occurred. Toward that end, the effective grain size (D_{10}) and percentage of fines within each sample were carefully noted.

Densities

To evaluate in situ compaction characteristics of the basecourse and subgrade materials, in situ density was measured with one or two techniques. Both the backscatter and direct transmission modes of a nuclear density gauge were employed for initial density measurements at the first site investigated. Out of concern for possible inaccuracies associated with using the nuclear density gauge in a pit, densities were also measured with a standard sand cone test procedure. At subgrade subsequent sites, only the sand cone was used.

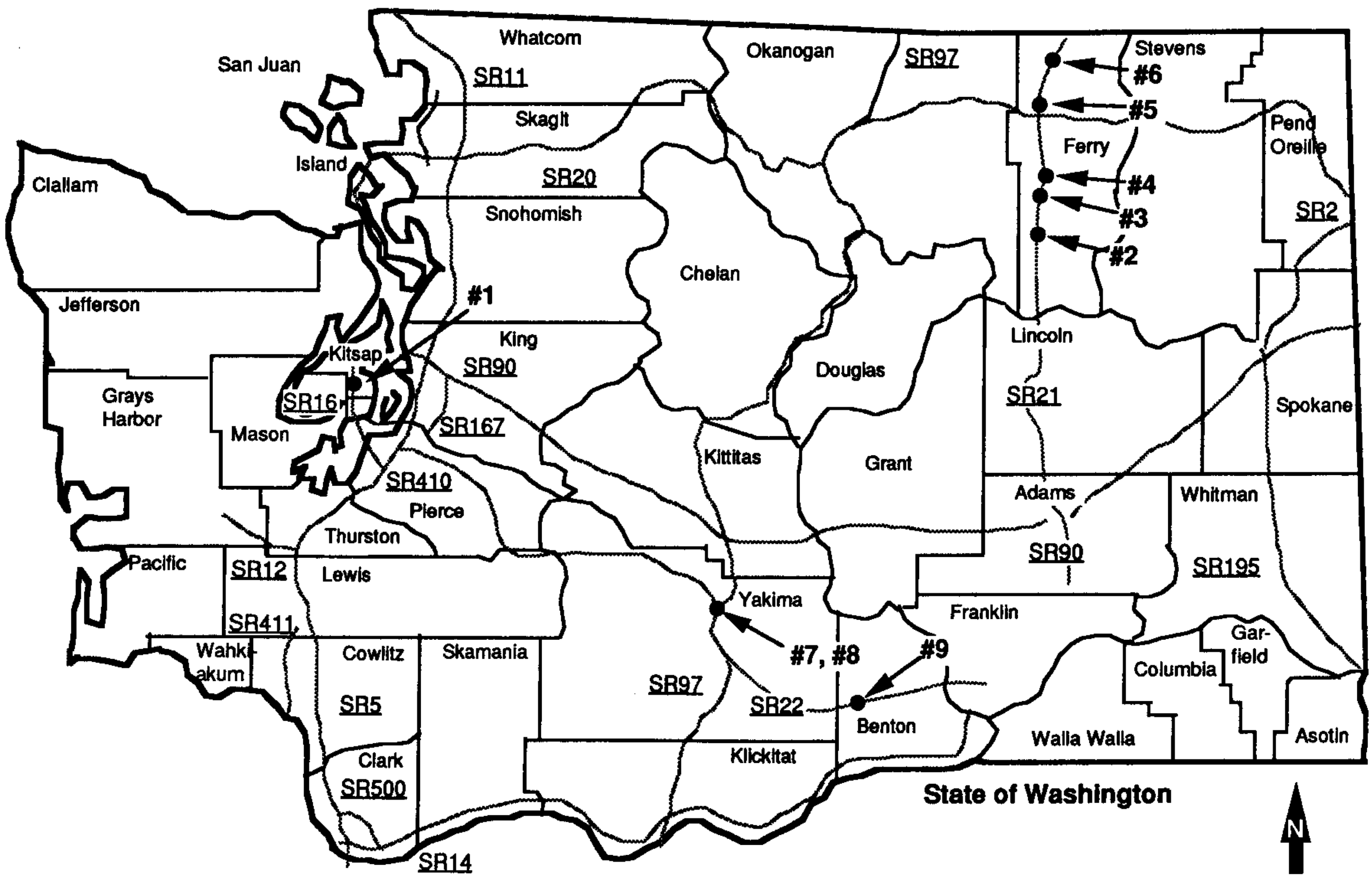


Figure 1. Location of Field Investigations

Bulk samples of the basecourse and subgrade materials were also obtained for laboratory measurement of compaction characteristics. Compaction curves were obtained with the modified Proctor method (ASTM D 1557-78, Method C) for each bulk sample. Determination of the maximum dry unit weight of the soils allows comparison of their in situ relative compaction with the relative compaction specified during construction. However, densities and relative compactions obtained by this procedure must be interpreted with some degree of caution, since vibration-induced densification may have occurred after construction. Vibrations capable of causing densification can be caused by vehicular traffic and by pavement removal during test pit excavation, particularly when pneumatic hammers are used to break the pavement.

Moisture Content

To evaluate the degree and pattern of soil saturation near the abutment, moisture content samples were obtained at various locations in both the basecourse and the subgrade. Samples were taken in metallic moisture content cans, which were immediately sealed for subsequent laboratory testing. The moisture content data were obtained to provide possible evidence of water entering the joint between the bridge deck and the approach pavement.

1. SIDNEY ROAD OVERCROSSING

Location

The Sidney Road overcrossing structure carries SR 16 over Sidney Road at mile post 25.06 in Kitsap County. The bridge is roughly 7.5 miles north of the Pierce County line and 6 miles south of Bremerton, Washington. The northbound overcrossing structure, a profile of which is shown in Figure 2, is a three-span, pretensioned concrete beam bridge 300 feet in length skewed 35 degrees to the mainline of SR 16. A photograph of the southern abutment of the northbound bridge is shown in Figure 3. It was constructed without approach slabs. The Sidney Road overcrossing appears to have the characteristics of a site in which approach distress should not be expected, and where approach slabs

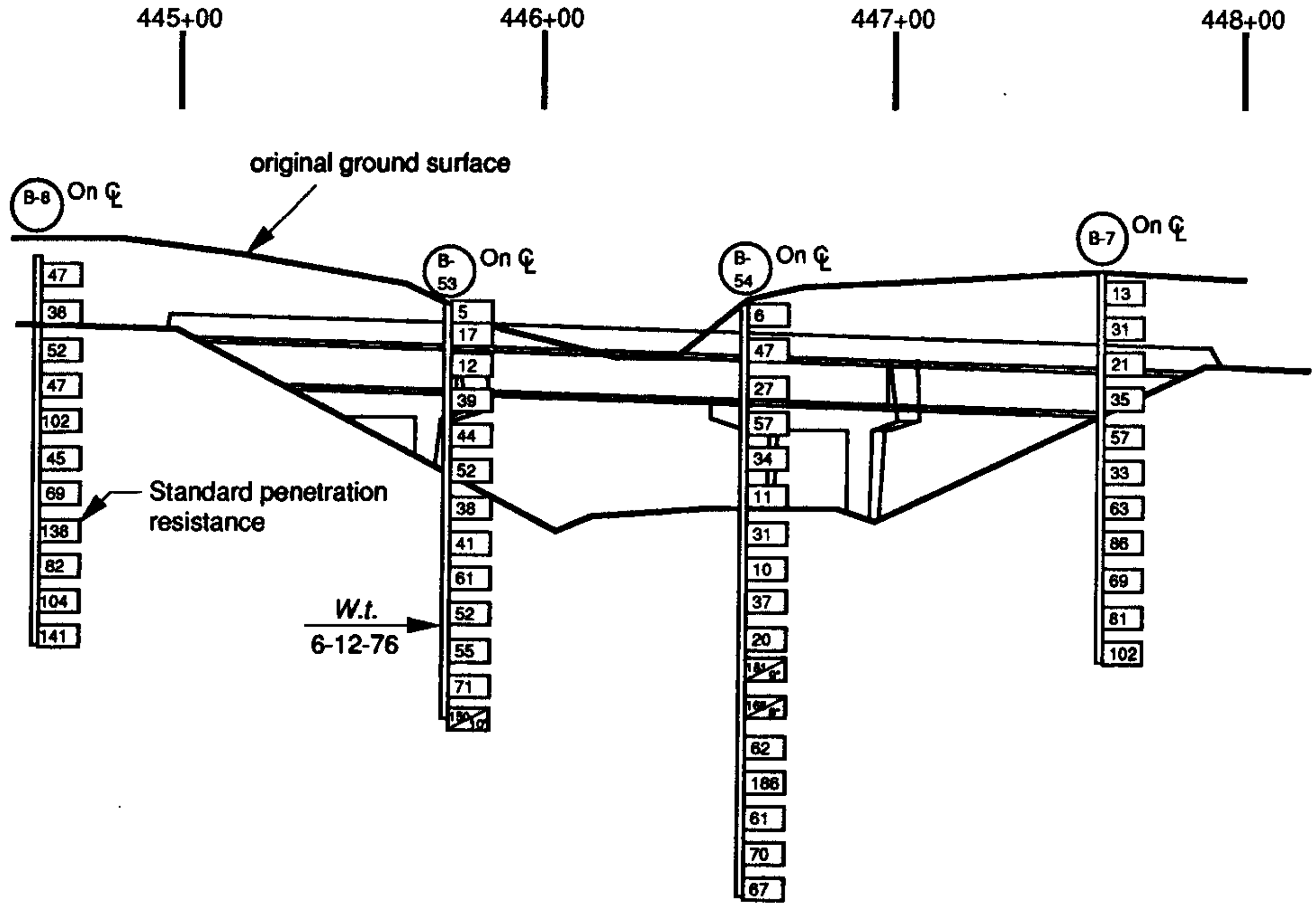


Figure 2. Profile of Sidney Road Overcrossing

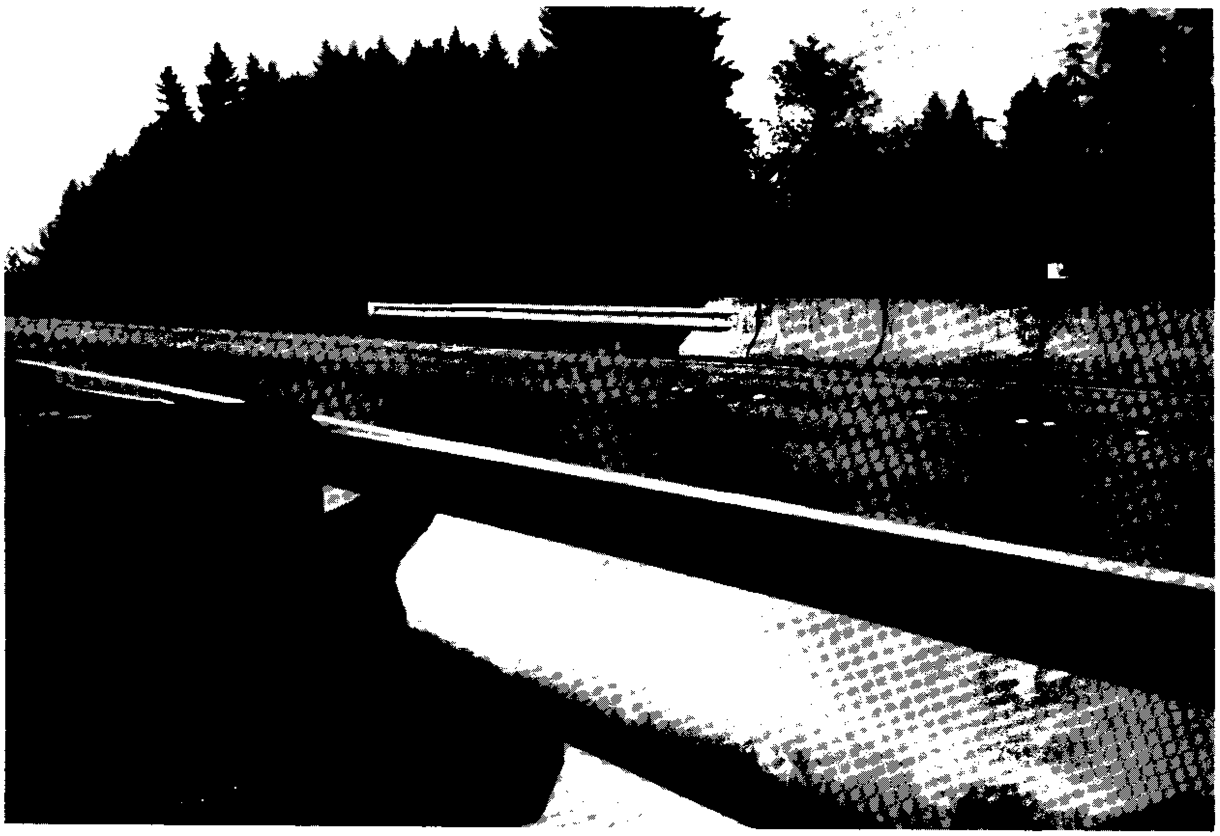


Figure 3. Southern Abutment of the Northbound Bridge

should not be needed. The south end of the northbound structure (Number 16/204 W) warranted a detailed investigation because it is located in a cut section where approach distress cannot result from foundation or embankment fill settlement. Despite these conditions approach distress had still occurred. Thus, the approach distress was expected to have been caused by some local mechanism of deformation.

File Search

A file search was undertaken to review documents pertaining to the Sidney Road overcrossing structure. Files available from the Materials Laboratory contained foundation reports detailing borehole information, recommendations of foundation type, and allowable bearing capacity calculations for the Sidney Road overcrossing structure. The files office in Olympia provided plan sheets, specifications, inspectors' diaries, and project accounting records. Unfortunately, important construction documents such as material and test records were not available for the Sidney Road Bridge, since specific files such as those are removed from the reserve after 12 years.

History

The northbound Sidney Road overcrossing structure was constructed in 1977. Approach construction on the northbound structure involved cuts of approximately 30 feet. The cuts were required to allow passage of Sidney Road beneath the northbound structure while maintaining the grade of SR 16. During earthwork cuts in this area, heavy erosion was often noted. Upon completion of the northbound structure, two-way traffic was routed across the bridge while the south bound structure was under construction. The southbound structure was completed in 1978.

Although detailed maintenance records of approach patching were not available for this structure, maintenance crews indicated that approach patching was performed on only one occasion roughly 5 years ago.

Soils, Foundation, Structure, and Pavement Section

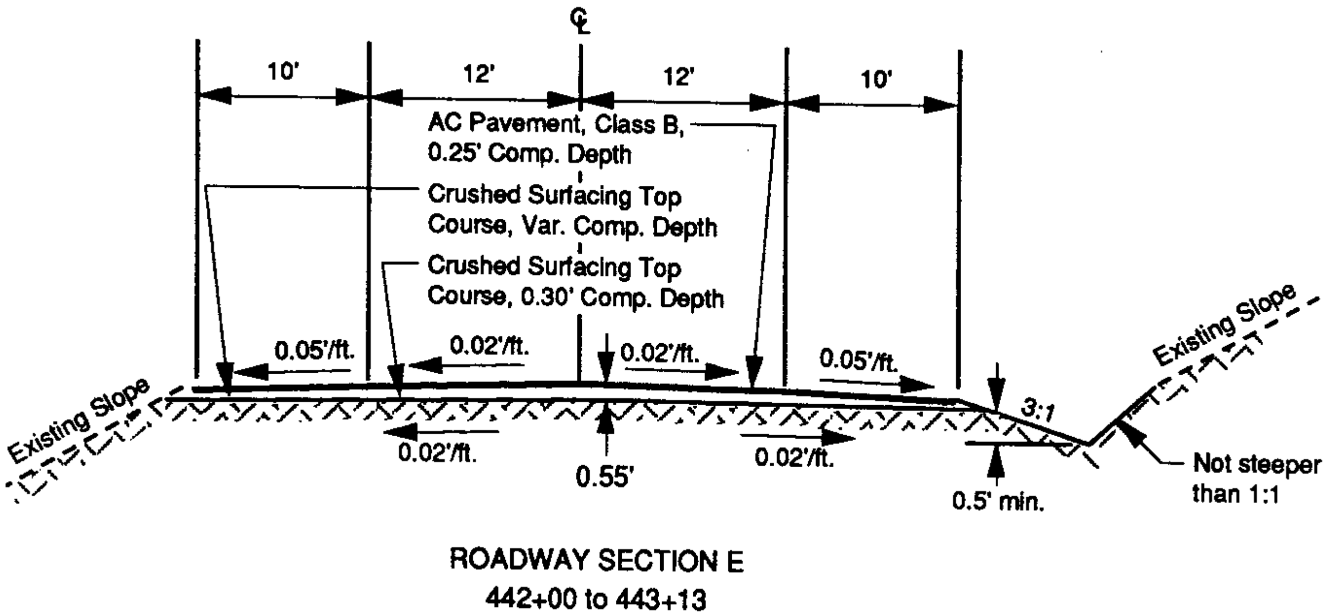
For the foundation investigation, conducted in 1976, a district materials crew drilled nine bore holes to evaluate foundation support requirements. The foundation soils consisted 3 to 17 ft. of loose to medium dense silt and sand underlain by 27 to 44 ft. of dense silt, sand, and gravel containing random, loose pockets down to very dense, silty sand and gravel. The random zones of loose material required temporary slopes to be not steeper than 1.25:1. The approximately 30 ft. deep end slope cuts were shown to be stable on the proposed 2:1 slopes.

The bridge deck is at an average elevation of approximately 242 feet mean sea level. Spread footings were designed at both piers and both abutments to support loads of up to 3 tsf with negligible settlements. Minimal footing elevations were established by the Bridge Division (for piers 1 and 4, 231 and 238 respectively). Footing elevations for the northbound line piers 1 and 4 were located in firm foundation material. Footings for piers 2 and 3 were located at elevation 210 feet and were designed for loads up to 4 tsf. All test holes were dry with the exception of one hole, which indicated water at elevation 202 feet.

A typical pavement section in the approach area is shown in Figure 4. Note that this section is reproduced from the original plans and, therefore, may be different from the "as-constructed" plans.

Approach Distress

Approach distress on the north bound structure was evident from the patched/cracked zones that extended 6 to 9 feet from the bridge entrance. Approach distress was also observed at the bridge exit in the form of patching, which extended 5 to 6 feet from the bridge. Asphalt approach patching did not extend onto the concrete bridge deck, but it covered both lanes and a small portion of the shoulders. Approach patching appears to have been effective in reducing what would probably have been a rough riding bridge transition.



SR 16, MP 24.71 to MP 26.00
Sedgwick Rd. I/C & Undercrossing
and Sidney Rd. Overcrossing

Figure 4. Design Pavement Section for Sidney Road Overcrossing

Site Conditions

The field investigation of the northbound structure took place on Friday, July 13, 1990, under sunny skies. A state maintenance crew of three men controlled traffic by closing the right travel lane. The state crew also recompacted and patched the test sections after the field investigation had been completed.

Three test sections were chosen at the locations shown in Figure 5. Sections 1 and 2 were situated in the right wheel path of the travel lane. Test section 1 was located adjacent to the abutment, and test section 2 was near the edge of the zone in which distress was observed. For comparison, test section 3 was located adjacent to the abutment in the shoulder, where wheel loading was expected to be minimal.

Observations

State crew members cut the asphalt pavement with a jackhammer and carefully removed it to differentiate the skin patches and any other asphalt that had been placed after original construction. As the cutting continued, possible pavement overlays or other paving events were noted but were not always easy to distinguish. A photograph of test section 1 after the AC pavement had been removed is shown in Figure 6. Upon removal of the asphalt pavement, basecourse samples were taken and tests performed. The basecourse materials were then removed to expose the surface of the subgrade soils. At approximately the level of the bottom of the pavement, the backwall of the abutment protruded horizontally for about 7 inches, forming a pavement seat as shown in Figure 7. Asphalt and basecourse section thicknesses were measured and are shown in Figure 8. A 1- to 2-inch layer of gravel larger than the basecourse was encountered on top of the subgrade. Generally, the soils near the abutment did not appear to be as stiff and dense as those further from the abutment backwall.

Using a digging bar as a probe, the crew encountered a large void in an area behind the abutment backwall near the "corbel," as shown in Figure 7. This void appeared to have formed along the entire backwall, since it was nearly impossible to compact soils under the

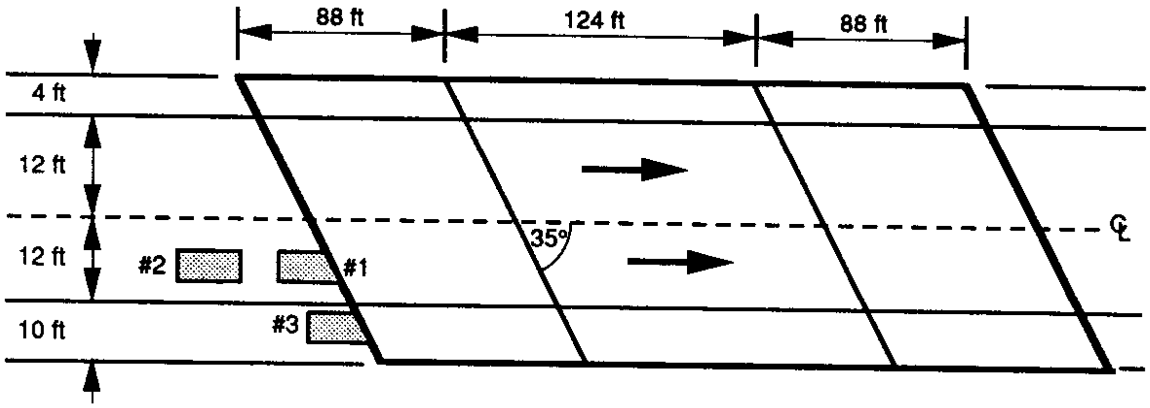


Figure 5. Location of Test Sections in Northbound Lanes of Sidney Road Overcrossing

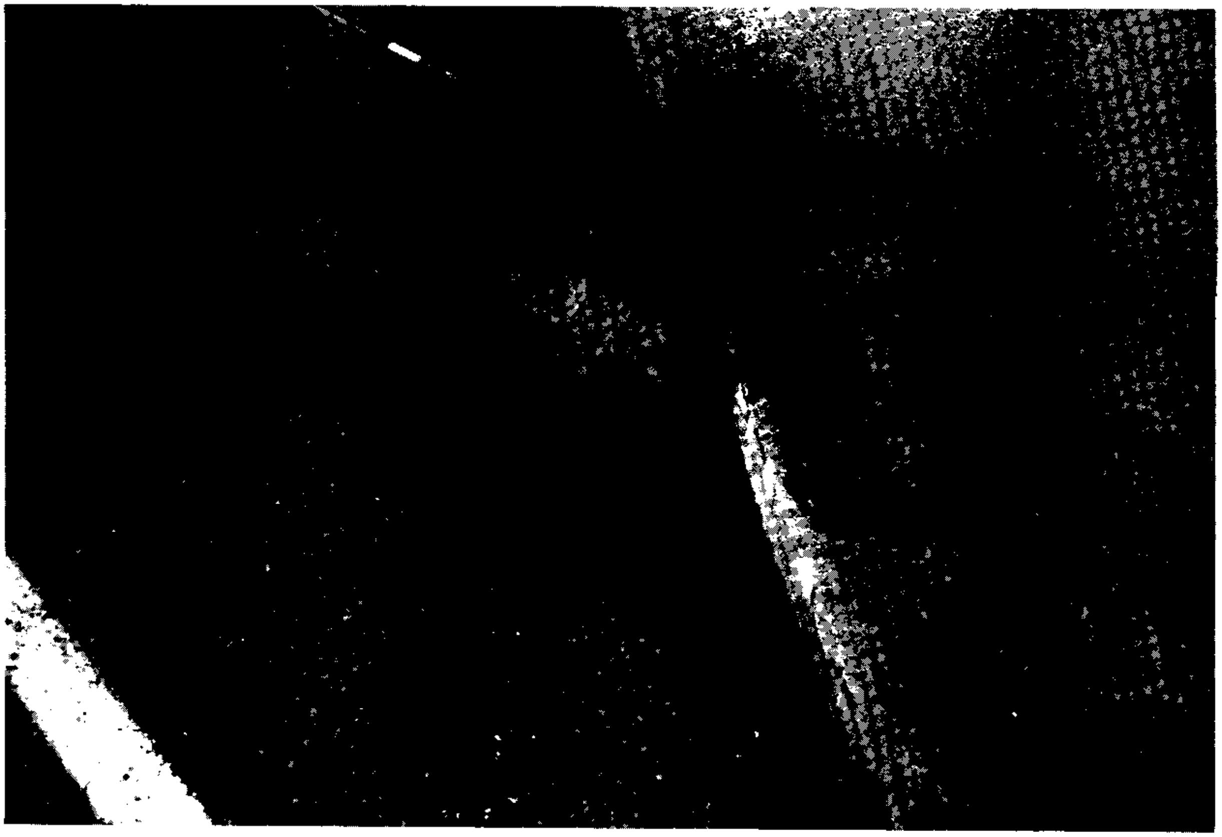


Figure 6. Test Section 1 After Test Section has been Removed

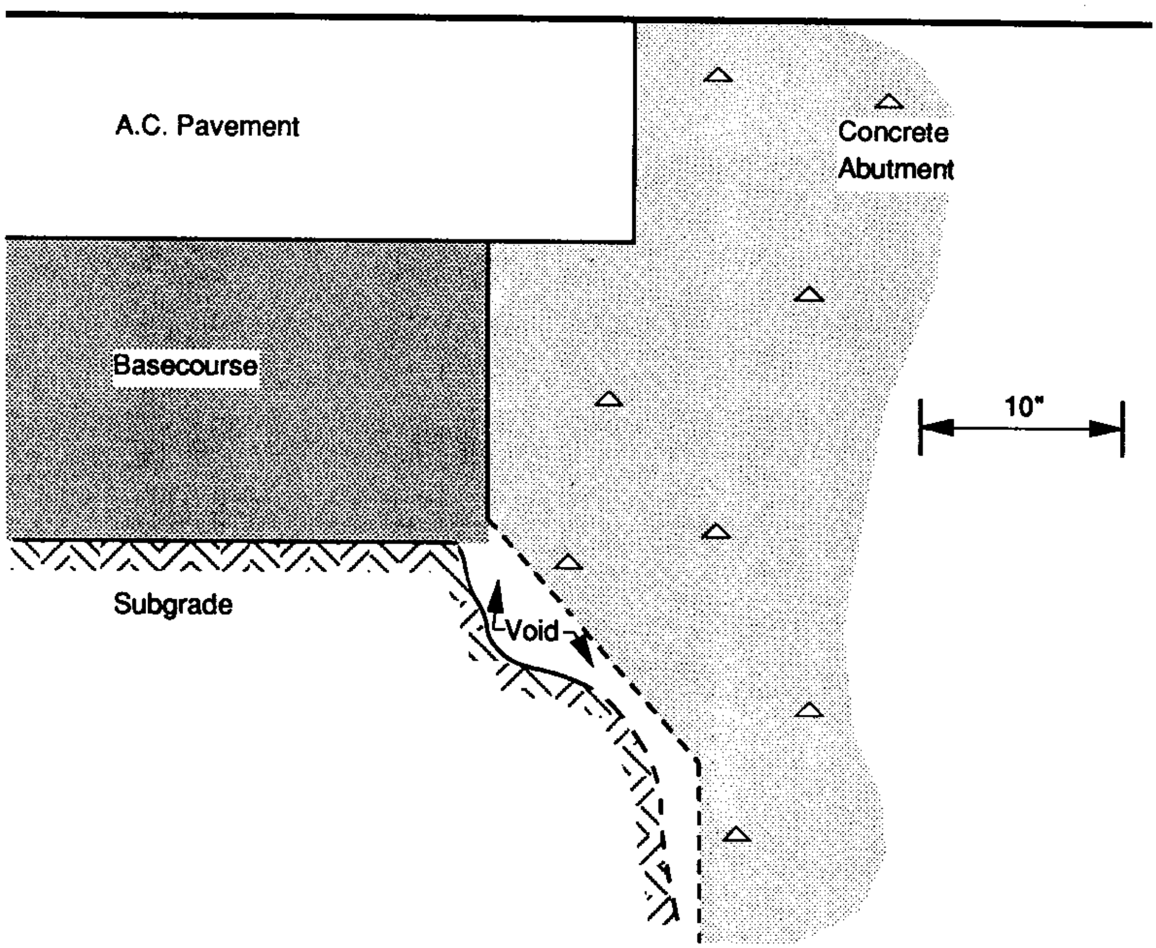


Figure 7. Profile through Sidney Road Overcrossing Abutment

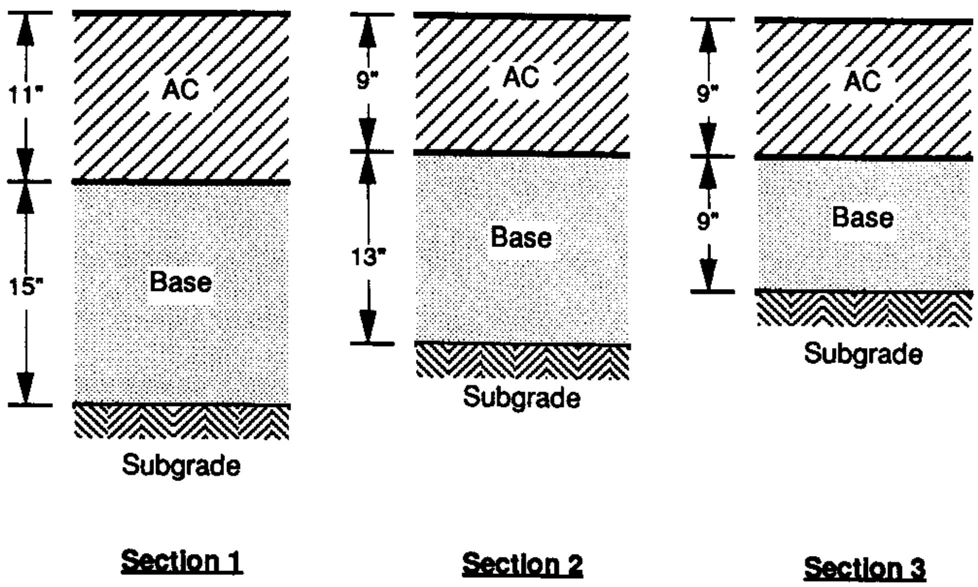


Figure 8. Subsurface Conditions at Excavated Sections, Sidney Road Overcrossing

corbel. The void directly under the corbel ranged from 4 to 5 inches deep and extended 5 to 6 inches from the abutment backwall. The void appeared to continue down along the abutment backwall as a 1- to 2-inch gap between the abutment and the soil. Around the void, the soils were loose and soft enough to excavate by hand. Figure 9 is a photograph of the upper portion of the void.

Deposits of fines and fine sands had been washed out from under the bridge and onto the slope protection beneath the bridge. Water passageways were also noted on the west side of the bridge in an area of significant erosion and very damp soils. Apparently, the surface water was supposed to have been trapped in catch basins located several feet off opposite ends of the bridge in the ditch line. However, the catch basins were not in the optimum drainage path for this section of crowned highway.

Test Results

1. Gradation. Grain size samples were obtained at three locations within the basecourse and within the subgrade. Grain size distributions for the basecourse are shown in Figure 10 and compared with the WSDOT specification 9-03.9(3) for crushed surfacing top course in Table 1. The grain size distribution for test section 2, the section furthest from the abutment, was relatively close to the WSDOT specification, though the distribution from test sections 1 and 3 showed higher percentages of grains retained on the 5/8" sieve. Gradations for test sections 1 and 3 also produced lower percentages of grains that could pass the 1/4" sieve. The sand size (#40) and fine portion (#200) were within specification. The fines content was within specification for samples taken within the first 6 inches of the basecourse. Deviations from the specification may have resulted in part from difficulties in spreading the basecourse near the abutment of a skewed bridge. The percentages of fines and the D_{10} values are shown at the locations of the various grain size distribution samples in Figure 11.

Figure 9. Upper Portion of the Void in an Area Behind the Abutment Backwall Near the "Corbel"

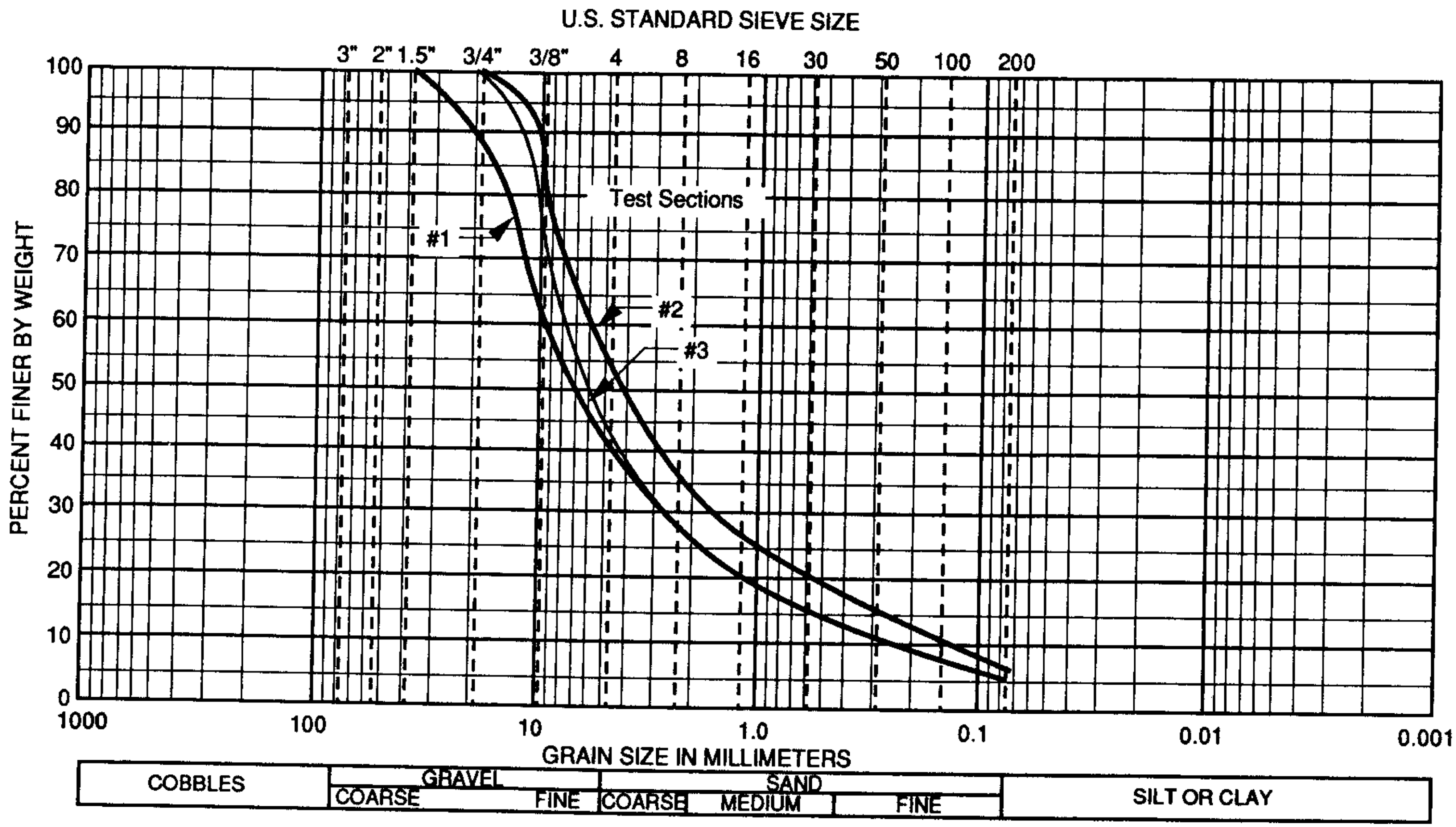


Figure 10. Grain Size Distribution of Sidney Road Overcrossing Basecourse

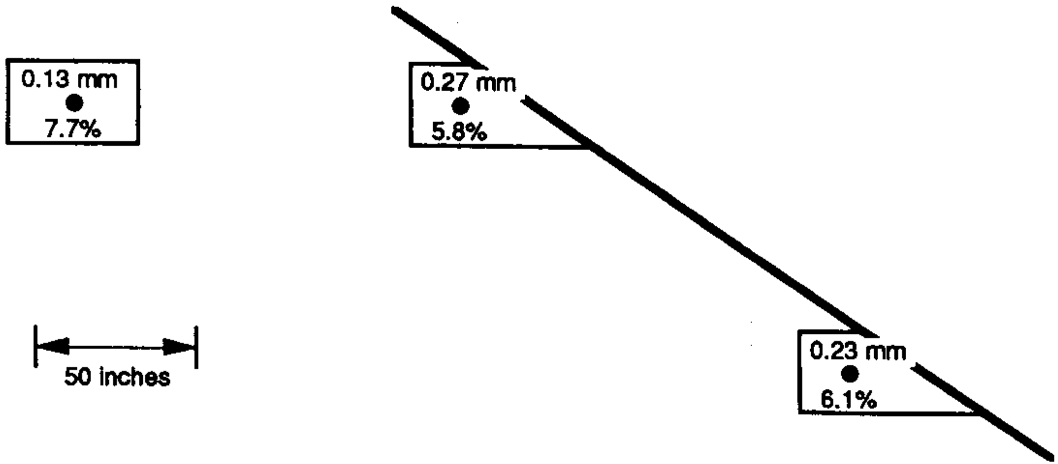


Figure 11. Percent Fines and D_{30} in Sidney Road Overcrossing Basecourse

Percent Passing by Weight

SIEVE	9-03.9(3) Crushed Surfacing Topcourse	Test Section #1	Test Section #2	Test Section # 3
5/8"	100	76	97	93
1/4"	55 - 75	47	62	51
#40	8 - 24	12	17	12
#200	10 max.	5.8	7.7	6.1

Table 1. Basecourse Gradations from Test Sections Compared with WSDOT Specifications

The grain size distribution of the subgrade materials, obtained from samples taken within the upper foot of the subgrade, was relatively consistent. It was approximately 21 percent fines (-#200) and had a D_{30} of 0.15mm, as shown in Figure 12. The percentage of fines and the D_{30} values are shown at the locations of the various subgrade samples in Figure 13.

2. Densities. Densities of the basecourse and subgrade, measured at a number of locations, were as shown in Figures 14 and 15. They may have been affected by vibrations caused by the jackhammer during the pavement cutting operation. Density measurement with the nuclear density gauge may have been flawed in deeper test pits because of uncertainty associated with the trench corrections required for moisture content. This problem probably explains the large differences in measured density the sand cone and nuclear gauge produced. Sometimes lower densities were observed in areas closer to the bridge. However, overall, the values were much lower than the maximum dry densities obtained from proctor tests. Modified proctor test results indicated a maximum dry density of 139.5 pcf at an optimum moisture content of 6.5 percent for the subgrade, and a

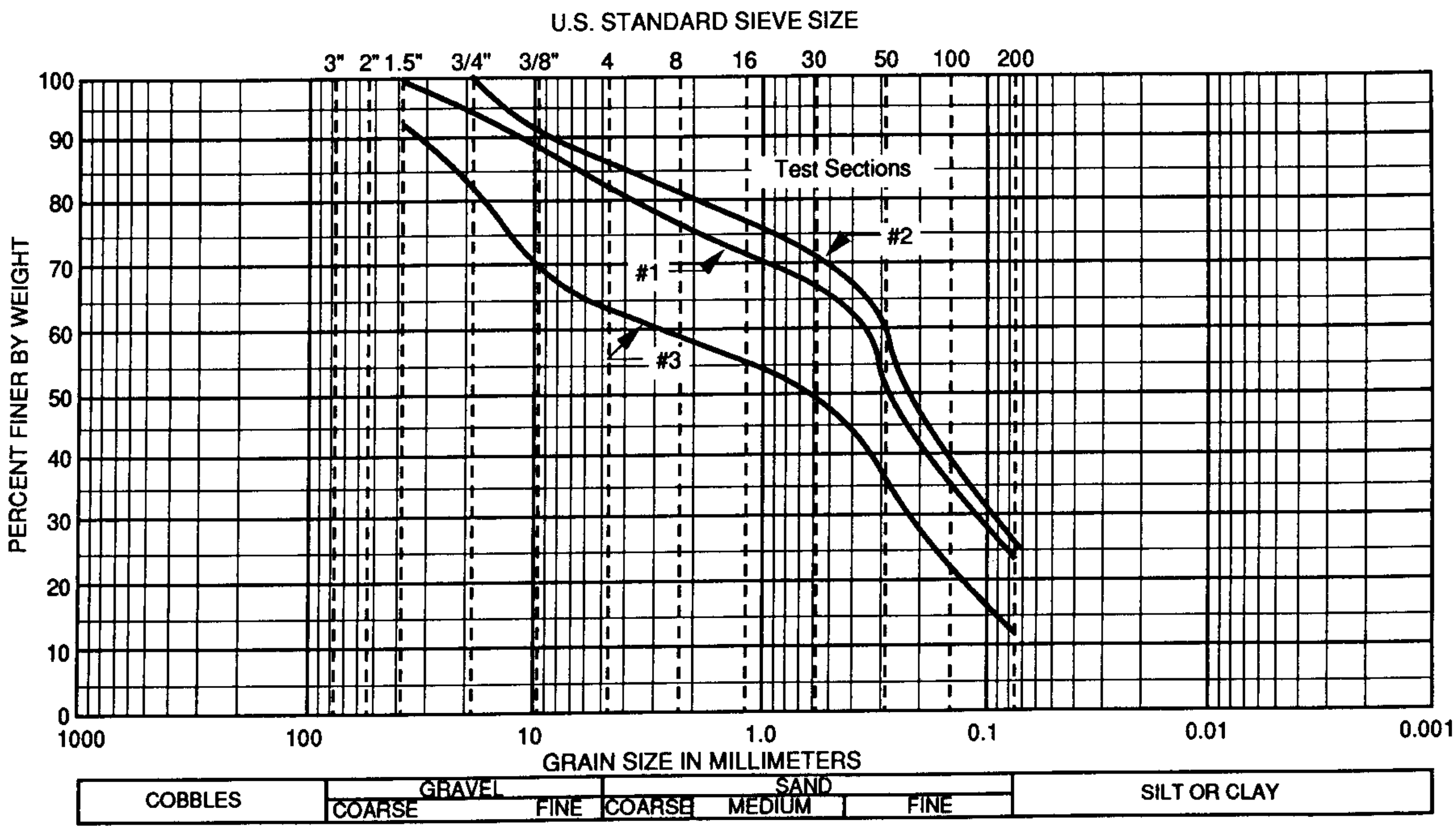


Figure 12. Grain Size Distribution of Sidney Road Overcrossing Subgrade

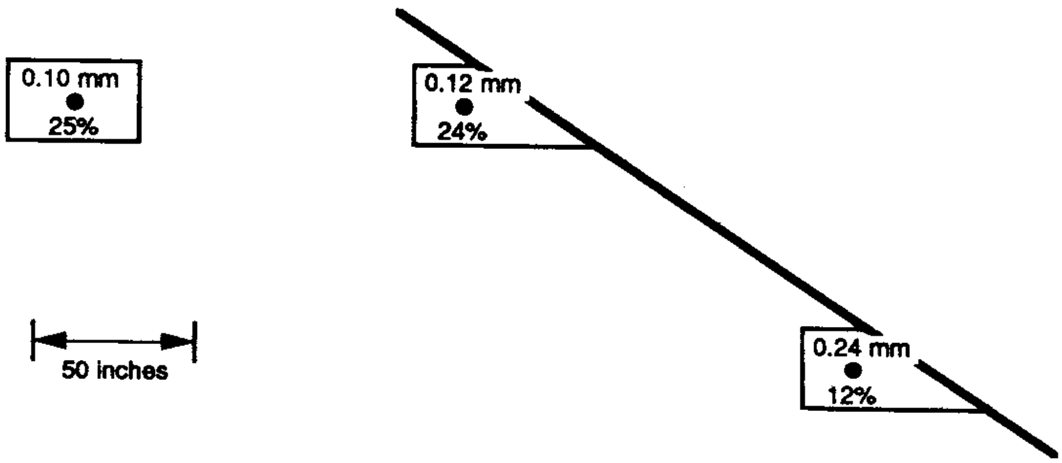


Figure 13. Percent Fines and D_{30} in Sidney Road Overcrossing Subgrade

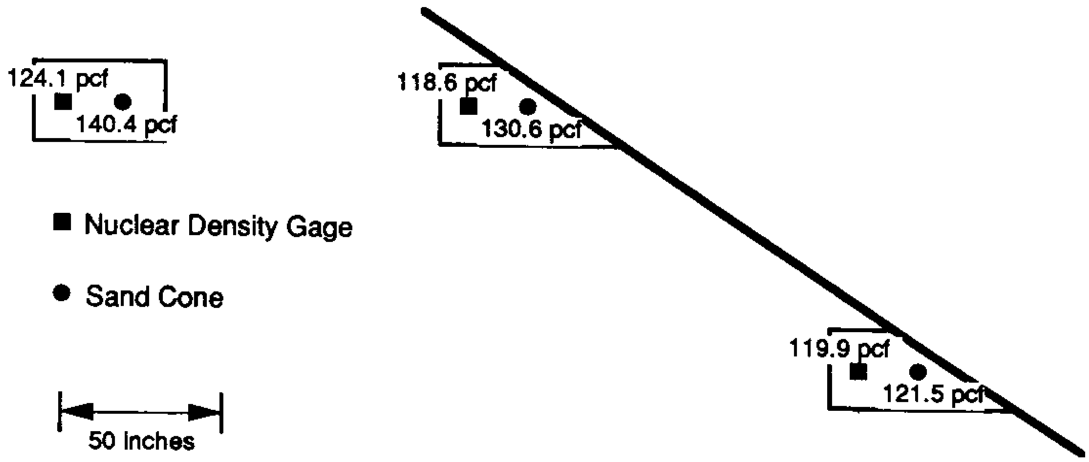


Figure 14. Dry Unit Weights for Sidney Road Overcrossing Basecourse

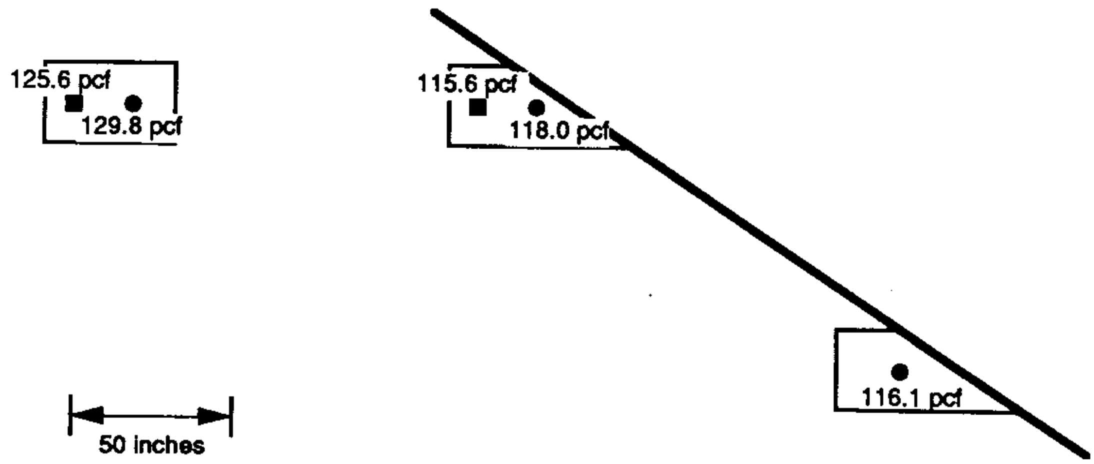


Figure 15. Dry Unit Weights for Sidney Road Overcrossing Subgrade

maximum dry density of 144.0 pcf at an optimum moisture content of 7.5 percent for the basecourse. From these maximum dry densities, relative compactions for the basecourse ranged from 82 to 86 percent when calculated with the nuclear density gauge readings and 84 to 97 percent when based on the sand cone. Relative compaction for the subgrade soils ranged from 82 to 90 percent with the nuclear density gauge and 83 to 93 percent with the sand cone.

3. Moisture Content. Moisture content samples were taken at several locations in each of the three test sections. The results of the moisture content tests on samples from the upper 2 inches of the basecourse are shown in Figure 16. The tests indicated no clear pattern of moisture content in relation to distance from the pavement/abutment joint. Measured moisture contents from samples in the upper 2 inches of the subgrade are shown in Figure 17. Again, no clear relationship between moisture content to location was apparent.

Conclusions

The field investigation at the Sidney Road overcrossing revealed relatively uniform subsurface conditions near the abutment. Some differences in basecourse and subgrade soil properties, e.g., grain size characteristics, density, and moisture content, were observed; however, no distinct pattern capable of causing the observed approach distress was evident.

The primary cause of approach distress at the Sidney Road overcrossing appears to be the void that had formed under the corbel of the abutment backwall. Abutment backwalls designed with corbels that project from the abutment wall create an area in which proper soil compaction is difficult. Excavation into the corbel area clearly displayed a large void at this bridge. The void may have been enlarged by erosion due to water infiltration, particularly if compaction around the abutment and corbel had not been as rigorous as required.

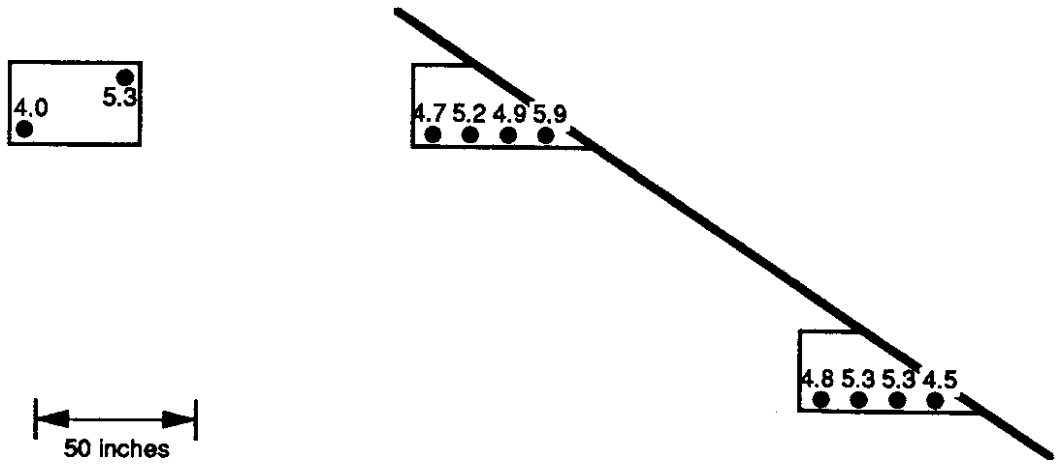


Figure 16. Moisture Content (in percent) of Sidney Road Overcrossing Basecourse

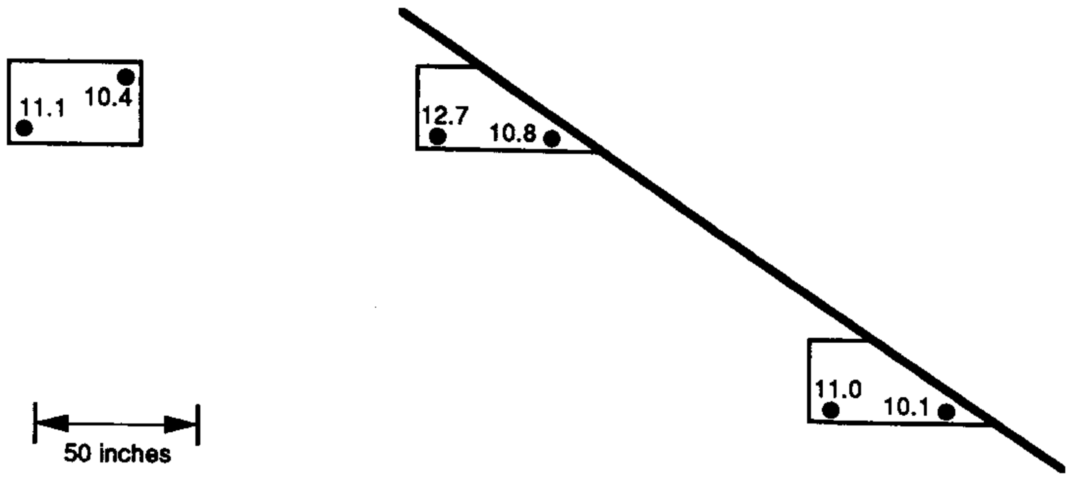


Figure 17. Moisture Content (in percent) of Sidney Road Overcrossing Subgrade

2. SAN POIL RIVER BRIDGE AT MILEPOST 106

Location

The San Poil River Bridge at milepost 106 is 24 miles north of the Lincoln County line and 30 miles south of Republic, Washington, on SR 21. It carries two-way traffic over the San Poil River in Ferry County. A detailed investigation was conducted for this bridge because it met many important criteria: no approach slabs, relatively low approach embankments, and continued approach pavement maintenance. The field testing segment of the investigation focused attention on the southbound lane, where approach distress was the most evident.

File Search

Files were not available for this structure from any of the state engineering record offices.

History

The San Poil River Bridge at milepost 106 was constructed in 1931. The bridge crossing has received numerous applications of bituminous surface treatment (BST). Many sections of SR 21 have been paved with BST every few years, and this bridge was treated two weeks before the field investigation. State maintenance forces have also patched broken parapet walls with concrete.

Soils, Foundation, and Structure

The bridge is located in a steep walled, narrow, flood plain valley. Although foundation soil conditions are unknown, they most likely consist of fluvial deposits, with some swampy areas of finer grained soils. The bridge is a 46-ft., single-span, concrete T-beam structure with full-height abutments. The bridge does not have approach slabs and is not skewed relative to the highway alignment. The type of foundation used to support the abutments is unknown. The approach embankments, which are constructed of a sandy, gravel fill and are approximately 5-6 ft. high, are lined with rip-rap on their upstream faces. The pavement lies directly on the subgrade, without a basecourse layer.

Approach Distress

Observed approach distress was minimal, since this bridge had been resurfaced two weeks before the investigation. The BST extends across the structure in both directions for several hundred feet of the bridge. However, approach distress was indicated by distinct wear marks on the ends of the bridge. The pavement wear marks were areas in which the road surfacing was worn down by automobile tires to reveal the underlying black asphalt. They indicated that approach problems might recur; yet at the time of the investigation, the transition across the bridge was smooth.

Site conditions

The field investigation of the southbound lane was conducted on the morning of Wednesday, September 12, 1990. The morning weather was clear and cool. A state maintenance crew of two men controlled traffic by closing the southbound lane.

One test section was removed in the southbound lane on the departure side of the bridge. The test section, shown in Figure 18, was directly against the abutment and was in the wheel path closest to the shoulder. The state crew also recompacted and patched the test section after the field investigation had been completed.

Observations

The asphalt pavement was cut by state crew members with a jackhammer and carefully removed. The BST pavement was crumbly and adhered to the underlying subgrade soils. Within the test section, about 6 inches from the edge of the abutment, the pavement was only 2 1/2 inches thick, as shown in Figure 19. Farther than about 6 inches from the abutment, the pavement was approximately 5 1/2" thick. The soils were firm at every point, and no voids were observed anywhere in the test section.

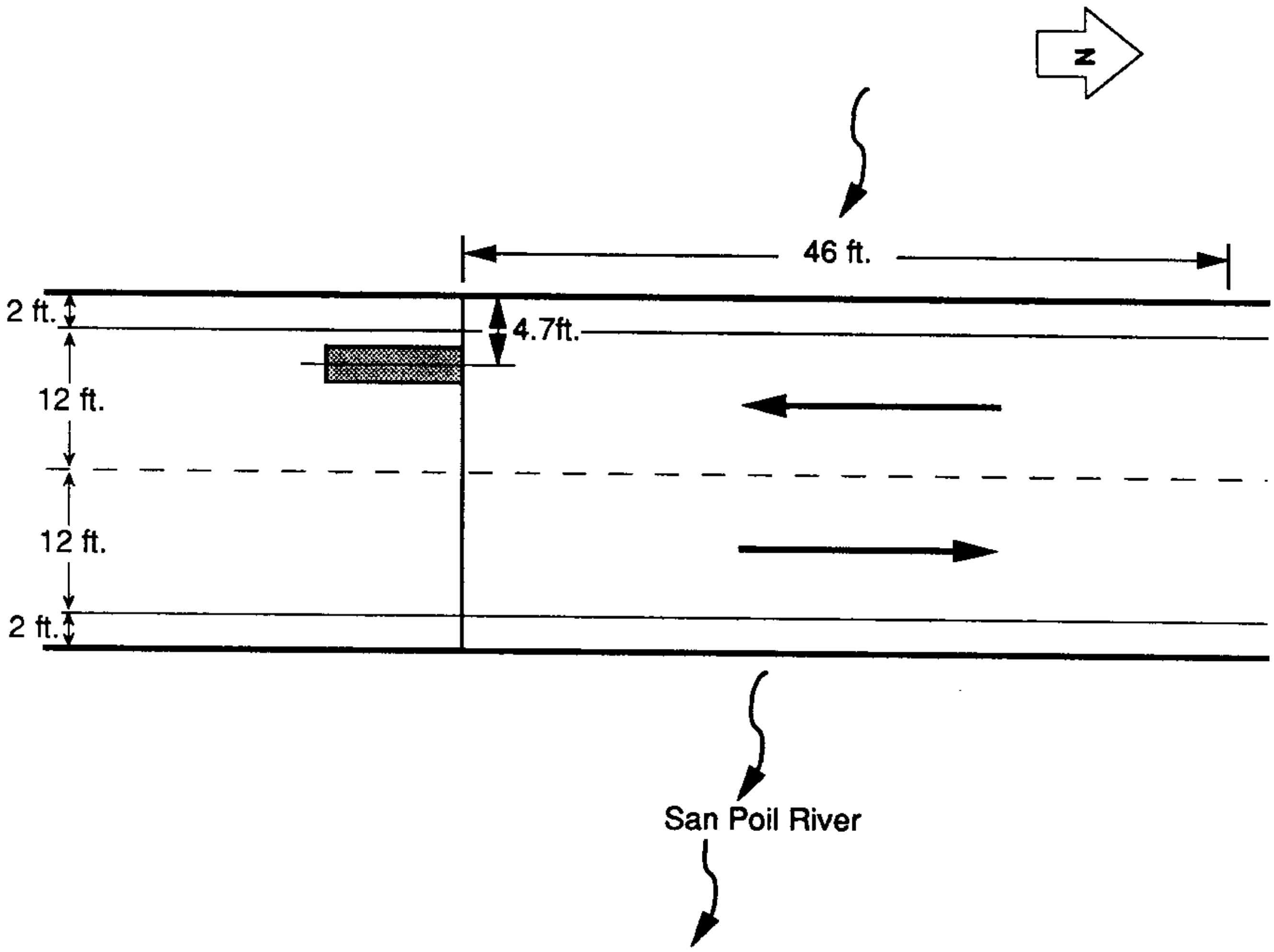


Figure 18. Location of Test Section in Southbound Lane of San Poil River Bridge (M.P. 106) (21/311)

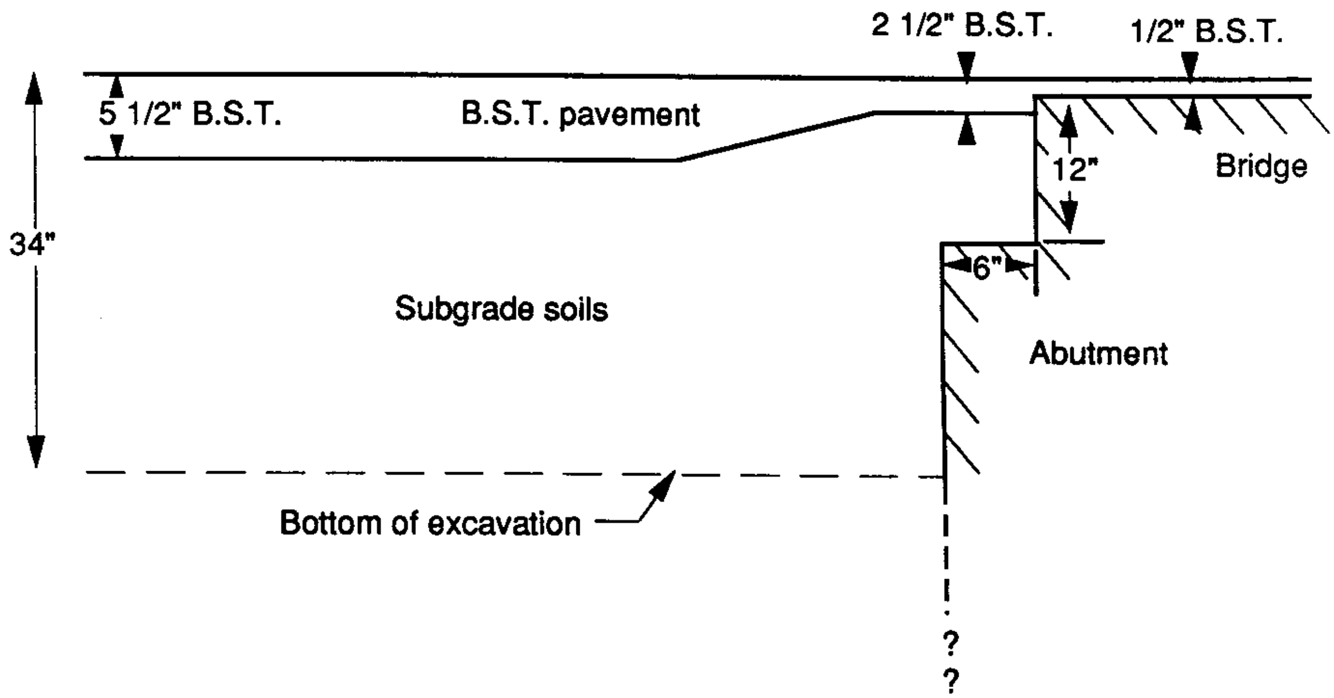


Figure 19. Approach Cross Section at Excavated Test Section, San Poil River Bridge (M.P.106)

Erosion of the embankments from past high water events had been effectively reduced by rip-rapped upstream faces and thick brush growth. The stream channel under the bridge is deep without much apparent deposition taking place.

Test Results

1. Gradation. Grain size samples were obtained from the fill soils in the test section. The samples were taken within the top 10 inches of the fill at distances of 13, 42, and 66 inches from the abutment backwall. Grain size distributions for the fill soils are shown in Figure 20. The grain size distributions of the fill were relatively consistent. The average was 11 percent of fines, with an average D_{30} of 0.39 mm. The distributions of fines percentages and D_{30} are shown in Figure 21.

2. Densities. The densities of the fill soils were measured at the locations shown in Figure 22. The densities were relatively consistent. The dry unit weights of the fill soils averaged 135 pcf, while modified proctor test results indicated a maximum dry density of 137 pcf at an optimum moisture content of 7.5. On the basis of these results, the average relative compactions of the fill was estimated to be 98.5 percent.

3. Moisture Content. Moisture content samples were taken at several locations in the test section, as shown in Figure 23. The tests indicated that water contents gradually increased toward the abutment, indicating that water either has entered near the abutment/embankment interface or is trapped near the abutment.

Conclusions

Approach distress on this bridge has resulted from differential settlement between the bridge deck and the approach embankment. The strongest indication of differential settlement was observed in the sudden change in pavement thickness over the pavement seat area. Field testing and hand probing indicated that the embankment soils were firm and properly compacted. Gradations did not show large deviations, though the measured moisture contents of the embankment soils in the test section tended to describe water

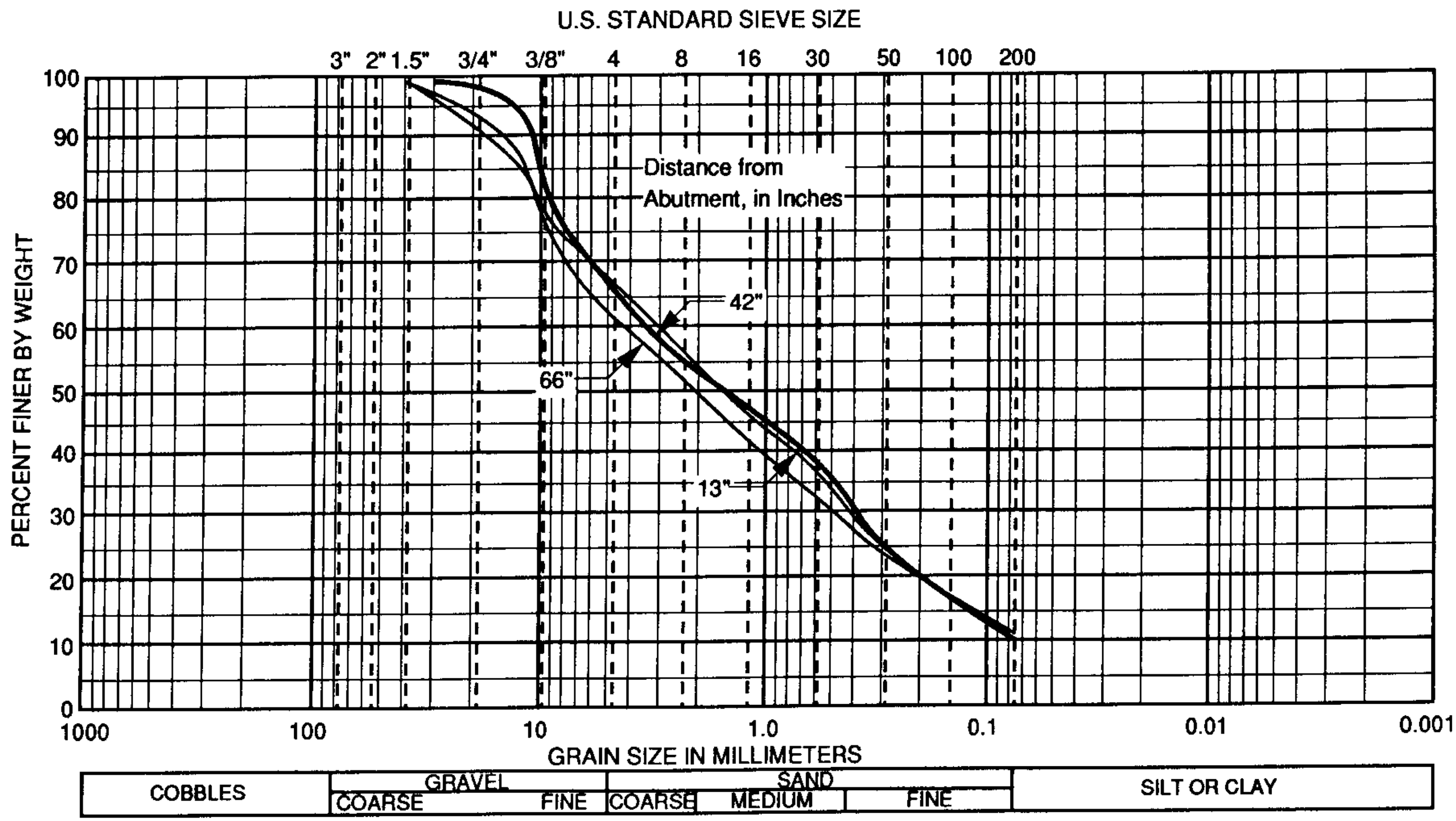


Figure 20. Grain Size Distribution of San Poil River Bridge at M.P. 106 Fill

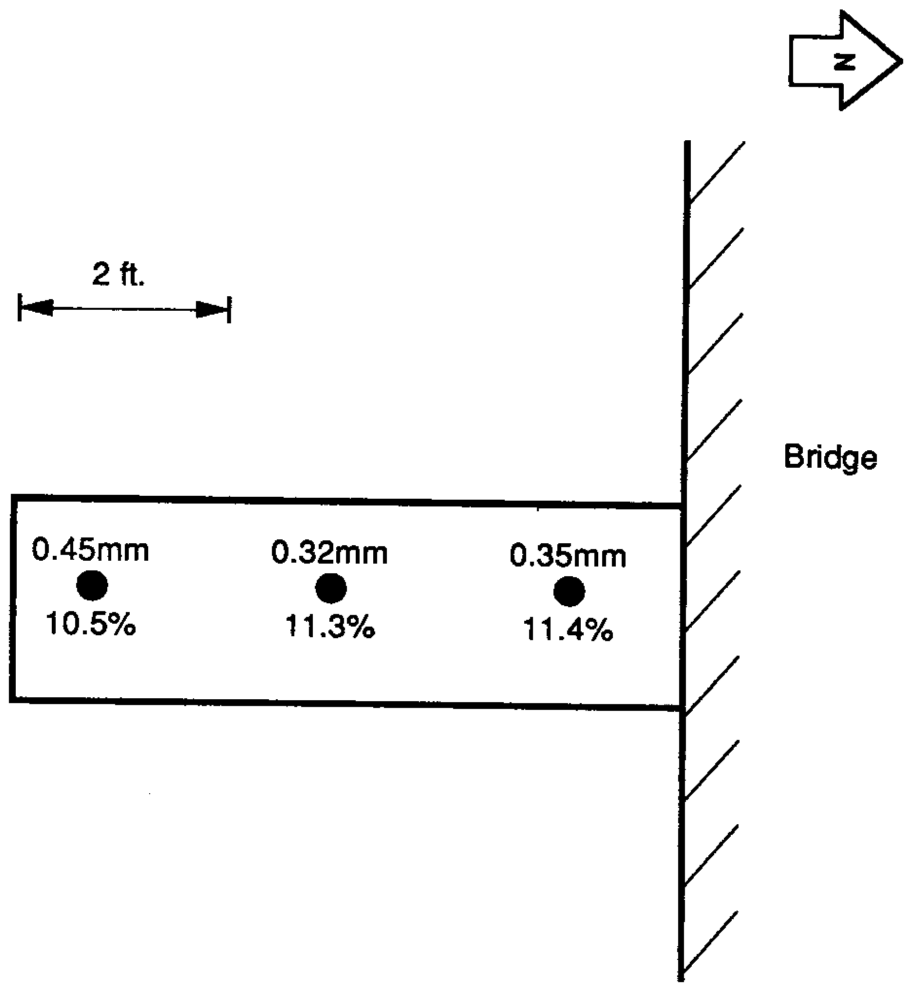


Figure 21. Percent Fines and D_{30} of San Poil River Bridge (M.P. 106) Fill Soils.

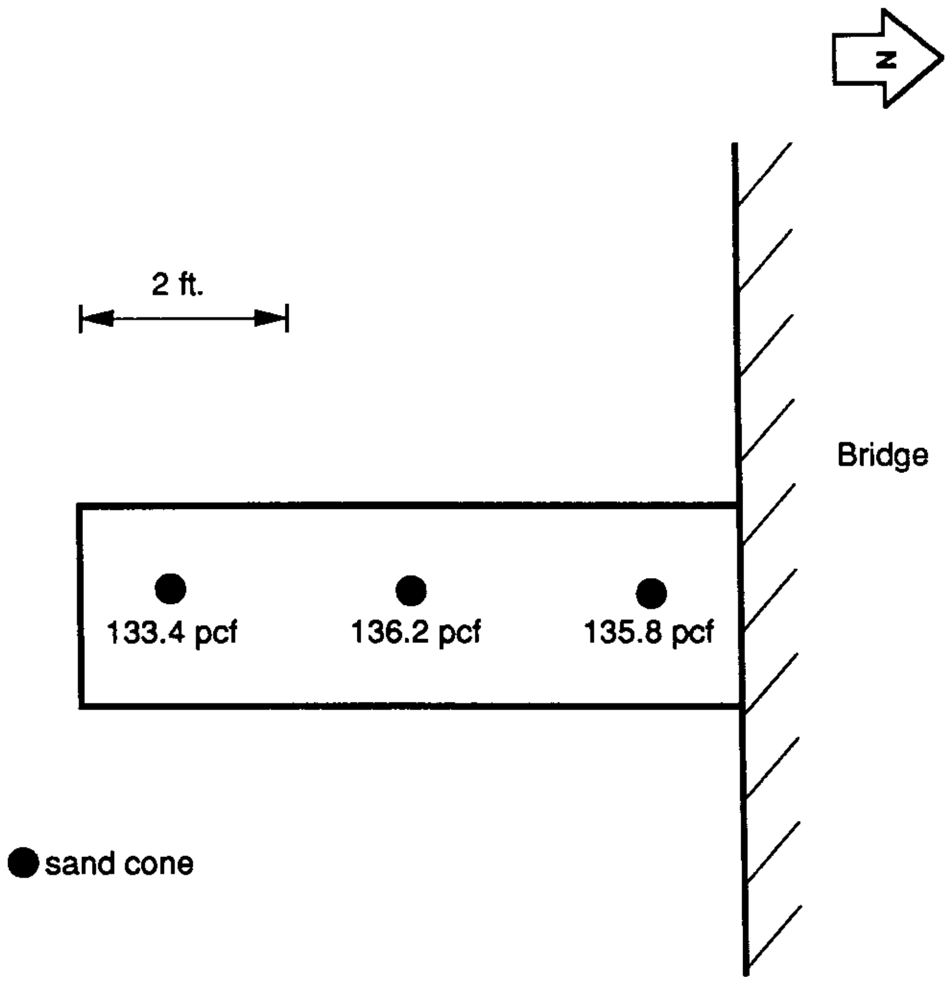


Figure 22. Dry Unit Weights for San Poil River Bridge (M.P.106) Fill Soils

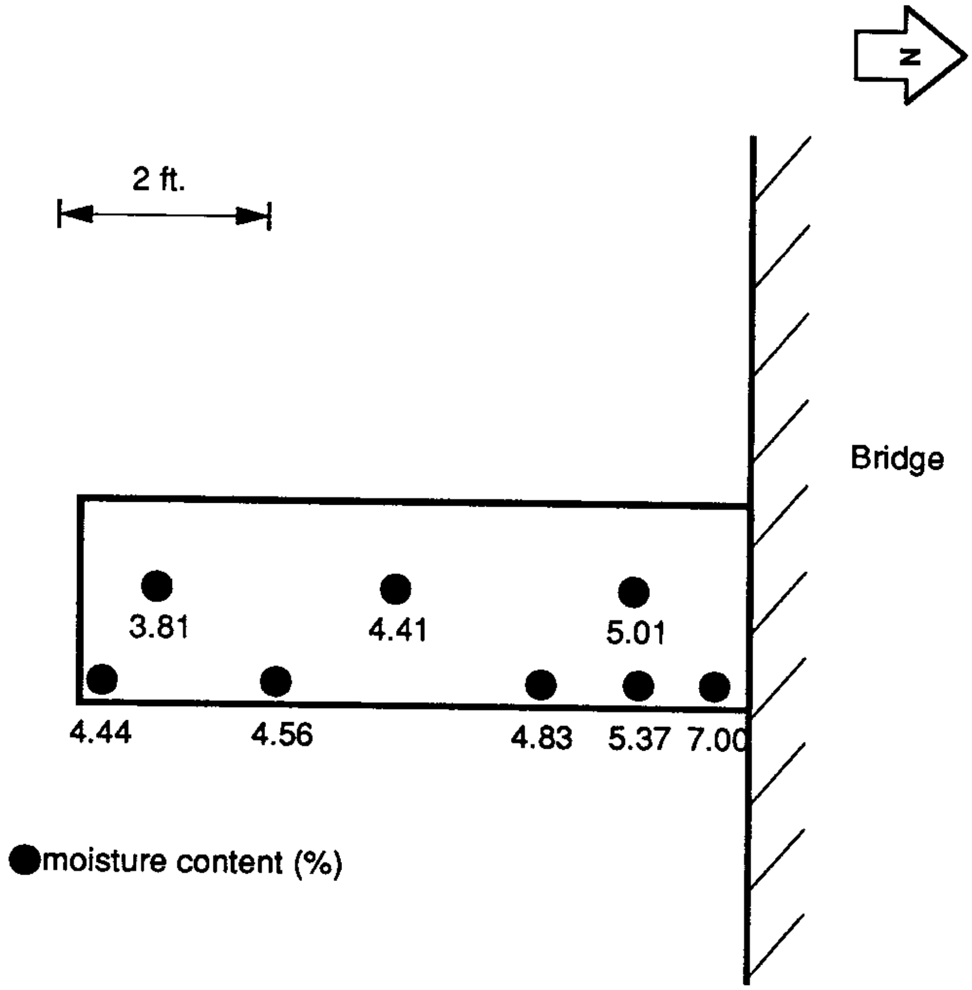


Figure 23. Moisture Content (in percent) of San Poil River Bridge (M.P. 106) Fill

infiltration. However, drainage and erosion problems did not appear to be significant for this structure.

3. SAN POIL RIVER BRIDGE AT MILEPOST 123

Location

The San Poil River Bridge at milepost 123 is approximately 41 miles north of the Lincoln County line and 13 miles south of Republic, Washington, on SR 21. The bridge (Number 21/322) carries two-way traffic over the San Poil River near the Colville Indian Reservation-U.S. Forest boundary. This bridge was selected for investigation on the same basis as the previous structure: it was constructed with no approach slabs and low embankment fills, and has required continued pavement maintenance. The test section was in the southbound lane, where pavement cracking at the bridge ends indicated approach distress.

File Search

No files were available for this structure from any of the state engineering record offices.

History

The bridge was constructed in 1931. No construction or maintenance records were available for this bridge. A maintenance record may be inferred by the number of BST applications that have been applied over a section of road; none of them appear to be recent for this bridge.

Soils, Foundation, and Structure

This bridge is situated in a narrow, steep-walled river valley. Foundation soil conditions are most likely characteristic of alluvial deposits. The highway follows the river through the center of the valley at this bridge site. Lower lying, swampy areas exist to the east of the bridge. These swampy areas are probably traps for organic and other fine grained soils.

The bridge is a 43-ft. single-span, concrete T-beam structure with full-height abutments. The bridge does not have approach slabs and is skewed 39 degrees in relation to the highway alignment. The foundation support for the full-height abutments is unknown. The approach embankments are approximately 6 to 7 ft. high and have safety berms on their sideslopes. The safety berms were not part of the original construction and are composed of a native, volcanic rock fill.

Approach Distress

Approach distress was observed as pavement cracks, approximately 1/8" to 1/4" wide, that outlined an area corresponding to the bridge deck, which was hidden under the BST. Bridge approach distress was also noted as slight rutting in the approach pavement area. A drive across this bridge produced only minor, but noticeable bumps at the bridge ends.

Site conditions

The field investigation of the southbound lane was conducted on the afternoon of Wednesday, September 12, 1990, under clear skies. A state maintenance crew of two men controlled traffic by closing the southbound lane. The state crew also recompacted and patched the test section after the field investigation had been completed. One test section was removed in the southbound lane on the entrance side of the bridge in the wheel path closest to the shoulder at the location shown in Figure 24.

Observations

The BST pavement was cut by the state crew members with a pneumatic hammer and removed easily. At approximately 17 inches from the edge of the abutment — measured parallel to the alignment of the road — the pavement thickened from 3-1/2 inches to 6-1/2 inches. The variation in pavement thickness can be clearly seen in Figure 25. As the excavation continued, a very unusual subgrade soil profile was observed in the test section, as shown in Figure 26. Under the pavement was a firm, 7-inch layer of basecourse underlain by rockfill with cobbles ranging from 4 to 8 inches, mixed into a

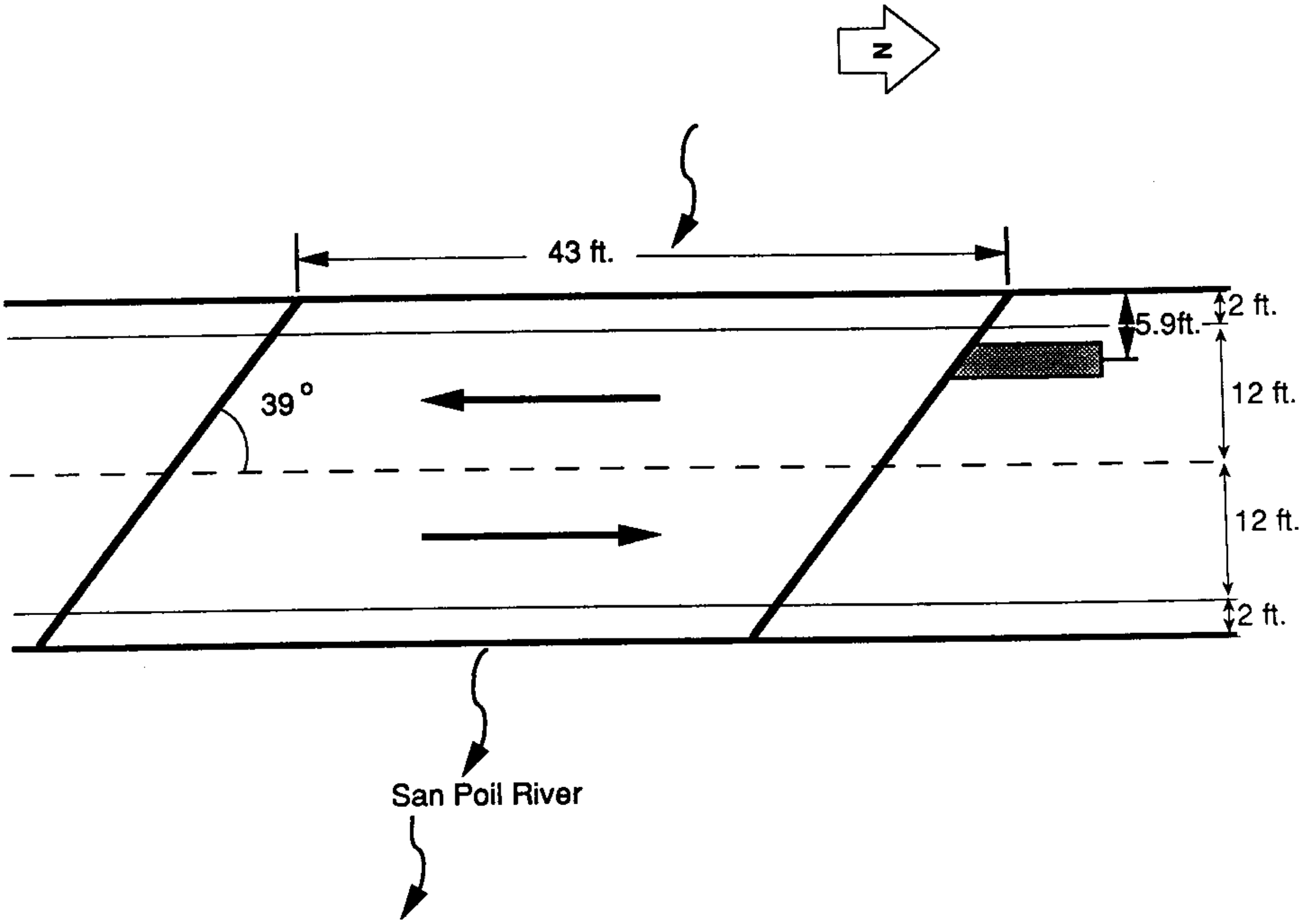


Figure 24. Location of Test Section in Southbound Lane of San Poil River Bridge M.P. 123

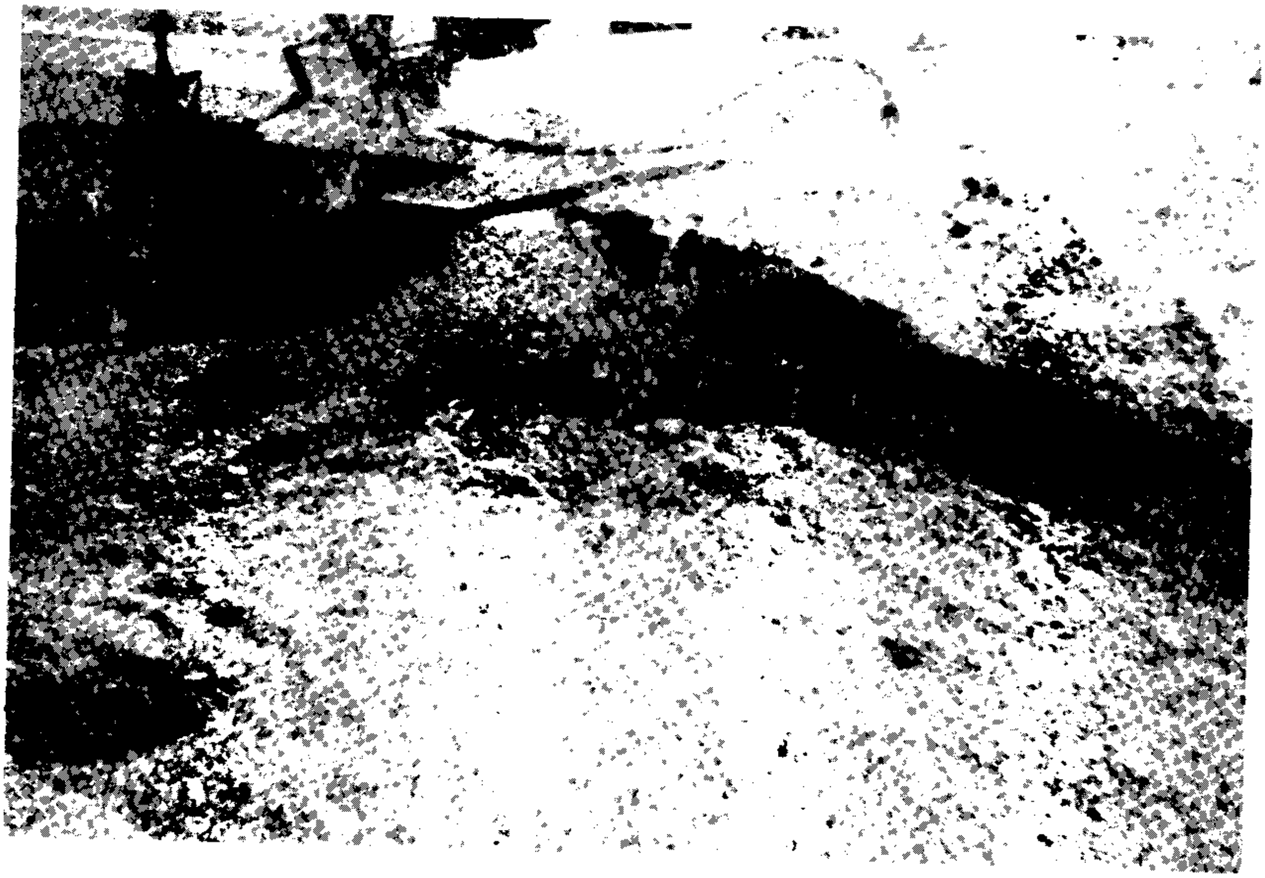


Figure 25. Photograph Showing Pavement Variation at the San Poil River Bridge (M.P. 123)

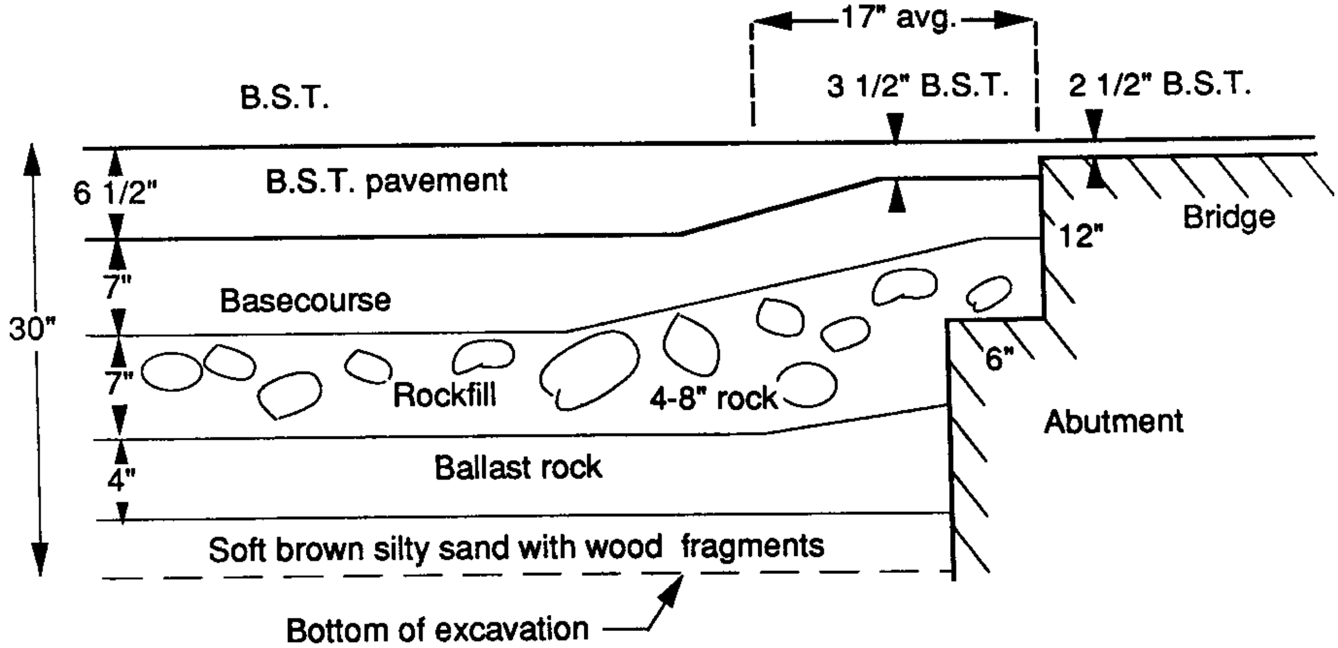


Figure 26. Approach Cross Section Parallel to Highway Alignment at Excavated Test Sections, San Poil River Bridge M.P. 123.

4-inch layer of ballast rock. This rockfill-ballast layer was jumbled and did not appear to be particularly dense, since it was easily removed with a pick and shovel. Below this layer was a dark brown, silty sand with some wood fragments. A pavement seat was encountered at 1 foot below the bridge deck elevation, but its face was vertical to the bottom of the 29-inch-deep excavation.

Test Results

1. Gradation. Grain size samples were obtained from the basecourse layer at distances of 12 and 60 inches from the abutment backwall. The resulting grain size distributions are shown in Figure 27. The grain size distributions indicated that the basecourse was finer-grained closer to the abutment, but the differences did not appear significant enough to cause the observed approach settlement. The percentages of fines and the D_{30} values are shown in Figure 28. For this test section, the average D_{30} was 1.8 mm and the average percentage of fines was 7 percent.

2. Densities. Densities of the basecourse were measured at the locations shown in Figure 29. The densities appeared to be slightly lower closer to the abutment. The lower densities near the abutment may have been due, in part, to soil disturbance from settlement over the pavement seat. The dry unit weight of the basecourse averaged 134 pcf, while modified proctor test results indicated a maximum dry density of 138 pcf at an optimum moisture content of 8.8. On that basis, relative compactions of the basecourse were estimated to be approximately 97 percent.

3. Moisture Content. Moisture content samples were taken at several locations in the test section in both the basecourse and the native fill. The results of the moisture content tests on samples from the upper 2 inches of the basecourse indicated that moisture contents were slightly higher in the middle of the test section, as shown in Figure 30. Moisture contents were lower and also more consistent at this site than for similar basecourses at other sites. The subgrade moisture contents varied across the test section, with a maximum difference of almost 3 percent, as shown in Figure 31.

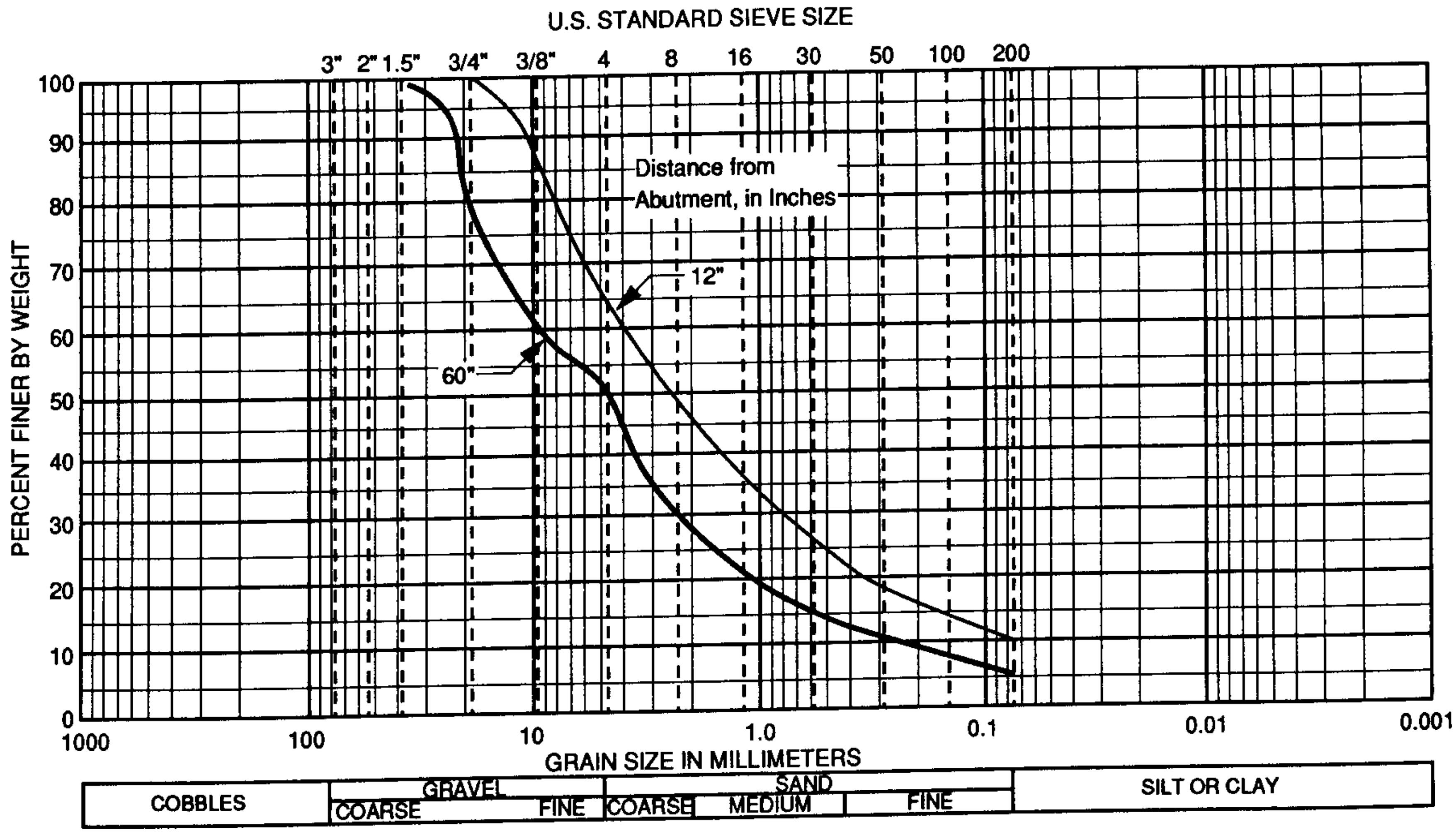


Figure 27. Grain Size Distribution of San Poil River (M.P. 123) Basecourse

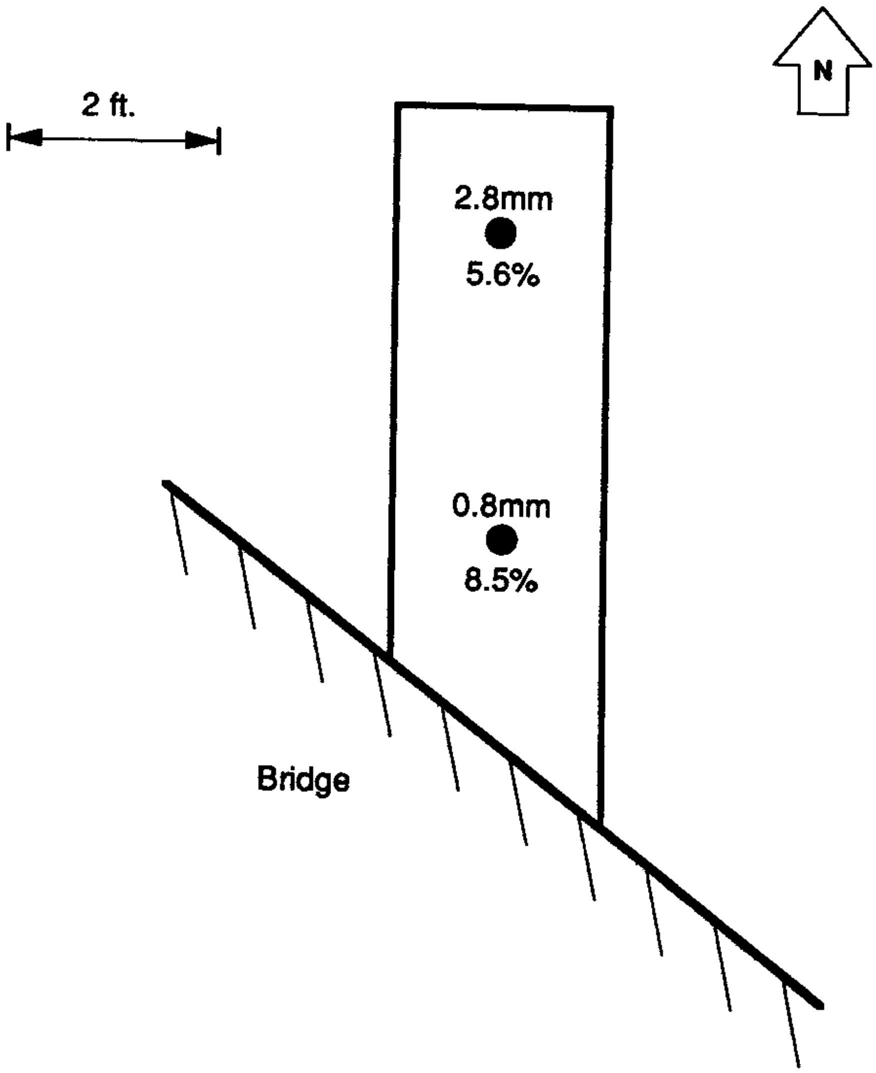


Figure 28. Percent Fines and D_{30} of San Poil River Bridge (M.P. 123) Basecourse.

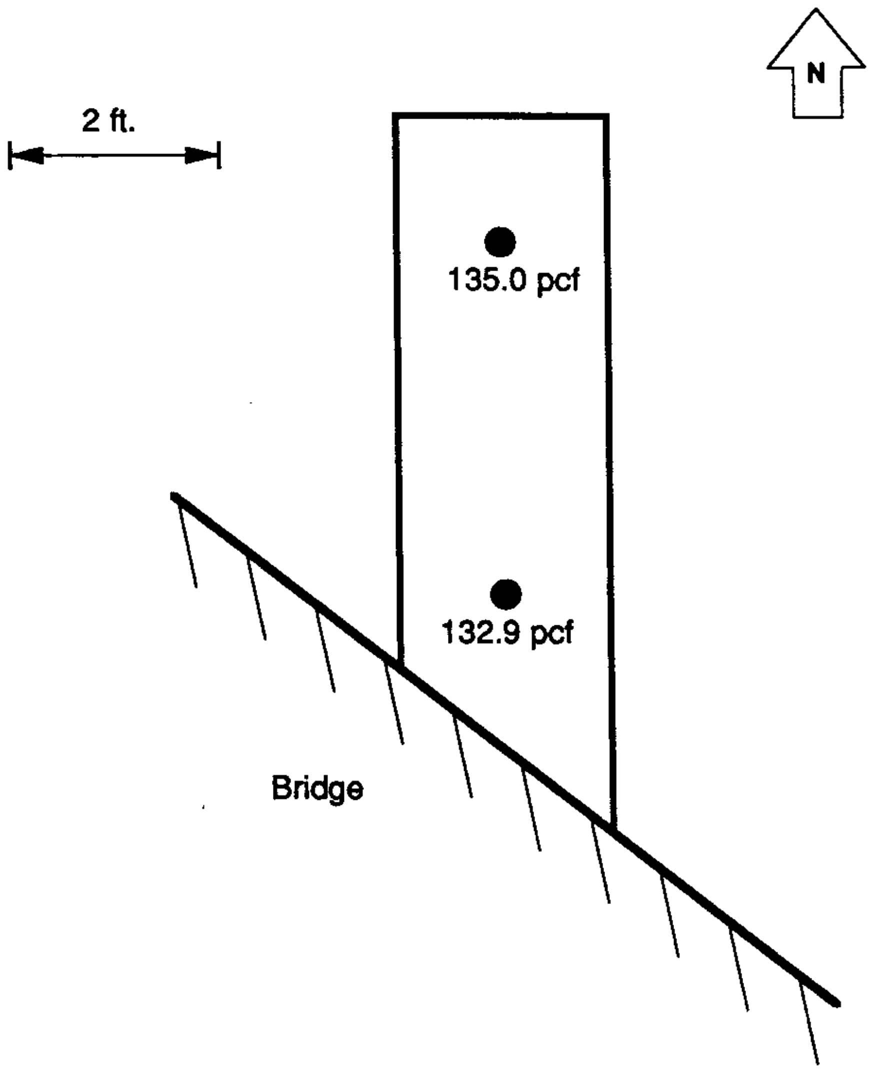


Figure 29. Dry Unit Weights for San Poil River Bridge (M.P. 123) Basecourse.

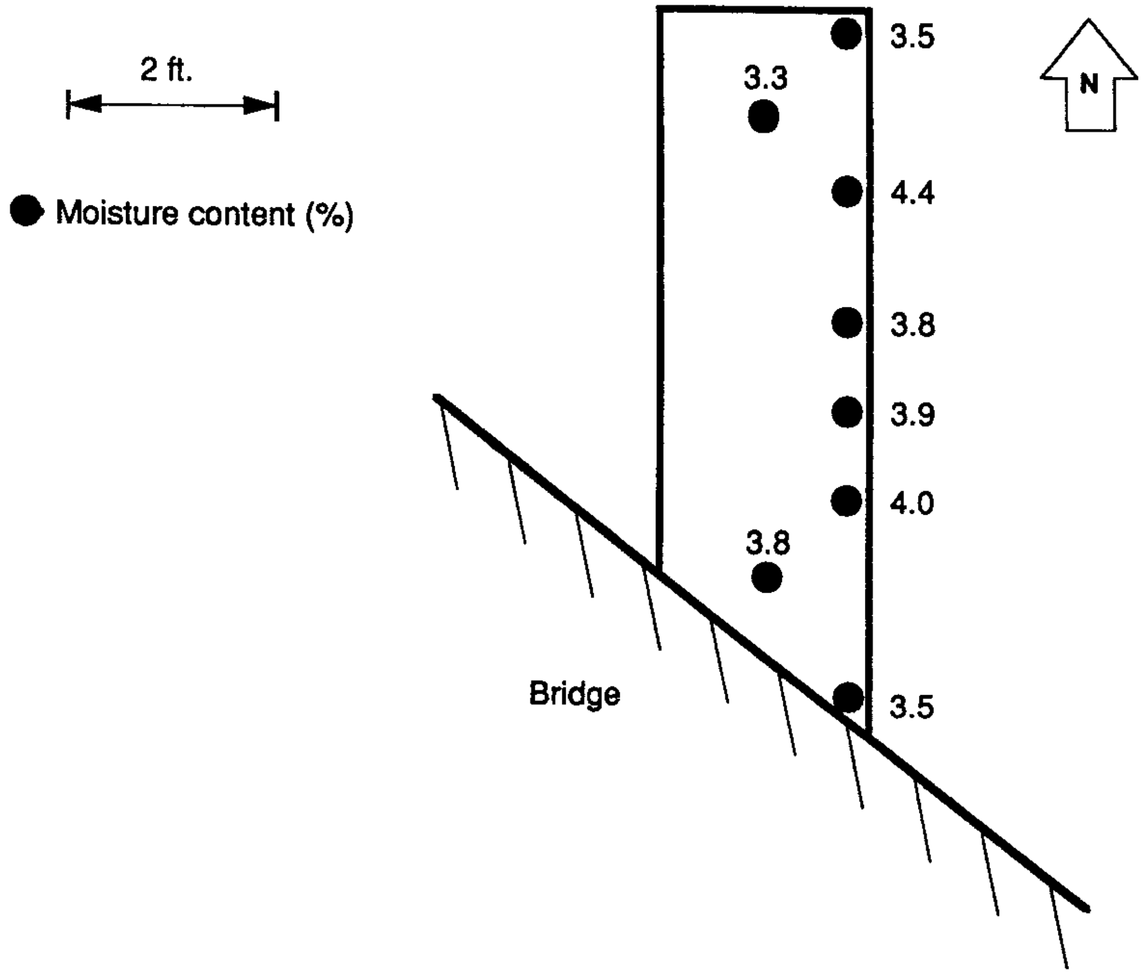


Figure 30. Moisture Content (in percent) of San Poil River Bridge (M.P. 123) Basecourse

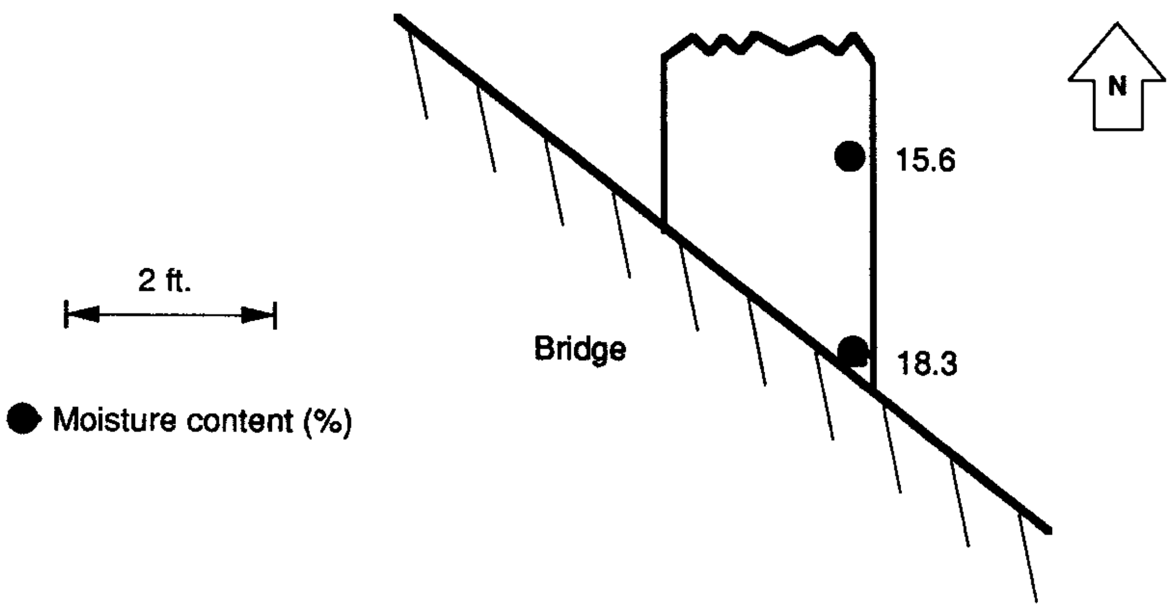


Figure 31. Moisture Content (in percent) of San Poil River Bridge (M.P. 123) Subgrade Soils

Conclusions

Approach distress on this bridge appears to have resulted from differential settlement between the bridge deck and the approach embankment. The approach pavement thickness suggested the amount of differential settlement that had occurred. Investigation of the previous structure at milepost 106 showed similar behavior, that is, backfill supported by the pavement seat remained at grade while the rest of the embankment settled. On this structure, the zone of pavement seat support was wider because of the skew of the bridge and the larger-sized cobbles that rested on it. Sandcone test data indicated that the basecourse materials were at a satisfactory level of compaction, but were less dense over the pavement seat area. The moisture contents of the basecourse materials indicated that water problems had been minimal and had not had a significant effect on the differential settlement. Embankment erosion problems were not evident.

4. SAN POIL RIVER BRIDGE AT MILEPOST 124

Location

The San Poil River Bridge at milepost 124 is approximately 42 miles north of the Lincoln County line and 12 miles south of Republic, Washington, on SR 21. The bridge, which carries two-way traffic over the San Poil River in Ferry County, is a 61 ft. double span, concrete T-beam structure. The bridge does not have approach slabs and is not skewed with the alignment of the highway. This bridge was targeted for investigation, but damage was serious enough to warrant immediate repair by state maintenance forces. The bridge is in the same age group and design as the two previous bridges.

This bridge was repaired by state maintenance crews two weeks before the investigation of other bridges on SR 21. High river levels from spring runoff eroded the upstream face of the approach embankment and created a large void that undermined the approach pavements. Repair work included constructing a lagging wall at the abutment, refilling and compacting with granular backfill, and placing rip-rap on the slope beneath the bridge. Compaction of the granular backfill in the excavation behind the bridge is shown in Figure 32. The road surface was subsequently patched.

5. CURLEW CREEK BRIDGE

Location

The Curlew Creek Bridge carries two-way SR 21 traffic over Curlew Creek at milepost 141 in Ferry County. The bridge, roughly 5 miles north of Republic, Washington, was selected for investigation because it is a relatively new structure that is already experiencing approach problems.

File Search

A file search was undertaken to review documents pertaining to the Curlew Creek bridge. Since the bridge was constructed 4 years ago, pertinent information regarding field construction and testing was available. Files reviewed at the Materials Laboratory in Tumwater contained bore hole information, foundation recommendations, and allowable bearing capacity calculations. The files office in Olympia provided plan sheets, specifications, and "as constructed" plan sheets. The District 2 materials laboratory in Wenatchee had construction records in their files.

History

The Curlew Creek Bridge was constructed in 1986 to replace an older structure. The newer bridge is located along the same alignment as the older structure. Only minor earthwork fills were needed to construct the approach embankments; the existing south approach was very close to grade. The design for this bridge, which was contained in the short-span bridge program, is standard. However, a foundation report was prepared to address any deviations from the standard foundation design and to provide estimates of foundation depths and capabilities. Maintenance records were not available for this structure.

Figure 32. Repair of Erosion Damage Behind Abutment of Bridge at M.P. 124

Soils, Foundation, Structure, and Pavement Section

SR 21 follows the Republic graben northward to the Canadian border. The general geology of the Republic graben is characterized by coarse granular terrace materials, silty sandy gravels with some silt from alluvial fan deposits, and rock outcrops. The field investigation for the design of the bridge included drilling two test holes to evaluate foundation support requirements. The drilling program also included 25 standard penetrometer tests taken at 5-foot intervals, as shown in Figure 33. The foundation soils at this site are typical of the terrace and alluvial fan deposits: 30 ft of medium dense silty sands and gravels with stream rounded cobbles and boulders in places, underlain by 11 ft of fine sandy silt. Below the medium dense soils are slightly silty to silty sands with lenses of silt, gravel, cobbles, and boulders. The water table is perched some 36 feet below the surface of Curlew Creek at this site.

The laboratory program consisted of the identification and classification of 25 disturbed samples obtained from the standard penetrometer tests conducted in the field. Grain-size analyses and moisture-content determination were performed on the disturbed samples.

The approach embankments required minor fills, which varied up to 5 ft. in height. The new fills were stable, with 1.75:1 or flatter slopes. The fills were anticipated to settle as much as 0.5 inches during construction, and post-construction settlements were expected to be negligible. Spread footings, designed for allowable loads of 3 tsf, were used at this site because of the thickness and density of the sand, gravel, cobbles, and boulder stratum. Rip-rap protection was placed to prevent scour of the spread footings.

The bridge is a single-span, precast, prestressed concrete slab structure 47 ft.long and is aligned normally in relation to the highway. The bridge is constructed with two 12-ft lanes and two 4-ft shoulders and does not have approach slabs. The design pavement section is shown in Figure 34.

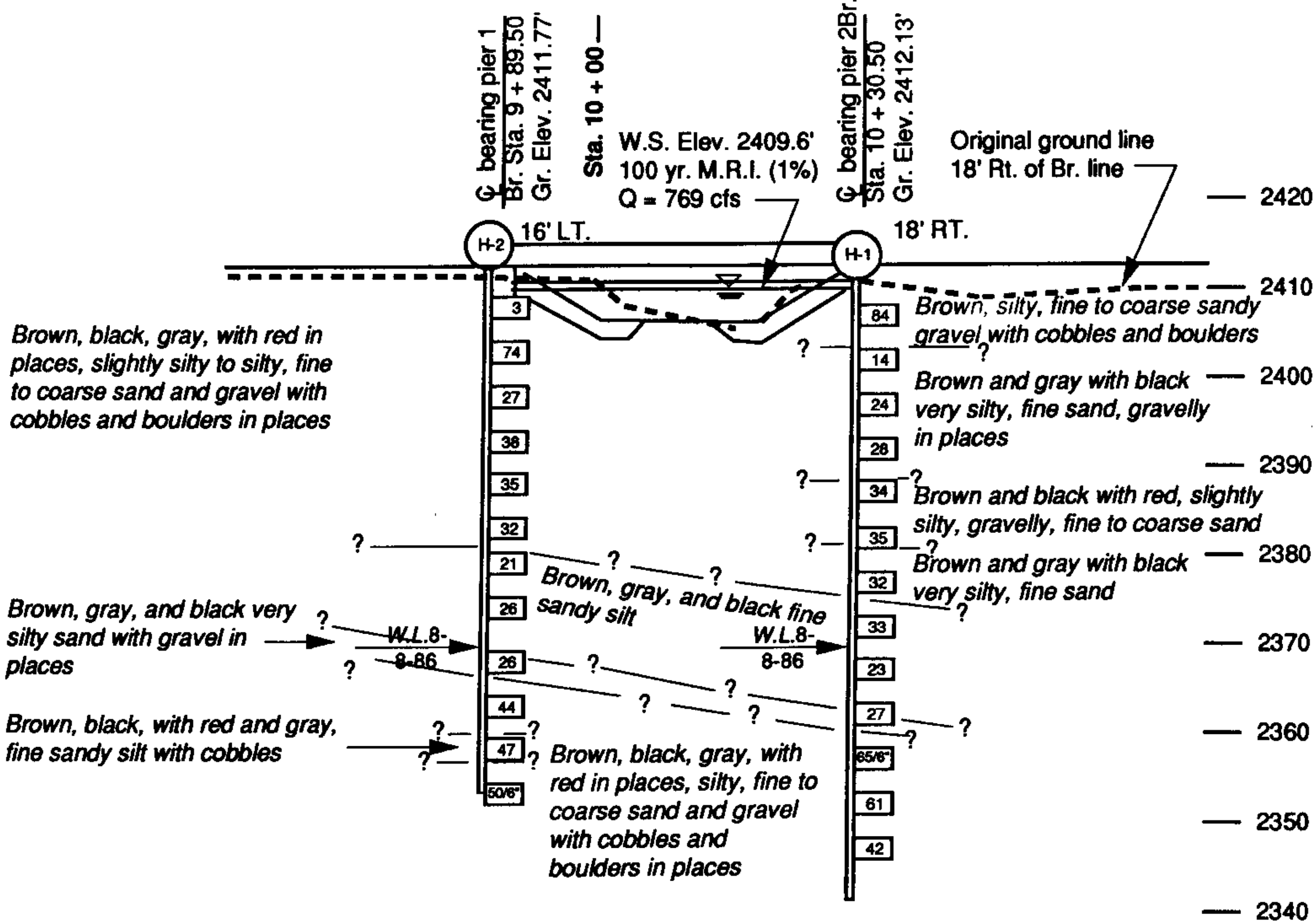


Figure 33. Subsurface Profile at Curlew Creek Bridge

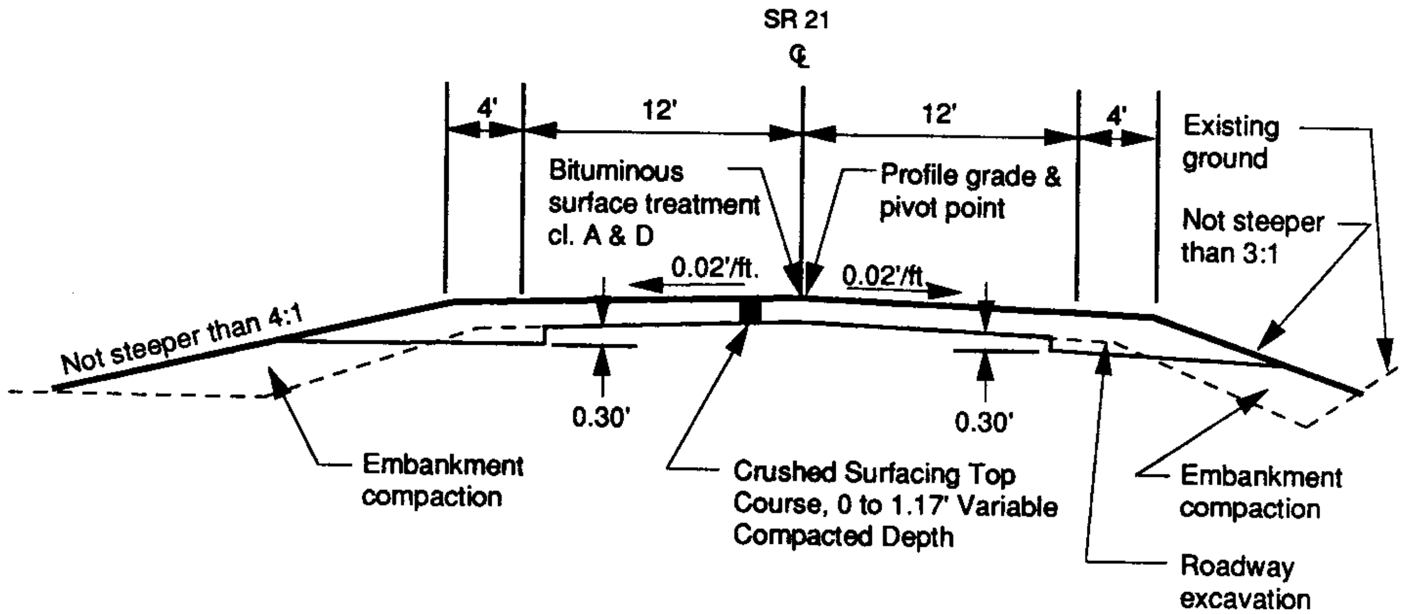


Figure 34. Roadway Cross-Section at Curlew Creek Bridge

Approach Distress

Approach distress was evident in the form of transverse cracking at both bridge ends. The cracking delineated the bridge deck under the BST. Isolated transverse cracking was also observed at approximately 20-25 ft. from the south end of the bridge, but it appeared to be unrelated to bridge distress.

Site Conditions

The field investigation of the northbound lane, on the entrance side, took place on the morning of Thursday, September 13, 1990, under sunny skies. A state maintenance crew of two men controlled traffic by closing the northbound lane. The state crew also recompacted and patched the test sections after the field investigation had been completed. One test section was investigated at the location shown in Figure 35. The test section was situated in the wheel path of the northbound lane closest to the shoulder on the entrance side of the structure.

Observations

The BST in the test section was removed with a pneumatic hammer. The BST was approximately 4 inches thick in a zone immediately adjacent to the abutment and about 1-1/2 inches thick over the rest of the test section. Removal of the pavement revealed a noticeable depression in the basecourse directly behind the abutment, as shown in Figures 36 and 37. The depression extended along the back of the abutment for the full width of the test section. The basecourse materials were very firm and free of voids and extended to an average depth of 28 1/2 inches. Below the basecourse, the natural foundation soils were soft, dark brown, silty sands. These soils were relatively soft and could be excavated with a shovel between large boulders (1-1/2 to 2 ft. in diameter).

The actual abutment was encountered at 22 1/2 inches below the bridge deck. The abutment design utilized to connect the precast concrete bridge beam sections to the abutment is shown in Figure 38. The concrete beams had been installed with gaps of 2 to 3 inches between them. A waterproofing membrane was placed on the deck before

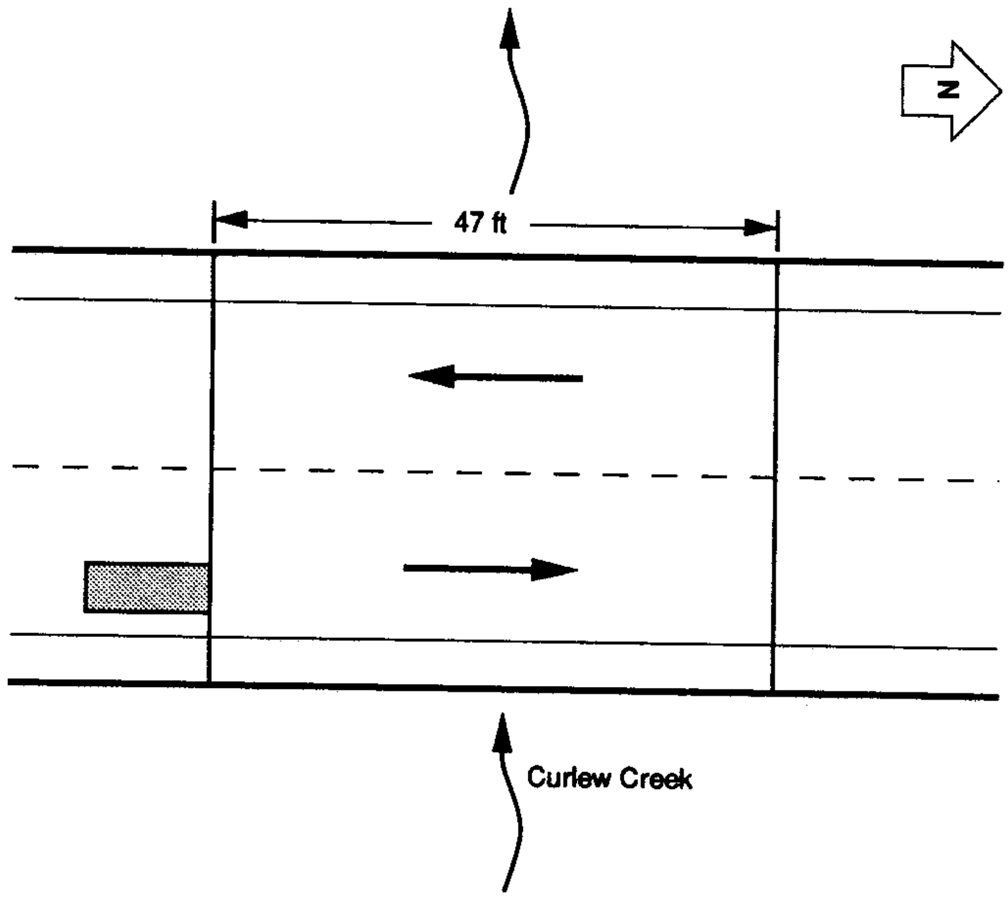


Figure 35. Location of Test Section in Northbound Lane of Curlew Creek Bridge (M.P. 140)

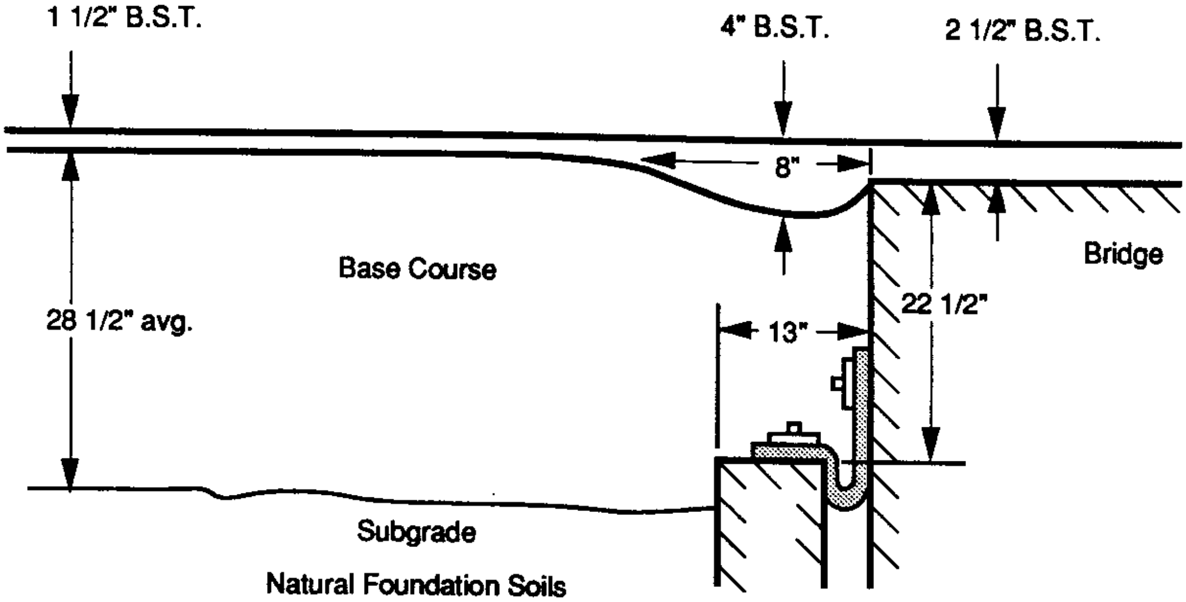


Figure 36. Profile through Curlew Creek Bridge Abutment

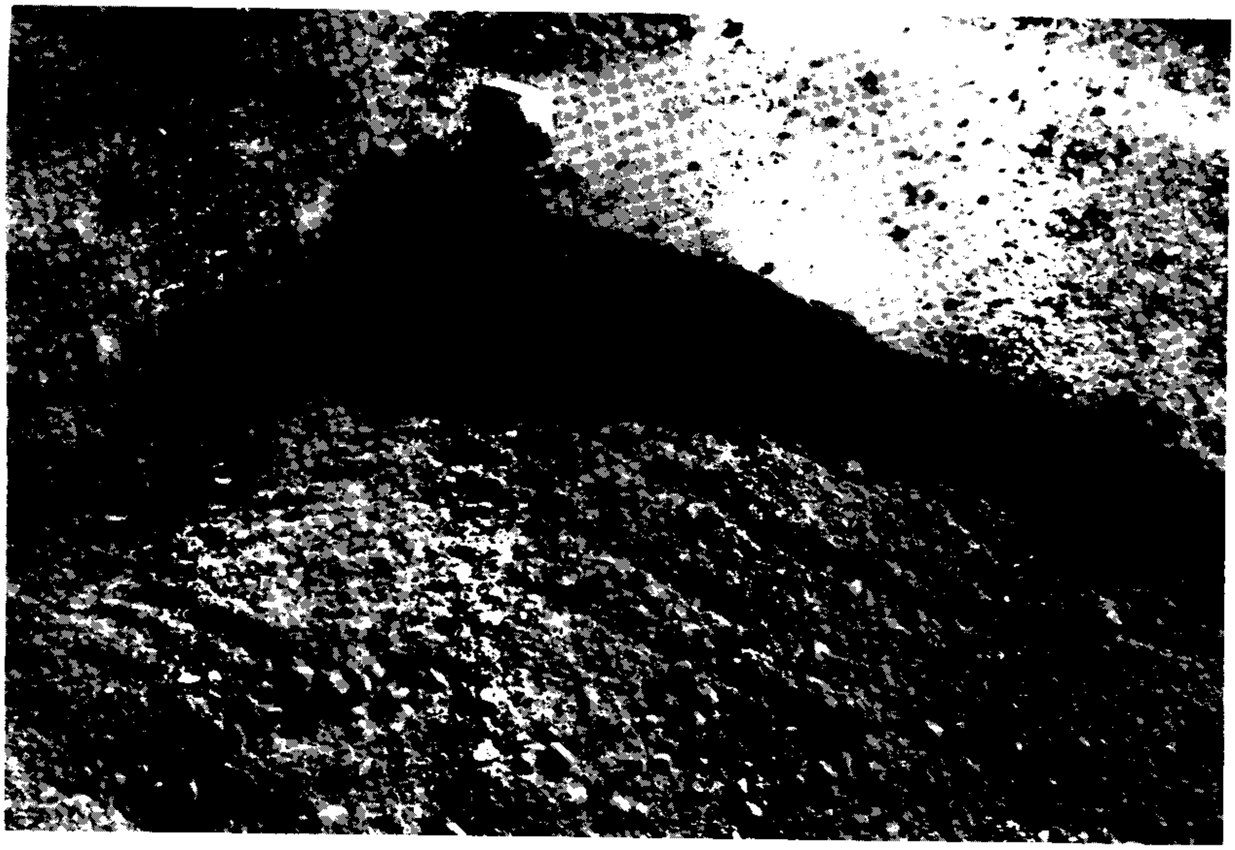
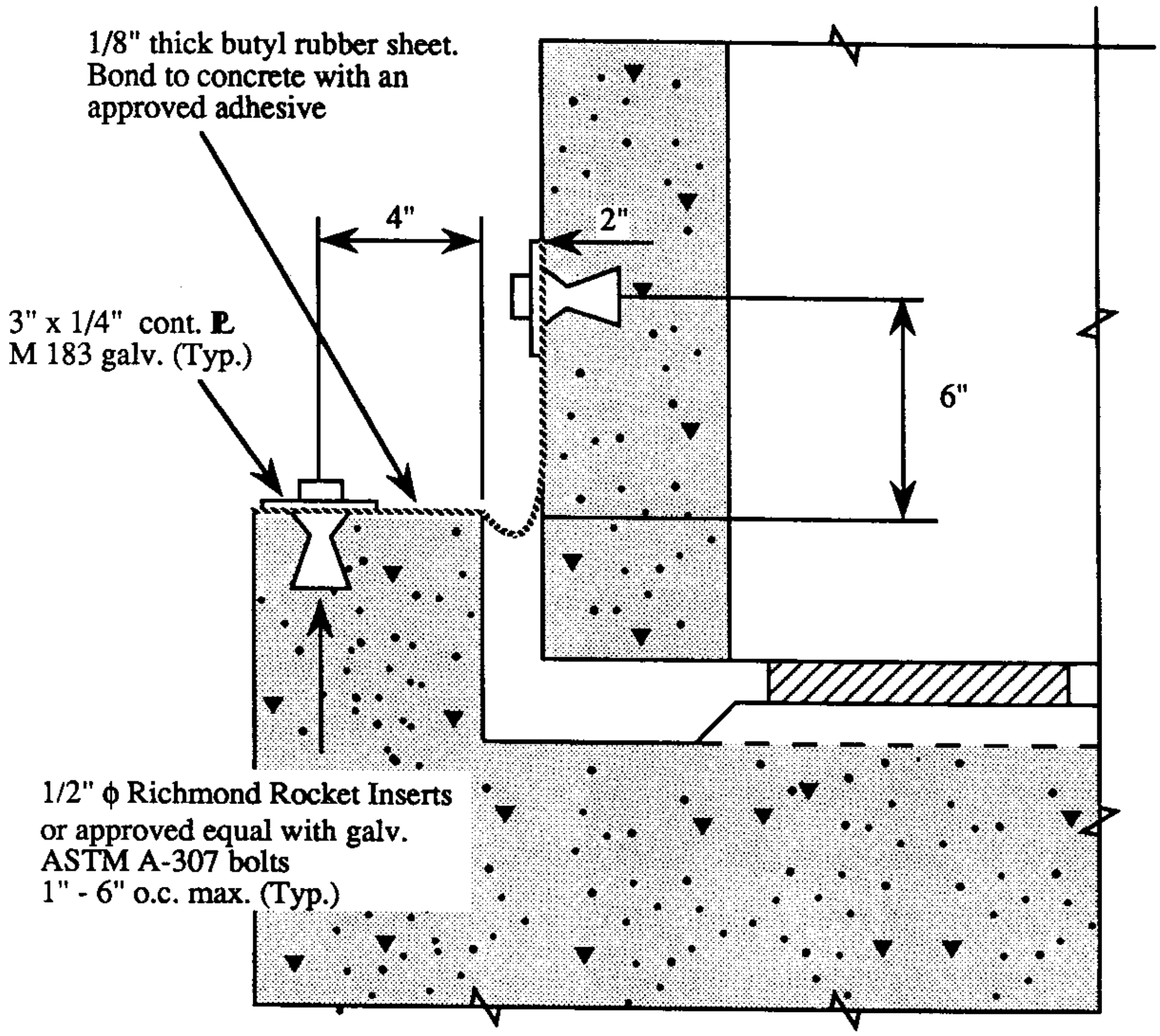


Figure 37. Depression in the Basecourse Adjacent to Abutment Backwall at Curlew Creek Bridge



Extend butyl rubber sheet full length along opening between superstructure and abutment and up side walls to seal out the backfill.

Figure 38. Section of Butyl Rubber Sheet on Curlew Creek Bridge Abutment

paving. A butyl rubber strip fit between the abutment and the precast girders and stretched the full length of the abutment to seal the backfill out. Apparently, there was some problem with the rubber seal, since backfill was observed on the underside of the bridge sitting in small piles on the abutment below the gaps between adjacent beams, as shown in Figure 39. The amount of material that had migrated out was indicated by the piles of soil and the water marks on the abutment under the the bridge. The small piles contained rock fragments from the basecourse, but consisted predominantly of fines. The rubber seals and fasteners connecting the the precast girders were intact and in good shape in the test pit.

Test Results

1. Gradation. Grain size samples were obtained from the basecourse and the subgrade soils. Grain size distributions are shown for basecourse samples taken at 8, 36, and 60 inches from the abutment backwall in Figure 40. The grain size distributions indicated that the basecourse would meet WSDOT specifications. The percentages of fines and the D_{30} values are shown in Figure 41. For this test section, the average D_{30} was 0.52 mm and the average percentage of fines was 9.1 percent. The basecourse samples were consistent, except in the fine gravel to coarse sand range of particle size.

Samples of the subgrade soils were taken 26 and 45 inches from the abutment backwall (Figure 42). At these locations, the D_{30} averaged 0.19 mm and the percentage of fines averaged 15.5 percent. The grain size distribution curves for the subgrade soils were not consistent because of the variation in the material subgrade fill, as shown in Figure 43.

2. Densities. Basecourse densities were measured at the locations shown in Figure 44. The densities appeared to be higher near the abutment. The dry unit weight of the basecourse averaged 127.3 pcf., while modified proctor test results indicated a maximum dry density of 138.5 pcf at an optimum moisture content of 7.5 percent. With this maximum dry density, relative compactions of the basecourse averaged 92 percent. Densities were also measured for the native subgrade soil, as shown in Figure 45.

Figure 39. Basecourse Material which has Migrated through Joints at Curlew Creek Bridge

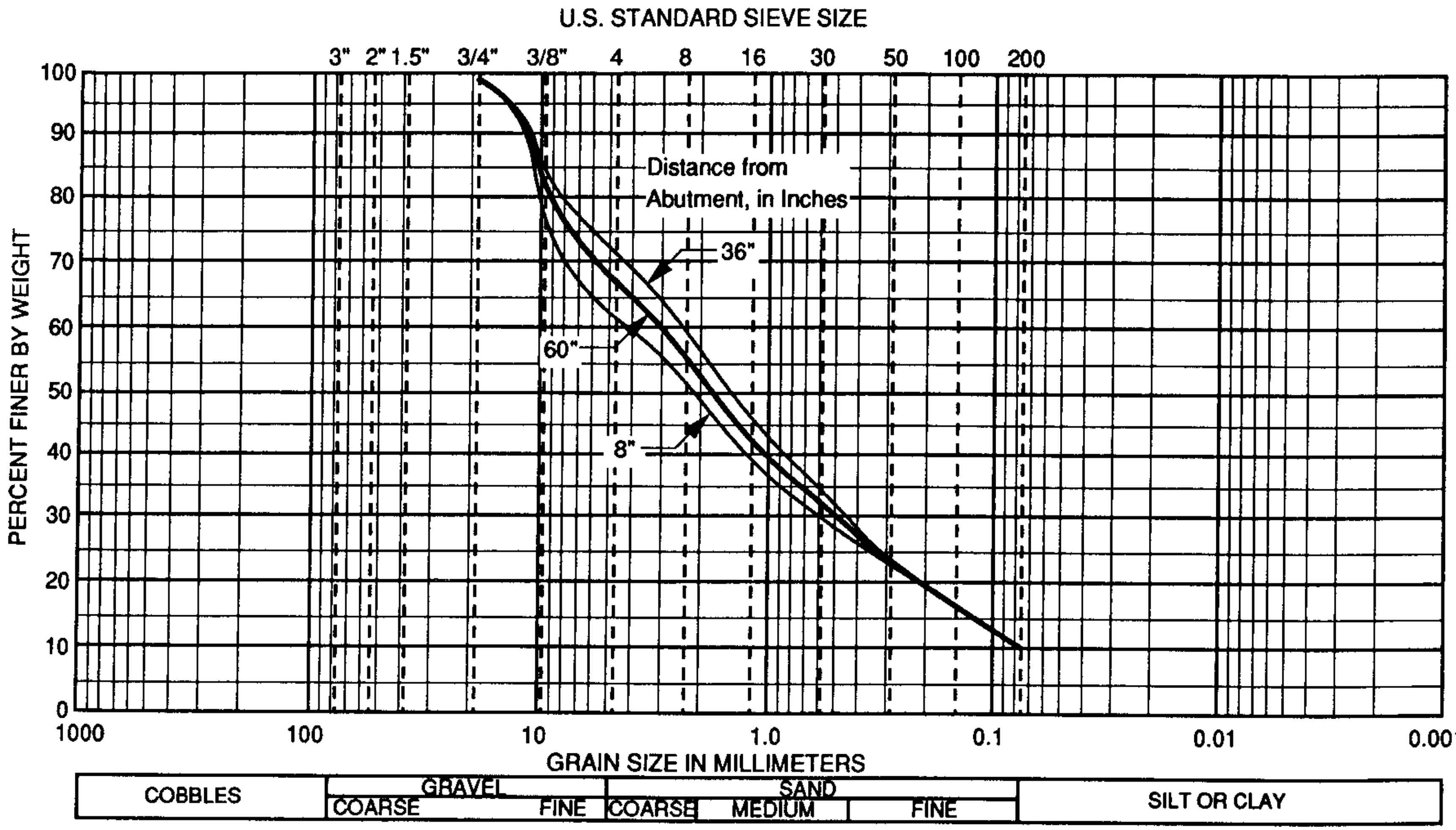


Figure 40. Grain Size Distribution of Curlew Creek Basecourse

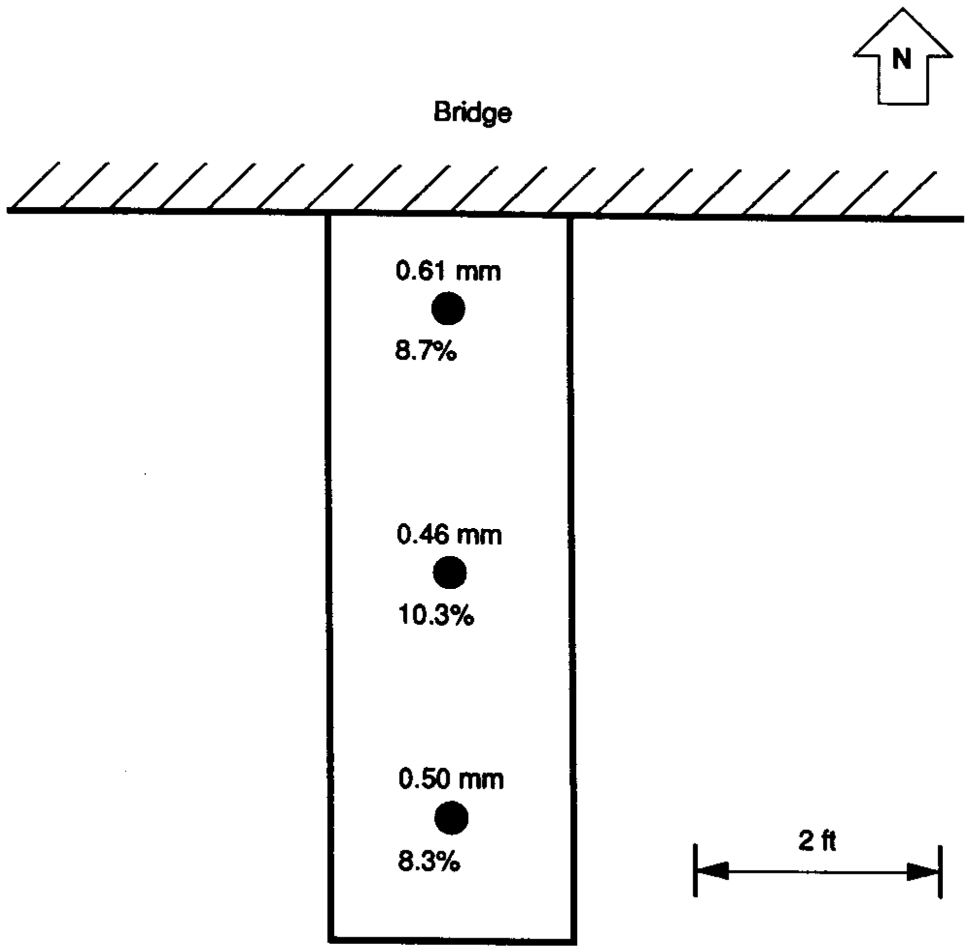


Figure 41. Percent Fines and D_{30} in Curlew Creek Bridge Basecourse

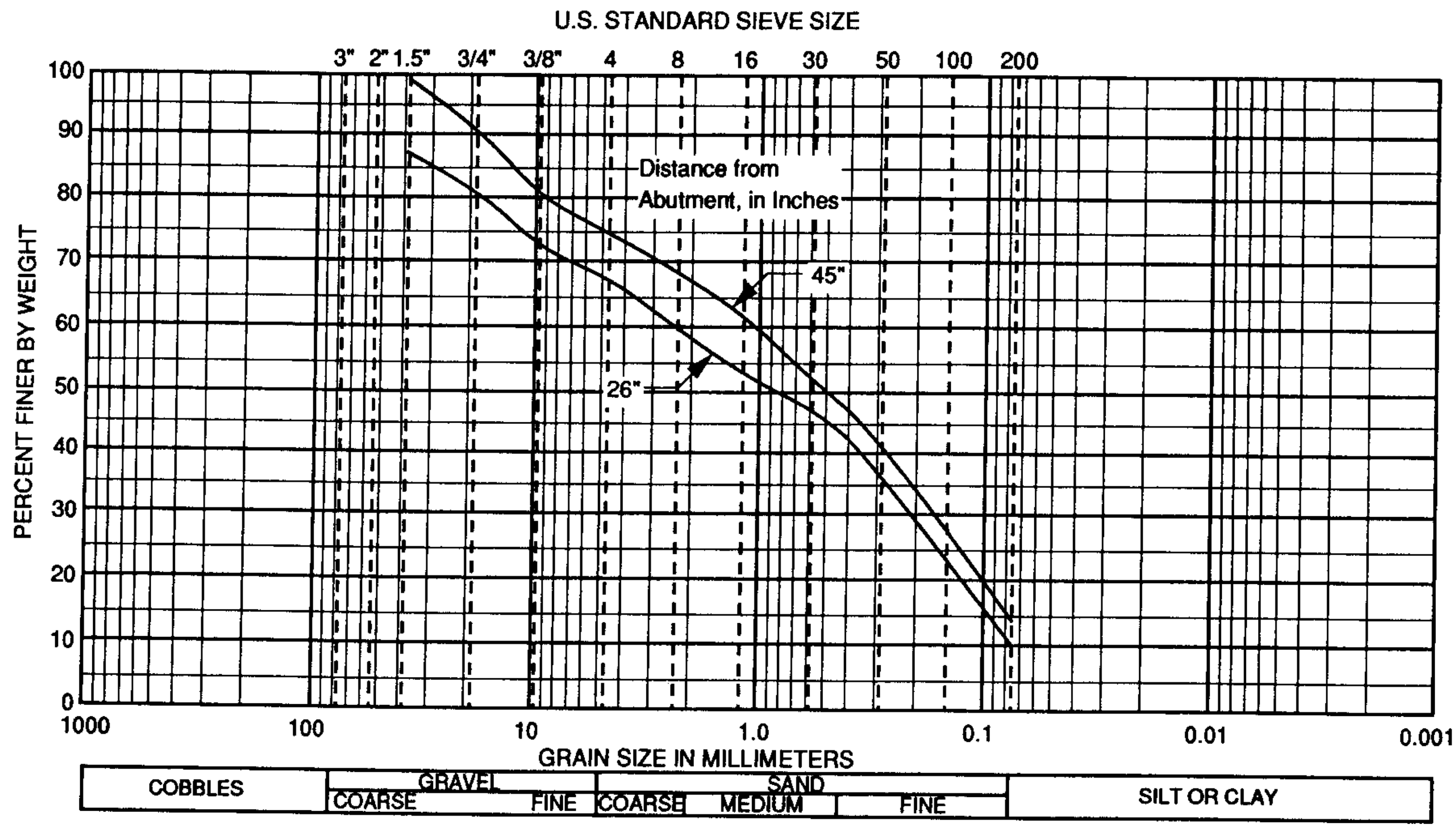


Figure 42. Grain Size Distribution of Curlew Creek Subgrade

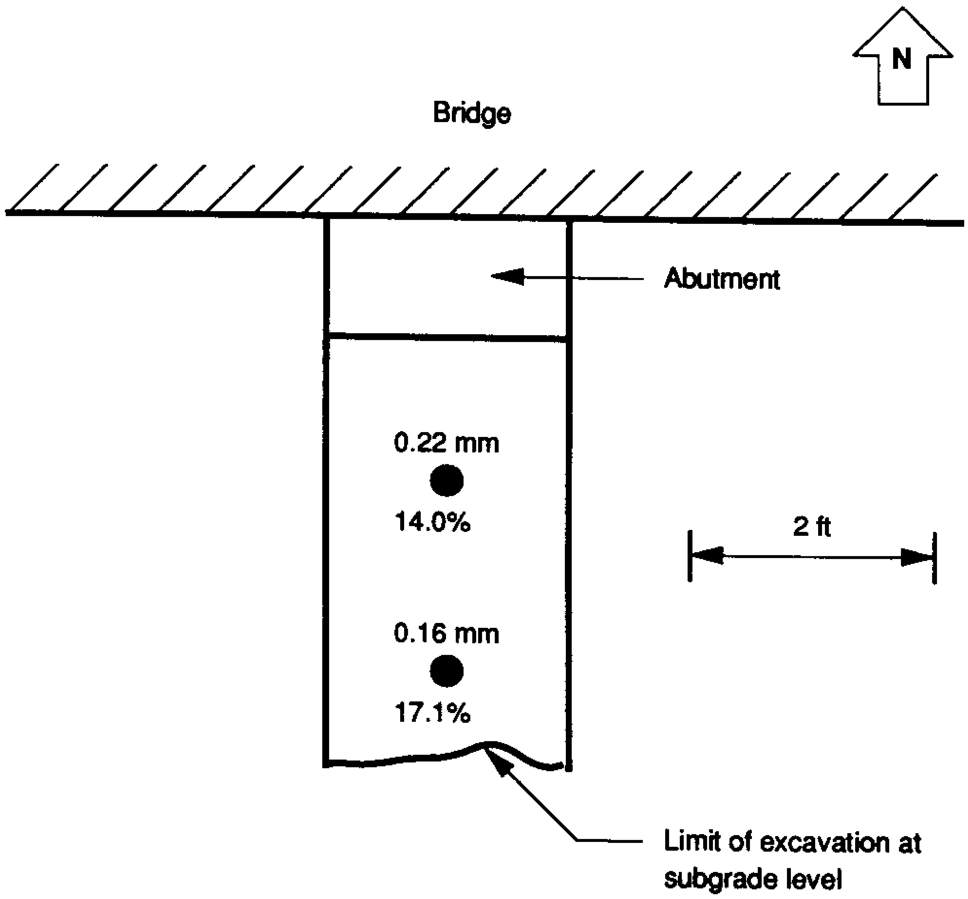


Figure 43. Percent Fines and D_{30} in Curlew Creek Bridge Subgrade

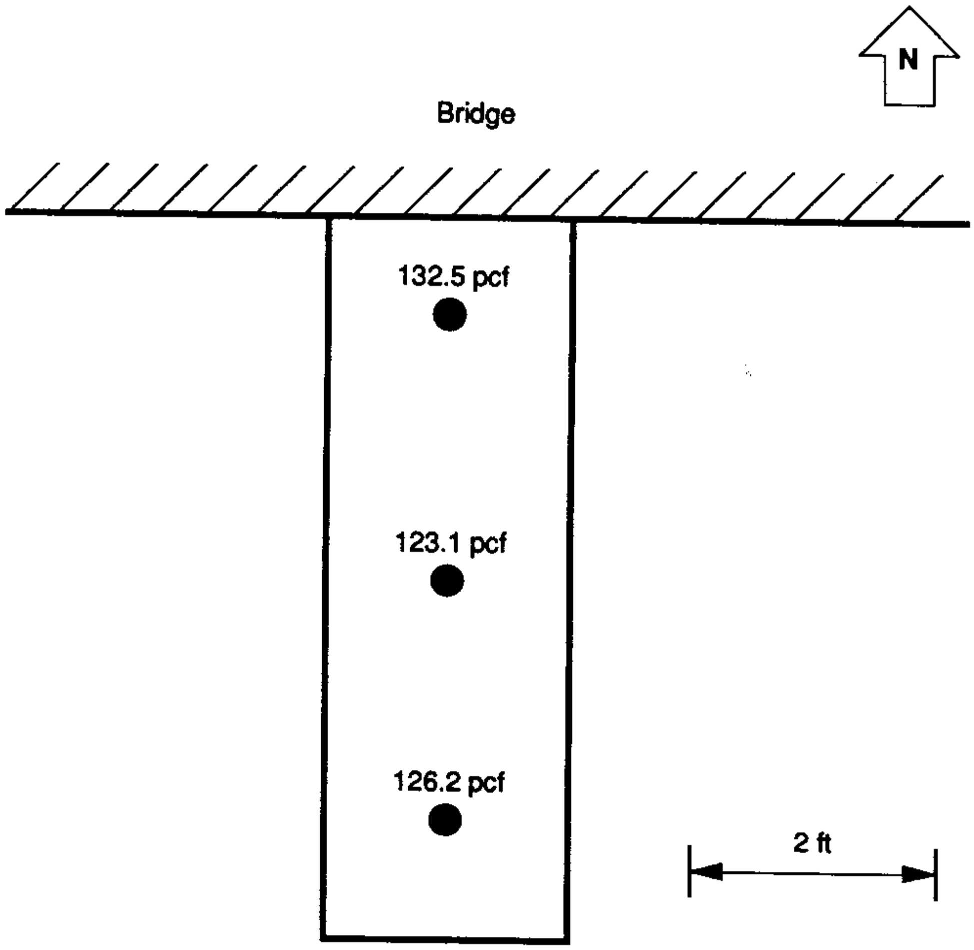


Figure 44. Dry Unit Weight for Curlew Creek Bridge Basecourse

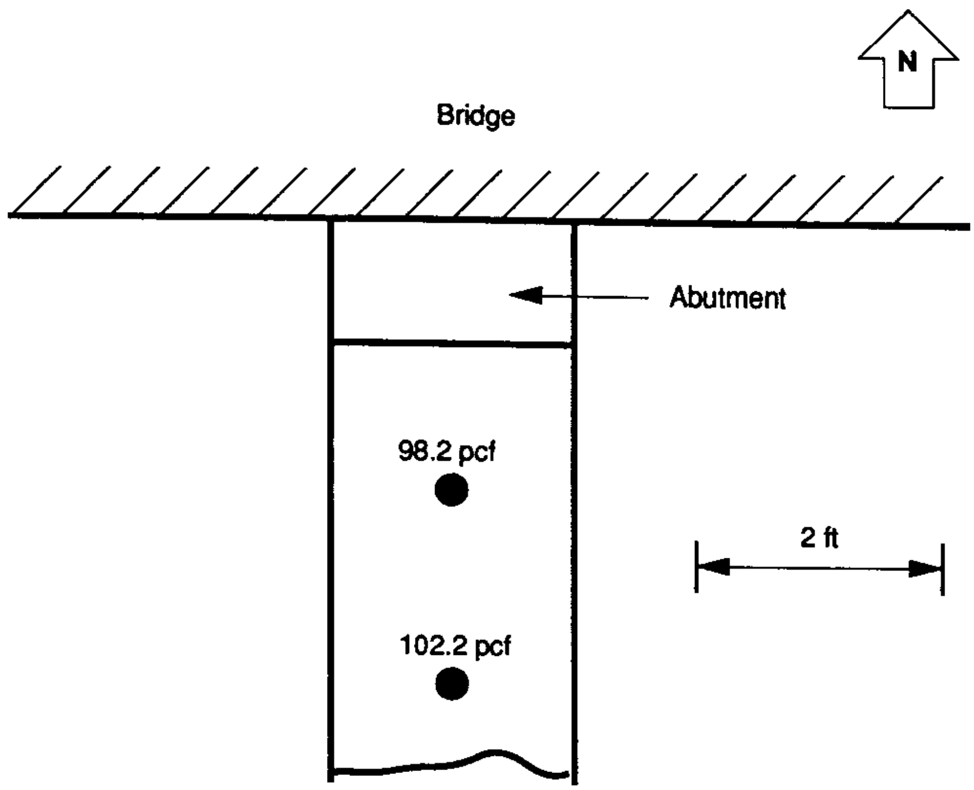


Figure 45. Dry Unit Weight for Curlew Creek Bridge Subgrade

Densities averaged 100.2 pcf and were lower against the abutment. Modified proctor test results indicated a maximum dry density of 129.5 pcf at an optimum moisture content of 8.8 for the subgrade soils. Relative compaction of the native subgrade fill averaged 77 percent.

3. Moisture Content. Moisture content samples were taken at several locations in the test section for both the basecourse and the subgrade. The results of the moisture content tests on samples from the upper 2 inches of the basecourse are shown in Figure 46. Moisture contents of the basecourse averaged 4.0 percent. Moisture contents averaging 12.3 percent for the subgrade soils are shown in Figure 47. In both cases, moisture contents gradually increased closer to the abutment.

Conclusions

The approach distress at this structure is most likely caused by localized settlement. The depression in front of the abutment could have been caused by a couple of mechanisms. One possible explanation is that the membrane that seals the ends of the precast sections is allowing backfill through the space between the sections. Though the excavated test section did not show any such leakage, other locations along the abutment may have leaked, especially if the membrane was damaged during construction. The second cause of localized approach problems may be that the basecourse was not compacted sufficiently on top of the connectors at the bridge segments for fear of damaging the rubber connector. Densities measured in the field attest to slightly lower than acceptable relative compactions. This second hypothesis logically explains the continuous depression along the end of the bridge. Erosion around the ends of the bridge did not appear to be a problem during the investigation but will probably become more significant within the next few years.

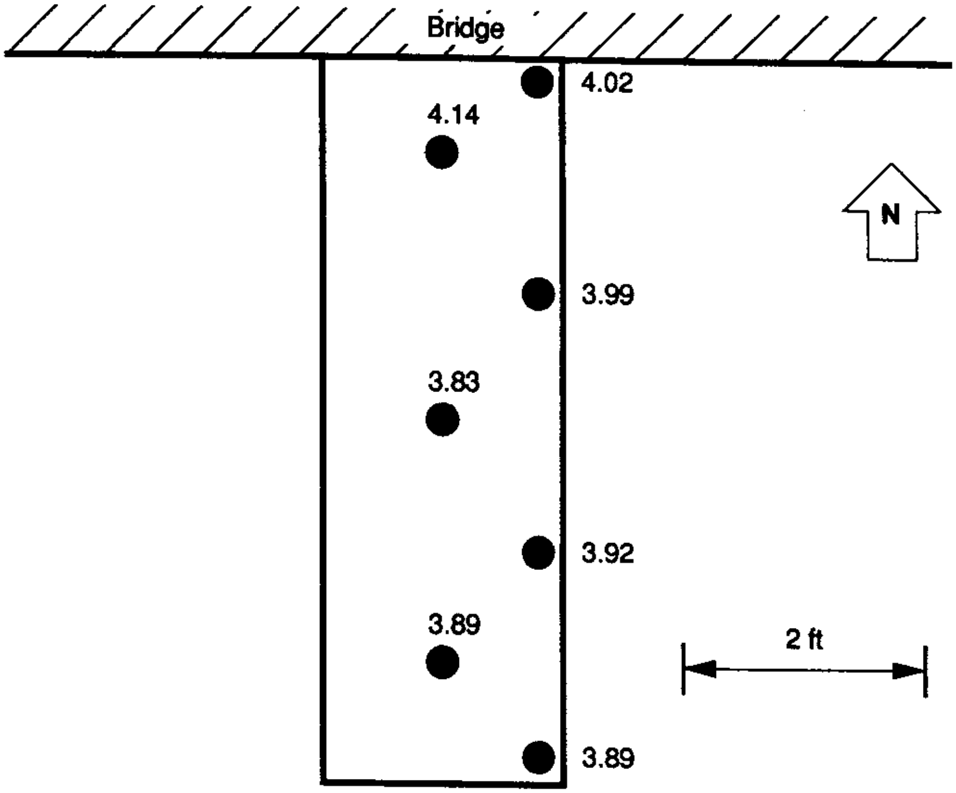


Figure 46. Moisture Content (in percent) of Curlew Creek Bridge Basecourse

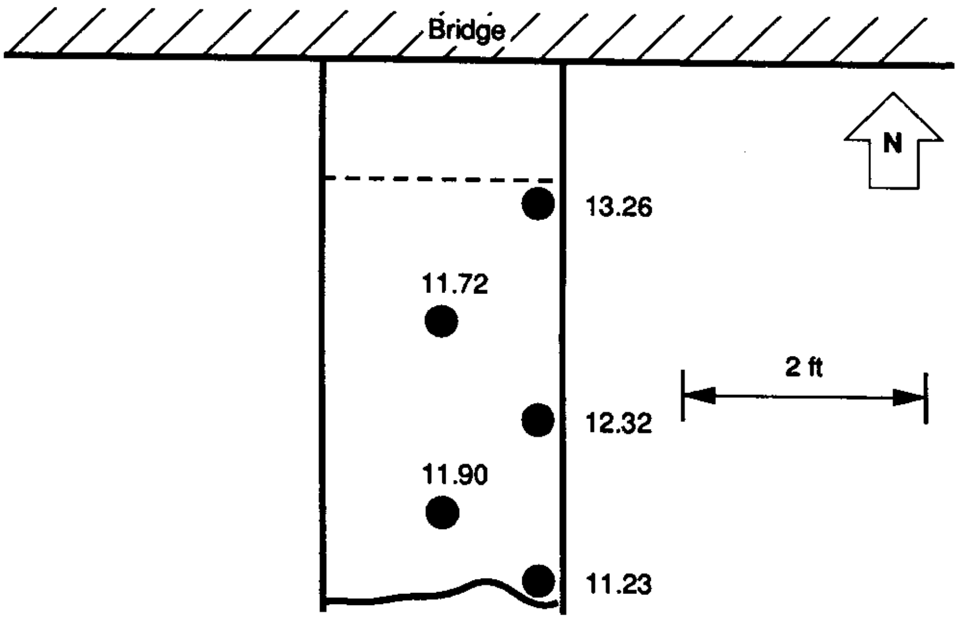


Figure 47. Moisture Content (in percent) of Curlew Creek Bridge Soils

6. KETTLE RIVER BRIDGE

Location

The Kettle River Bridge carries two-way SR 21 traffic over the Kettle River approximately 18 miles north of junction SR 20 and 10 miles south of the Canadian border in Ferry County.

File Search

Files for this structure were found in the Materials Laboratory in Tumwater. No files were available from the Main Engineering Records Office in Olympia. No construction or maintenance records were available for this structure.

History

The bridge was constructed in 1960. The structure replaced an older steel truss span and by-passed a section of substandard winding highway through the community of Curlew. The older facilities were unable to accommodate heavier logging vehicles.

Soils, Foundation, Structure, and Pavement Structure

The general geomorphology traversed by the alignment of SR 21 in this area is dictated by terrace formations, glacial-debris choked valleys, and the flood plains of Curlew Creek and of the Kettle River. The terraces consist of layered sand and gravel with occasional silty sand layers. The flood plain deposits contain stratified sands and sandy silts that vary from a nominal depth to 9 feet and overlie gravels. A considerable amount of organic material was noted in the silt fraction of the flood plain material at several locations. Layers of sand and occasional pockets of organic material were encountered during drilling. The subgrade consists of gravelly, silty sand. The embankment sections average 10 feet high and reside on a swampy flood plain with varying depths of saturated clean sands, silty sand, and sandy organic silts, which overlie firm material. Some settlement was observed during construction; most was immediate settlement due to previous sand strata interbedded with the organic soils. The slopes were determined to be stable for the intended design. Natural levees have been deposited on each side of the Kettle River; the embankments appeared likely to pond water during and after flooding if adequate drainage is not installed at topographic low points in the flood plain.

The bridge is a 205 ft. single-span steel structure with full-height abutments. It does not have approach slabs and is not skewed. A minimum total surfacing depth of 5 inches was anticipated for traffic, with a design frost penetration of 13 inches.

Approach Distress

The observed approach distress was minor because of continued BST applications. The BST does not extend onto the bridge deck, but the transition across the bridge ends causes only a slight drop off.

Site conditions

The field investigation was conducted in the afternoon of Thursday, September 13, 1990 in warm and sunny conditions. A state maintenance crew of two men controlled traffic by closing a section of the southbound lane across the bridge. The state crew also recompacted and patched the test section after the field investigation had been completed. One test section was removed in the northbound lane, on the departure side of the bridge, in the wheel path closest to the shoulder, as shown in Figure 48.

Observations

The BST pavement was removed easily and in nearly one piece. The pavement was consistently 4 inches thick, which was considerably thicker than pavement thicknesses for the other structures along SR 21. The 4-inch-thick basecourse beneath the BST was easily excavated and required only minor picking. The basecourse appeared very similar to the subgrade fill, but was more of a manufactured, fractured face mix. Digging down into the sandy gravel fill against the abutment, a large void - 4 inches out from the abutment and 6 inches down the abutment backwall - was uncovered at the corbel, as shown in Figure 49. The upper portion of the void is shown in Figure 50. The excavation was stopped at this level.

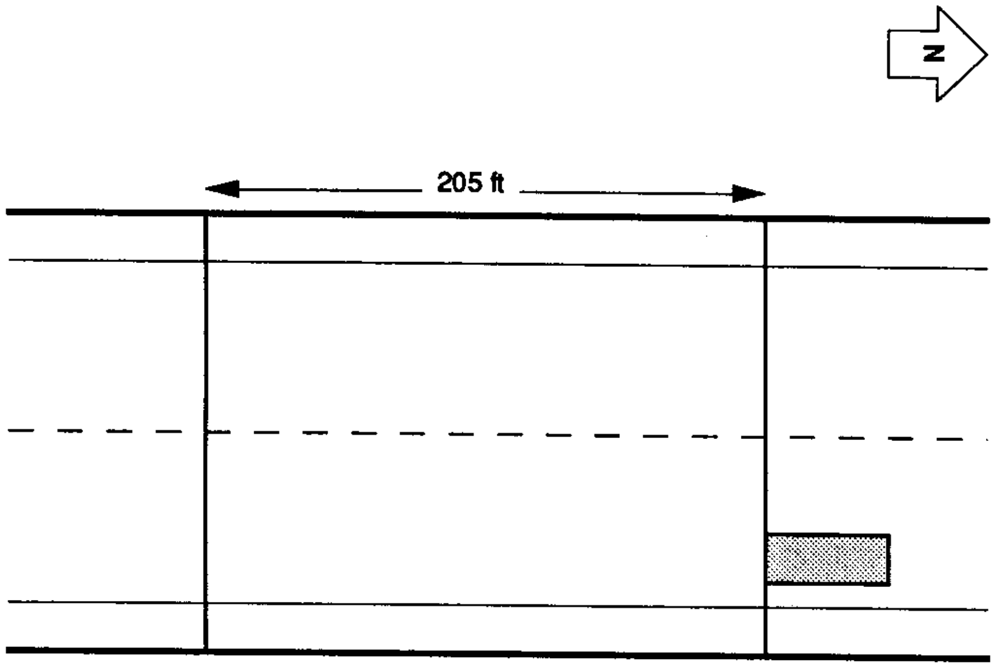


Figure 48. Location of Test Section in Northbound Lane of the Kettle River Bridge (M.P. 156)

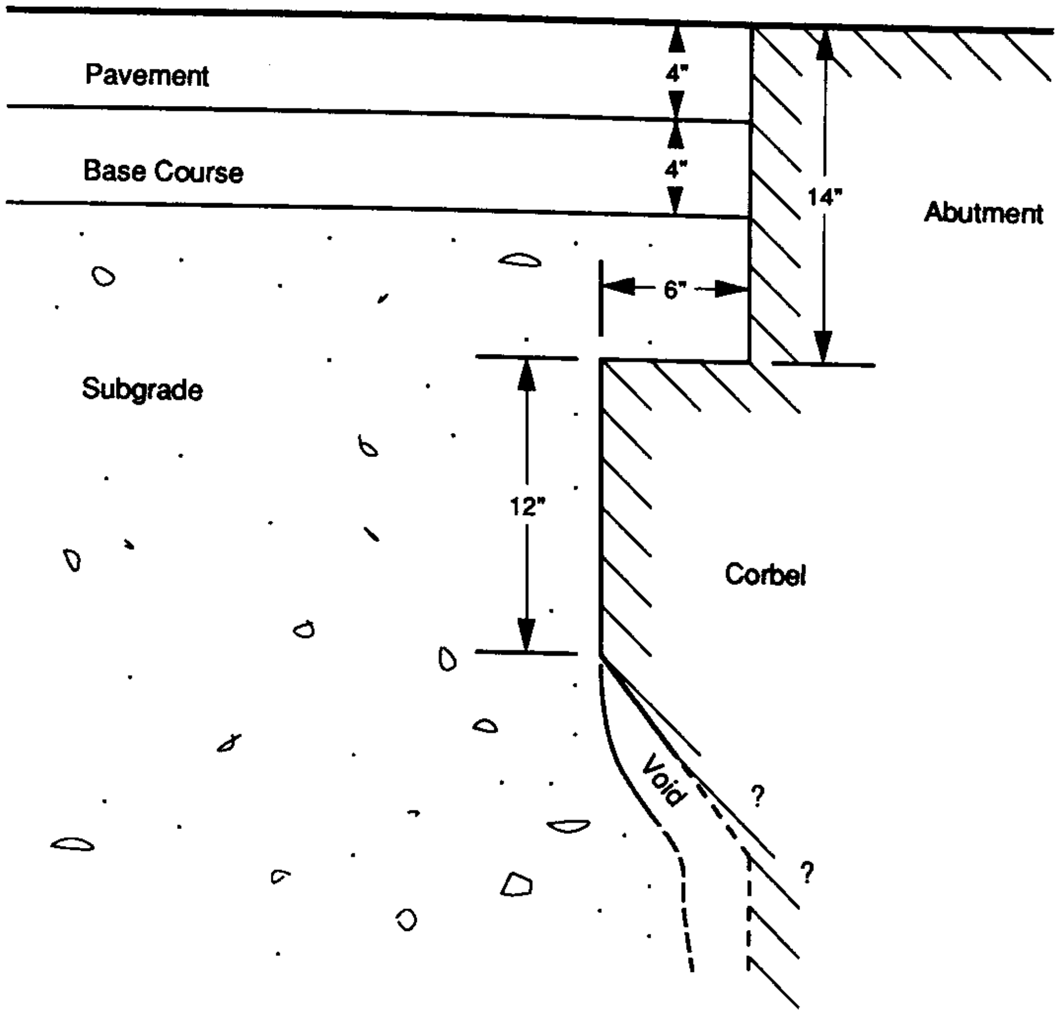


Figure 49. Profile through Kettle River Bridge Abutment

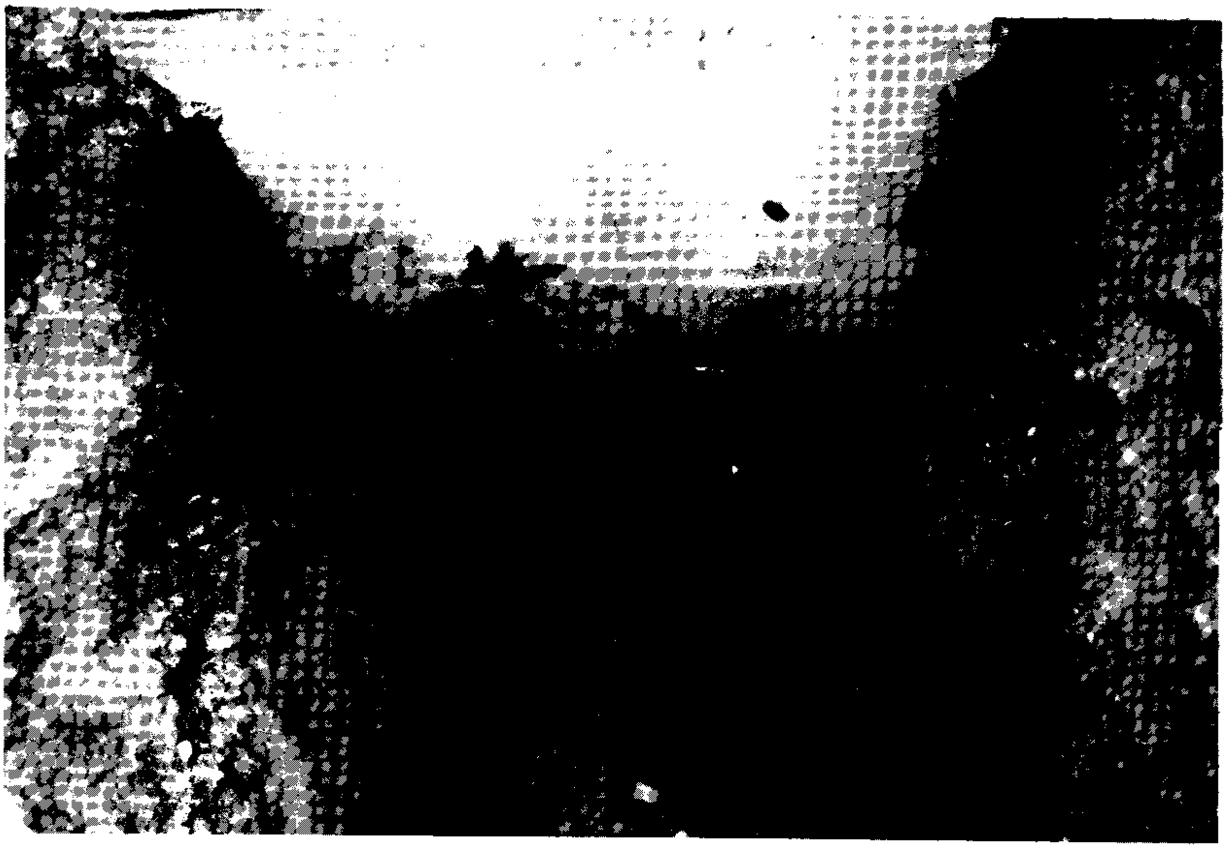


Figure 50. Void Under Abutment "Corbel" at the Kettle River Bridge

Test Results

1. Gradation. Grain size samples were obtained for both the basecourse and the subgrade soils. Grain size distributions are shown for the basecourse in Figure 51 with percentage of fines, and D_{30} is shown in Figure 52. The average percentage of fines for the basecourse was 6.0 percent minus #200, and the D_{30} equalled 1.1 mm. Basecourse gradations from the materials report for this project were similar to the gradations of the samples taken in the field, though the gradation near the bridge was slightly coarser. The subgrade grain size curves are plotted in Figure 53, and the percentage of fines and D_{30} are shown in Figure 54. Both samples, which were taken from the same level in the subgrade, were relatively consistent with respect to gradation.

2. Densities. The densities of the basecourse and the subgrade were measured at various locations in the test section. The densities of the basecourse were measured at 15 and 60 inches from the abutment backwall, as shown in Figure 55. The density averaged 130 pcf, but was lower nearer to the abutment. Modified proctor tests indicated that the maximum dry density of the basecourse was 139.0 pcf at an optimum moisture content of 8.0 percent, indicating average relative compactions of 93 percent. Subgrade density tests were taken at distances of 12 and 35 inches from the abutment (Figure 56). Once again, the densities were lower for points closer to the abutment. The maximum dry density was 137.5 pcf at an optimum moisture content of 8.0 percent, indicating that relative compactions averaged about 89 percent.

3. Moisture Content. Moisture content samples were taken at several locations in the test section. The results of the moisture content tests on samples from the upper 2 inches of the basecourse are shown in Figure 57. The moisture contents of the basecourse were very similar throughout the test section and averaged 2.6 percent. The subgrade moisture contents were also consistent, varying only 0.1 or 0.2 percent from the average value of 3.24 percent, throughout the test section, as shown in Figure 58.

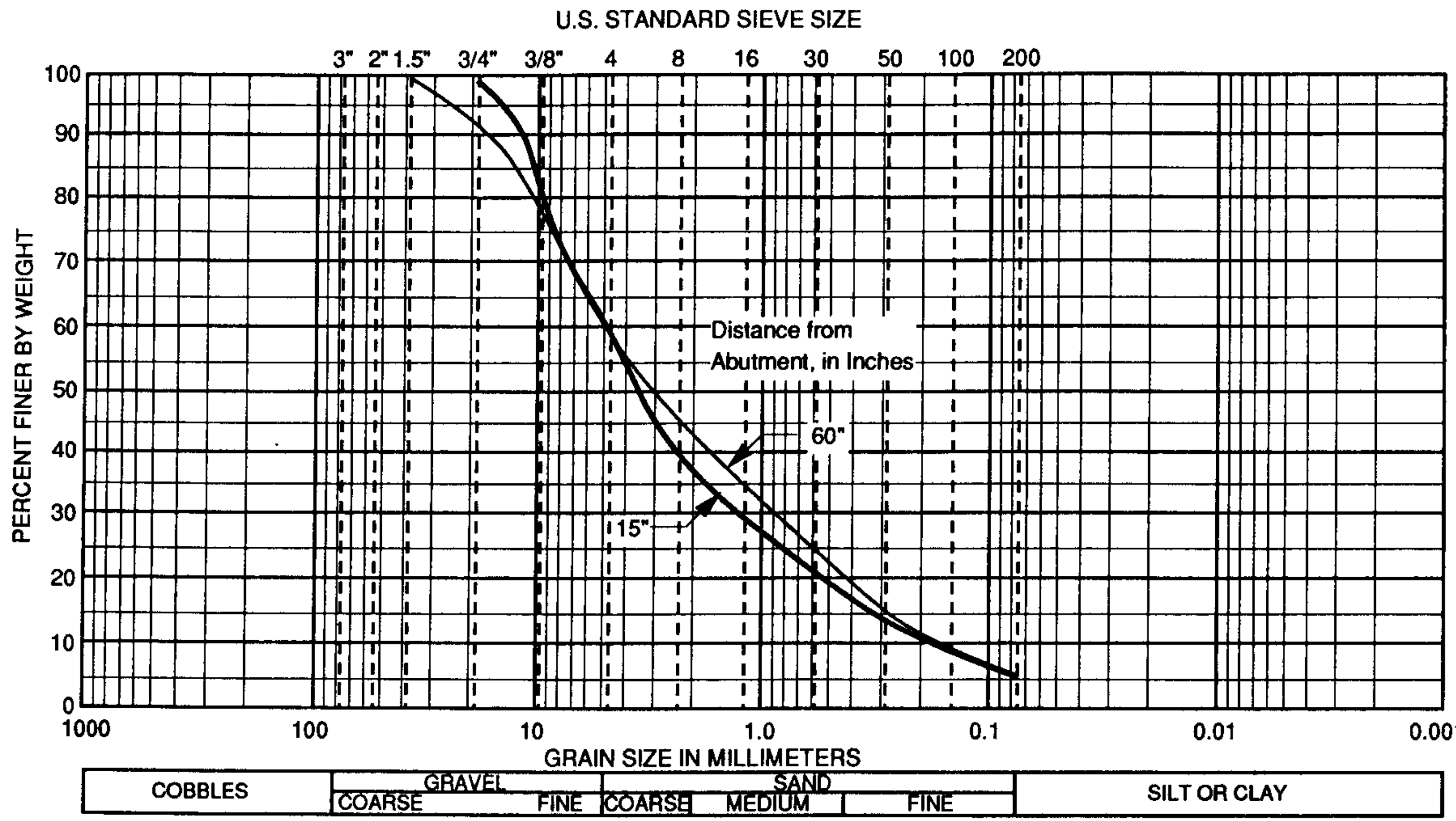


Figure 51. Grain Size Distribution of Kettle River Basecourse

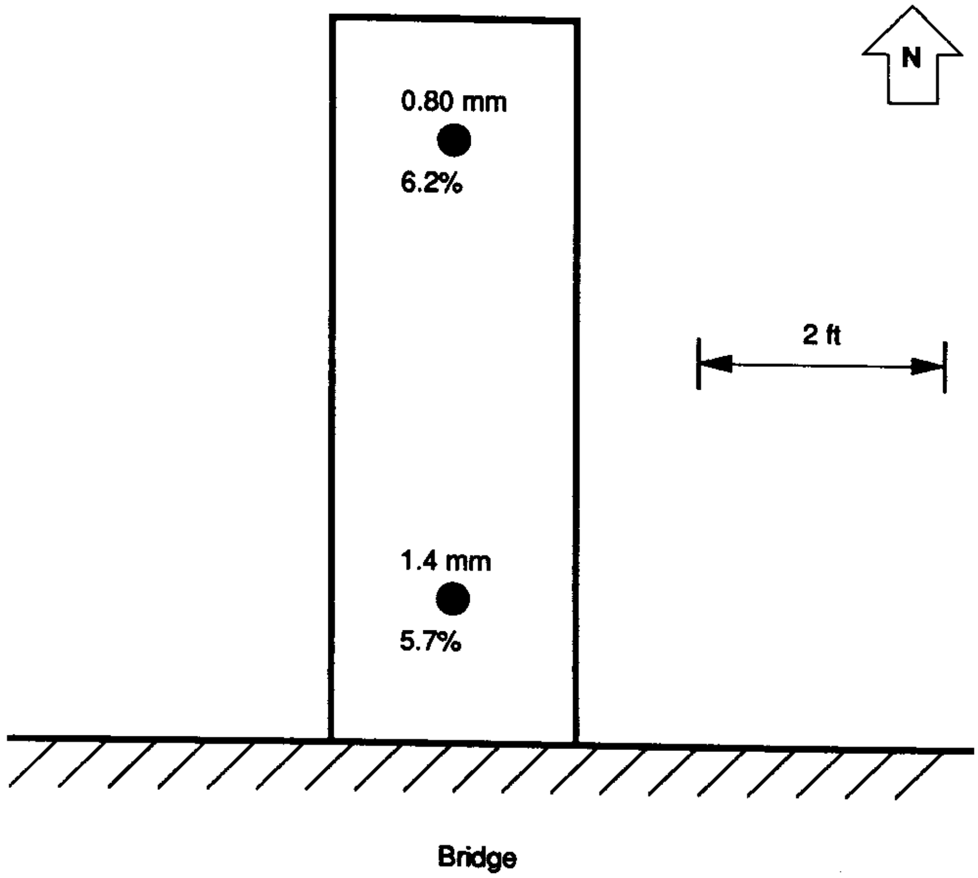


Figure 52. Percent Fines and D_{30} in Kettle River Basecourse

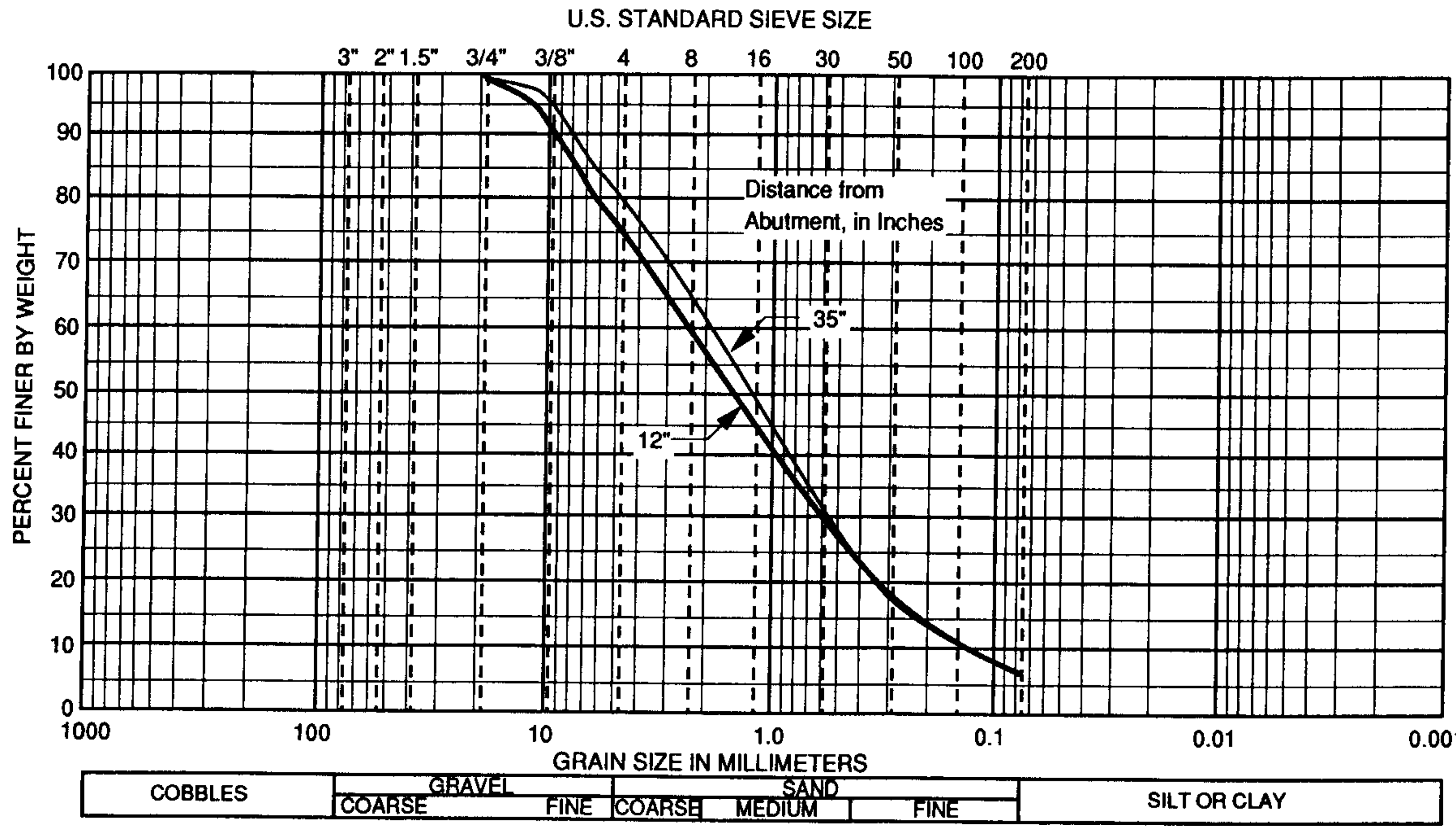


Figure 53. Grain Size Distribution of Kettle River Subgrade

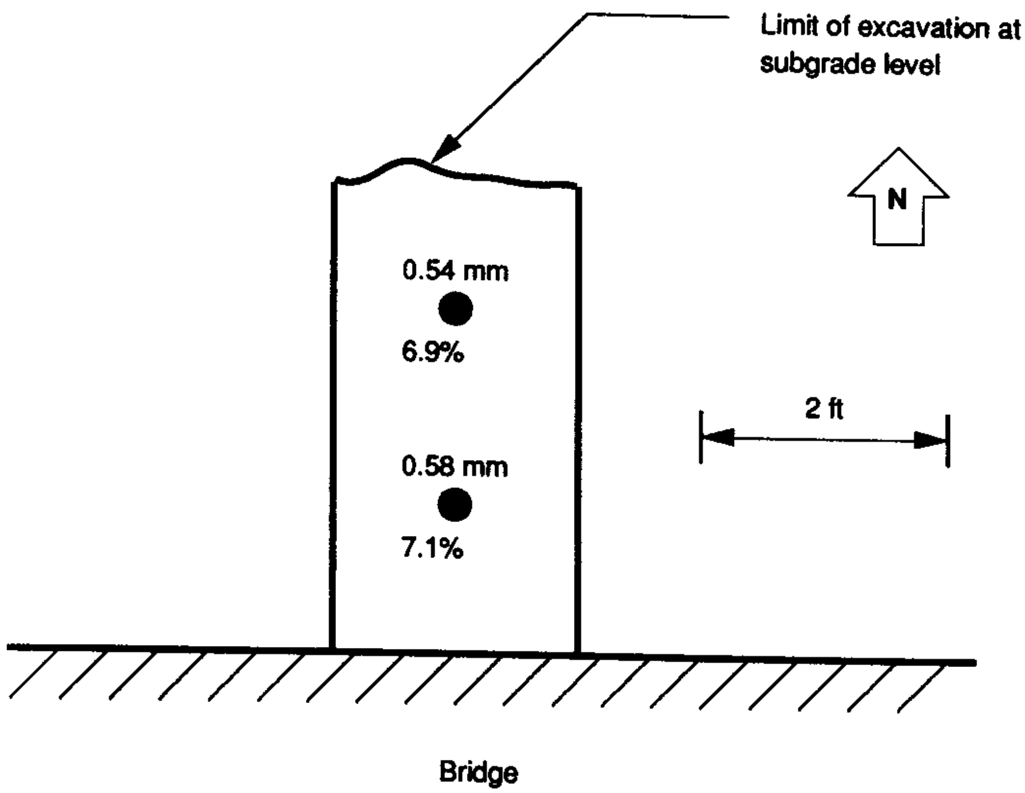


Figure 54. Percent Fines and D_{30} in Kettle River Subgrade

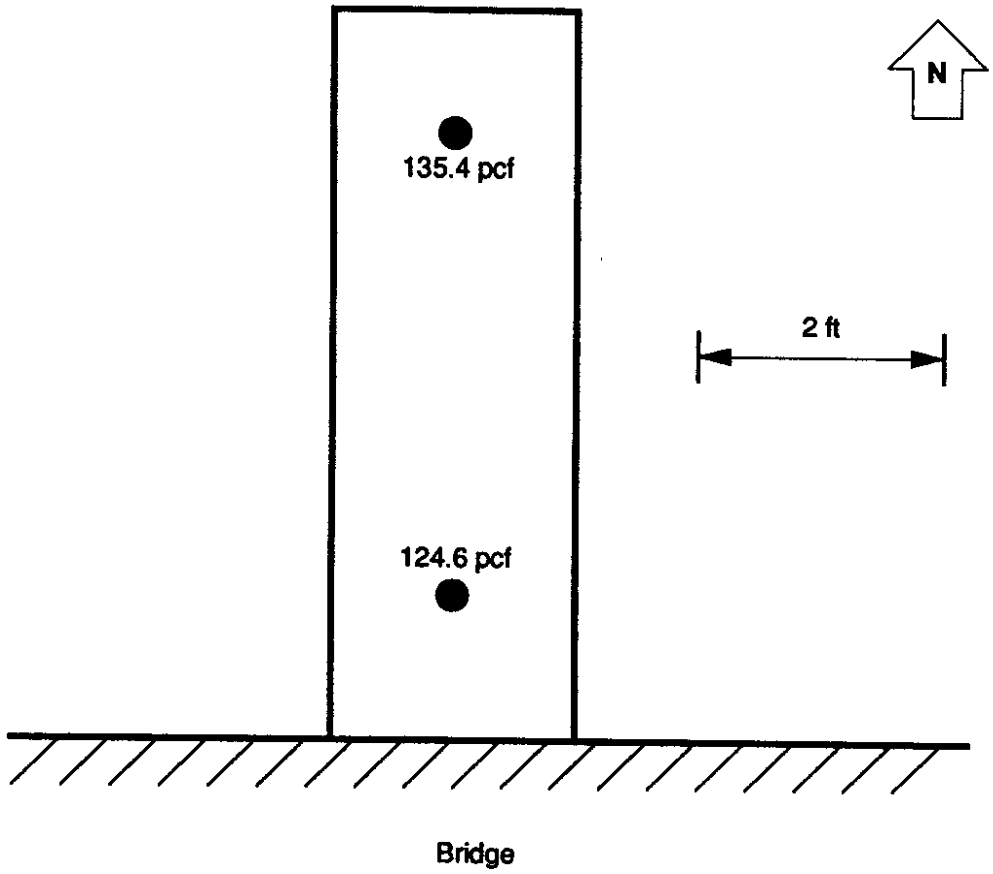


Figure 55. Dry Unit Weights for Kettle River Basecourse

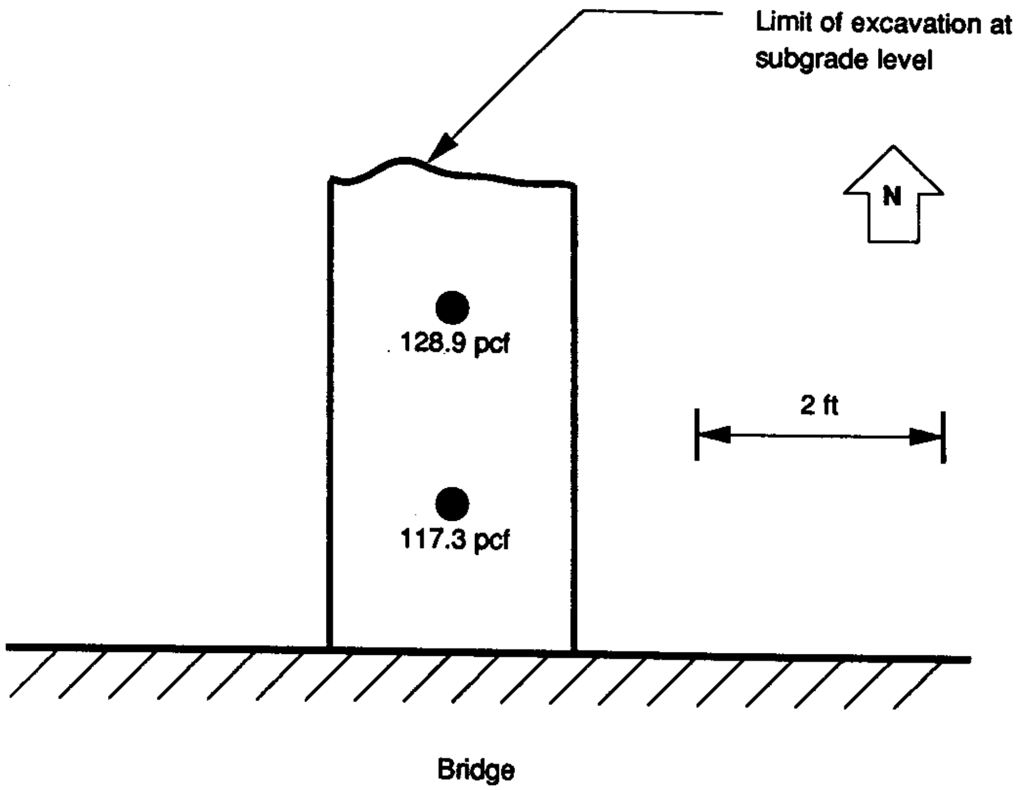


Figure 56. Dry Unit Weights for Kettle River Subgrade

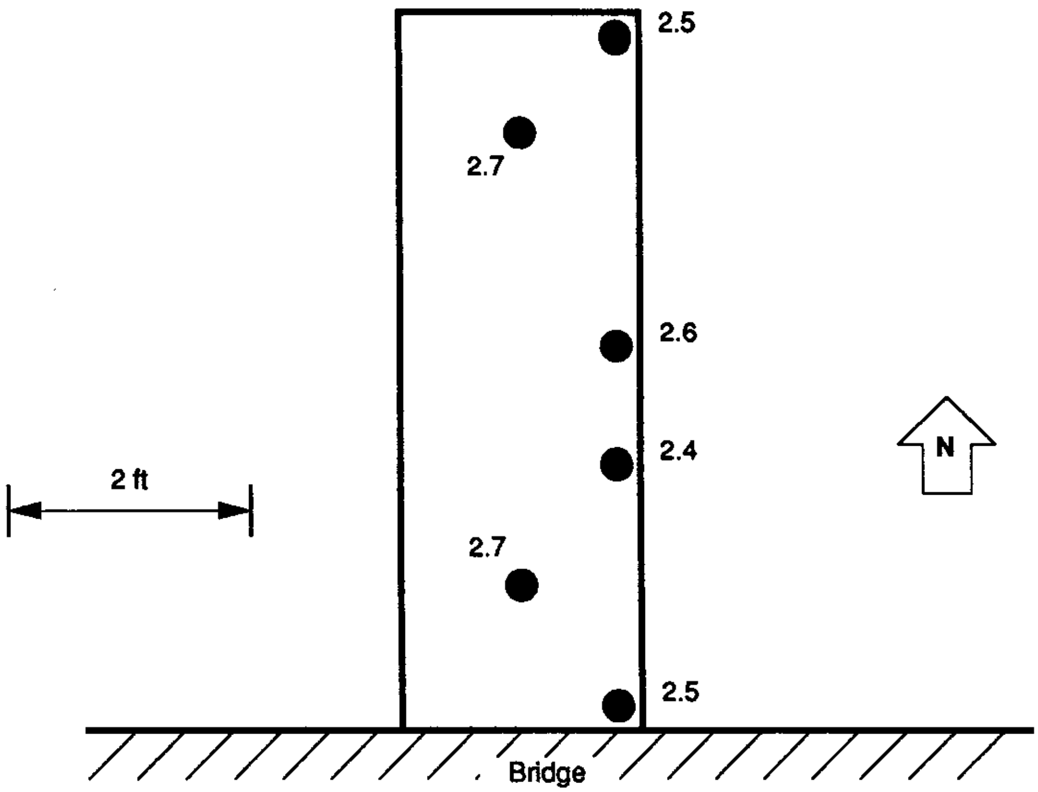


Figure 57. Moisture Content (in percent) of Kettle River Bridge Basecourse

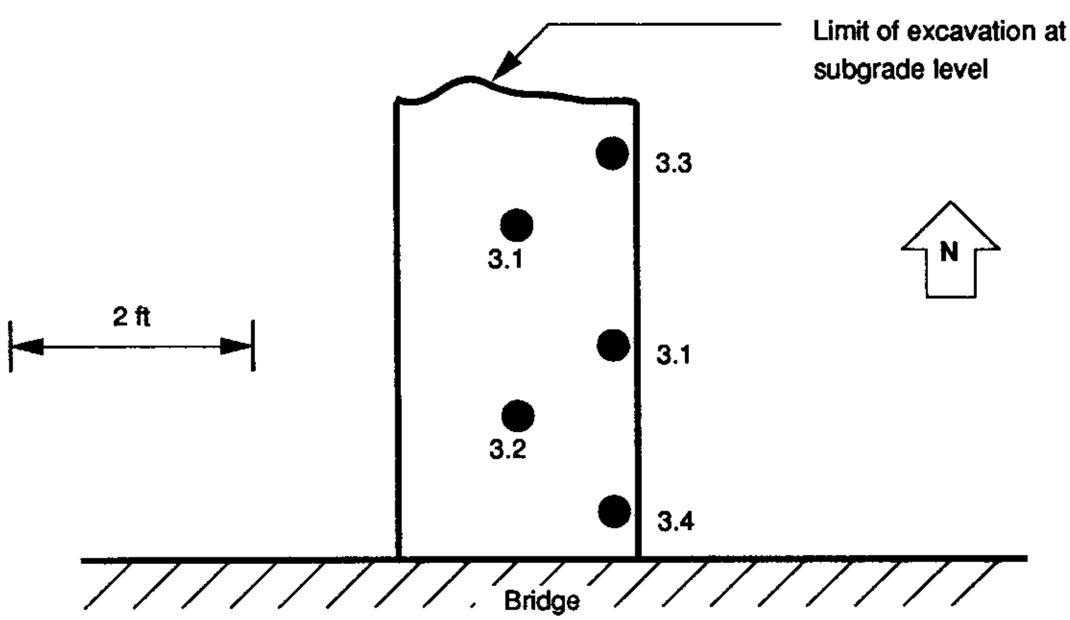


Figure 58. Moisture Content (in percent) of Kettle River Bridge Subgrade Soils

Conclusions

The apparent cause of approach distress on this bridge is a void that had been forming under the corbel of the abutment. Test results indicated lower densities in both the basecourse and the subgrade soils at distances close to the abutment. The lower densities may have been related to inadequate compaction or piping of fines caused by seepage to the void. Since the moisture contents and the gradations taken in the test section were relatively uniform, inadequate compaction is considered the likely cause of the observed distress.

7. NORTH FIRST STREET OVERCROSSING

Location

The North First Street Overcrossing structure carries SR 12 over SR 82 at milepost 202.50 in Yakima County. The bridge is roughly 1 mile north of Yakima, Washington.

File Search

A file search was undertaken to review documents pertaining to the North First Street Overcrossing structure. Files available from the Materials Laboratory contained foundation reports detailing borehole information, recommended footing elevations, and allowable bearing values. The engineering record office in Olympia no longer housed construction information regarding the North First Street Overcrossing.

History

The North First Street Overcrossing structure was constructed in 1969. Maintenance personnel indicated that final paving of the approaches took place during the early season when the fill and base materials were frozen. Approach pavements were later observed to settle significantly upon thawing.

Soils, Foundation, and Structure

As part of the foundation investigation, conducted in 1969, a district materials crew drilled eight bore holes to evaluate foundation support requirements. The foundation material consisted of very compact, silty, sandy gravel containing numerous cobbles and occasional boulders. Overlying this sequence was 10 feet of medium stiff to stiff silt. The 26-foot approach fills were determined to be stable with immediate settlements up to 0.5 ft. The very compact foundation material was determined to be able to adequately support spread footings at all piers. The bridge is a 208-ft., three-span, pretensioned, concrete beam structure with closed abutments and no approach slabs. It is not skewed in relation to the mainline of SR 12.

Approach Distress

Approach distress on the westbound lane was evident from the 3- to 5-feet wide patches across the approach area, as shown in Figure 59. Regular maintenance has usually been performed each spring and sometimes twice per year. Approach distress at this bridge is well known to truckers, who slow down to avoid bruising the apples they haul.

Site conditions

The field investigation of the westbound structure took place on Tuesday morning, September 25, 1990. Warm and sunny weather conditions prevailed. A state maintenance crew of three men controlled traffic by closing the right travel lane. The state crew also recompacted and patched the test section after the field investigation had been completed. One test section was chosen at the location shown in Figure 60. The test section was located in the right wheel path of the westbound lane.

Observations

The AC pavement was slowly removed with an electric jackhammer. While the AC pavement was being removed, several layers of patches were uncovered. The original AC surface appeared to be at a depth of 6 to 7 inches. About 12-1/4 inches of AC pavement were measured at the test section. The AC pavement was underlain by approximately 8 inches of basecourse. The basecourse appeared to be in acceptable condition: it had consistent gradation, normal water content, and good compaction. Beneath the basecourse

Figure 59. Approach Patching on the North First Street Overcrossing Structure

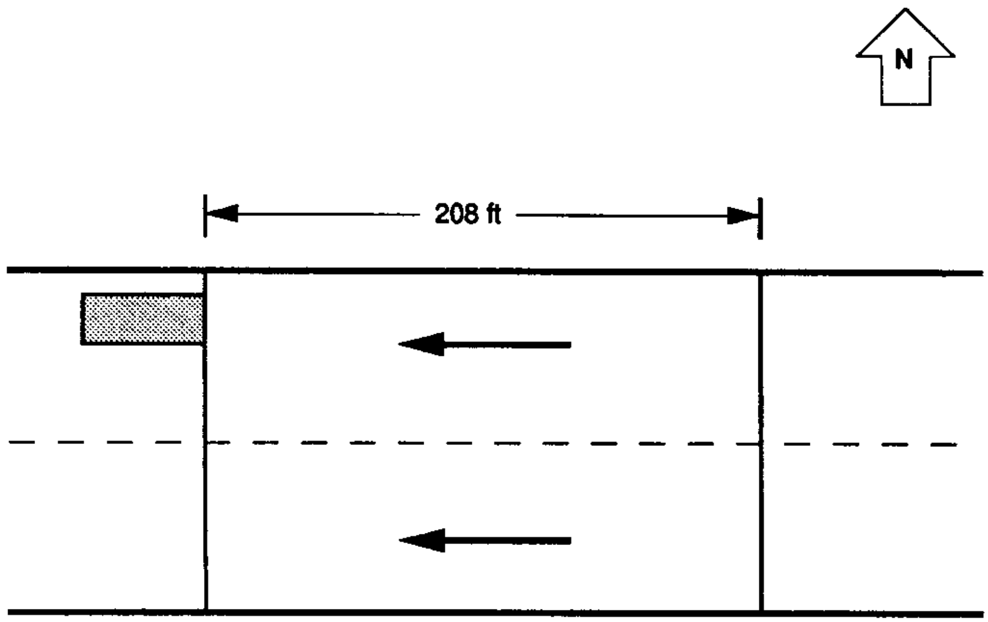


Figure 60. Location of Test Section in Westbound Travel Lane of the North First Street Overcrossing

was a fill subgrade composed of rounded gravel and numerous large cobbles. The remaining fill was of very rounded, sandy gravel. The pavement seat was found at a depth of 15-1/2 inches and extended outward for 6 inches, then dropped vertically down for 12 inches more, as shown in Figure 61. A void and/or loose zone was noted within approximately 2 inches of the sloping part of the corbel. The void was distinct in some places, but appeared to have filled with loose slough at other locations. The void and many cobbles in the embankment fill can be seen in Figure 62. Inspection of the appurtenant areas revealed no evidence of erosion underneath or at the sides of the bridge.

Test Results

1. Gradation. Grain size samples were obtained for the basecourse in the test section. A grain size curve is plotted in Figure 63 for a sample of the basecourse taken 9 inches from the abutment. The sample consisted of 8.0 percent fines and its D_{30} was 1.2 mm. The gradation of the basecourse was within the WSDOT specification for crushed surfacing.

2. Densities. The densities of the basecourse were measured at the location of the grain size distribution sample. The basecourse density, obtained by the sandcone method, was 123.7 pcf. This density was lower than expected for a typical basecourse. The density of the subgrade could not be tested because of the high content of closely spaced cobbles.

3. Moisture Content. Moisture content samples were taken at the locations shown in Figure 64. The average basecourse moisture content was 5.0 percent, and the moisture contents appeared to be consistent for the basecourse.

Conclusions

The probable cause of approach pavement distress at this bridge is poor compaction of the embankment fill adjacent to the bridge abutment, which may have been compounded by thermal movements of the bridge. Final paving construction completed on frozen subgrade soils could also have contributed to the overall approach problems. Subgrade soil

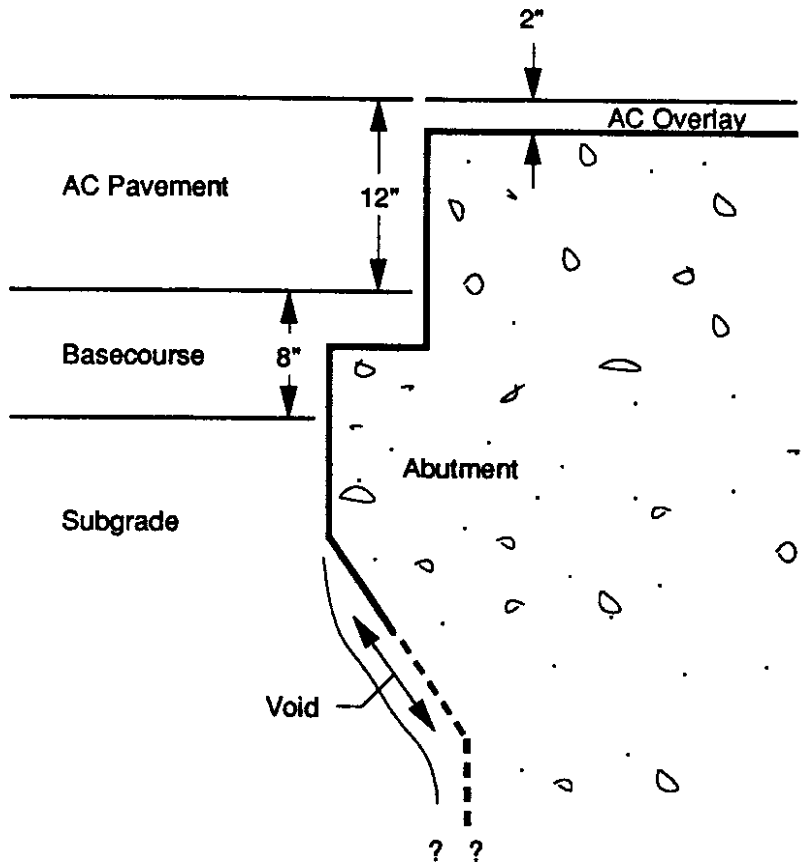


Figure 61. Soil Profile Behind Abutment

Figure 62. Void under the Corbel of the North First Street Bridge.
Note the many large cobbles

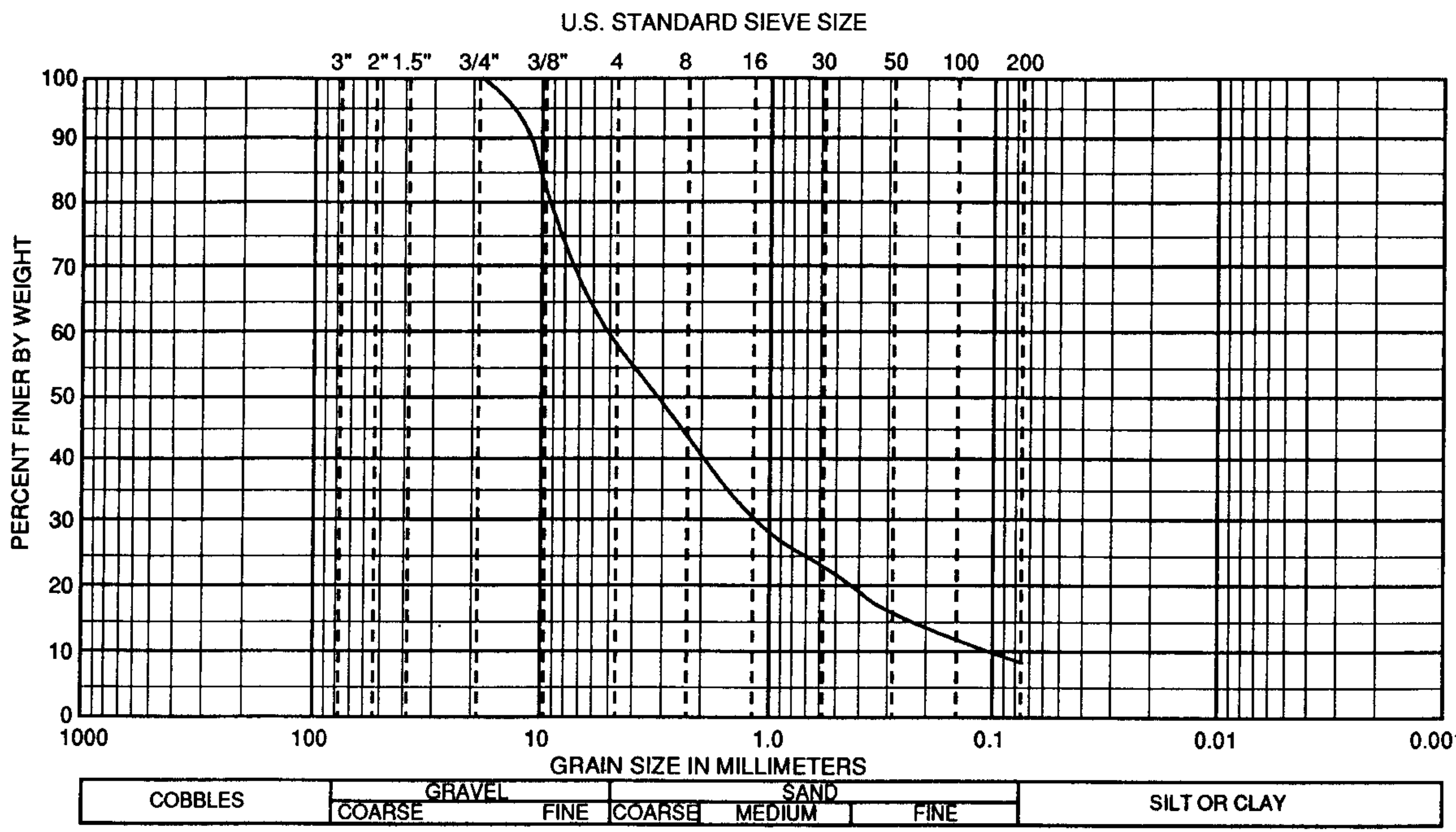


Figure 63. Grain Size Distribution of North First Street

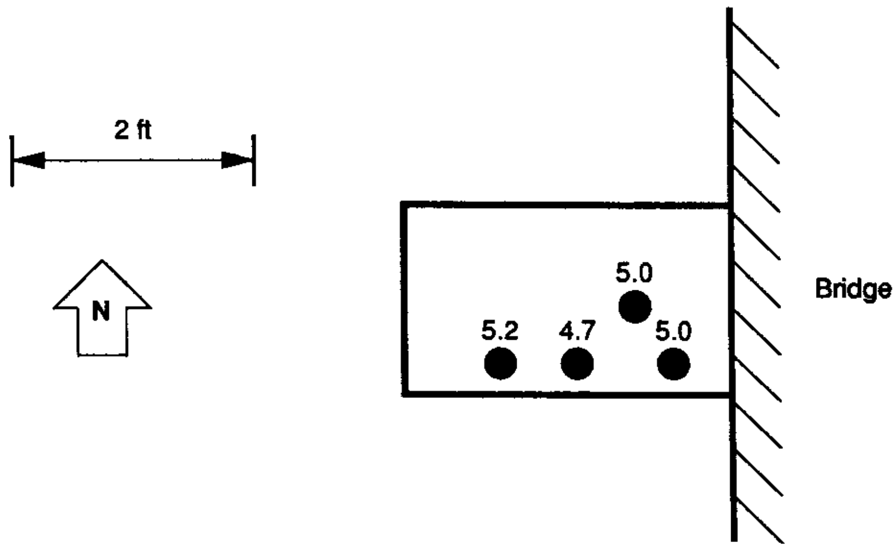


Figure 64. Moisture Content (in percent) of North First Street Overcrossing Basecourse

gradation, with cobbles up to 10 inches, allowed the small particles between the larger cobbles in the fill to get little to no compaction, particularly near to the abutment, where small wackers were used for compaction.

8. WEST-WEST RAMP, INTERSTATE 82 OVERCROSSING

Location

The West-West Ramp of the Interstate 82 Overcrossing structure is a one-lane bridge that carries westbound Interstate 82 traffic to westbound SR 12. This interchange is at milepost 202.50 in Yakima County, one mile north of Yakima, Washington, and approximately 100 yards east of the North First Street Overcrossing.

File Search

Files were not available for this structure from any of the state engineering record offices.

History

This bridge was constructed in 1969 along with the previous structure as part of the SR 12- Interstate 82 interchange. Final paving was also apparently performed on frozen soils for this bridge.

Soils, Foundation, and Structure

Bridge site soil conditions are as described for the North First Street overcrossing. The approach embankments are 50 feet high on the entrance side of the bridge, while the exit side embankment is approximately 30 ft. high. The West-West Ramp Interstate 82 Overcrossing is a four-span, pre-tensioned, concrete beam structure 385 feet long, with closed abutments. The overcrossing structure is a four-span, 385-foot, pre-tensioned concrete beam structure and is skewed 37 degrees relative to the mainline highway alignment.

Approach Distress

A recent patch, approximately 1-1/2 inches thick, covers the approach area. Patches exist at both ends of the bridge: 10 feet wide at the exit and 4 feet wide at the entrance. The bridge deck has 1-1/2 inches of overlay that appears at least several years old.

Site conditions

The field investigation of the westbound ramp took place on Tuesday afternoon, September 25, 1990. Weather conditions were favorable. One test section was chosen in the left side of the single ramp lane as shown in Figure 65.

Observations

Measurements of the cut pavement section disclosed roughly 5 inches of patches in approximately 1-inch-thick layers. The top of the basecourse was 18-1/2 inches down. Basecourse conditions were similar to those at the last bridge. The basecourse appeared wetter toward the bottom of the pavement seat. The abutment pavement seat and corbel were located 15 inches below the road surface, as shown in Figure 66. Again, a void between the abutment backwall and the adjacent soil was observed in the sloping portion of the backwall. The upper portion of the void is shown in Figure 67. The subgrade, which was 25 inches down, contained numerous, closely spaced cobbles.

Test Results

1. Gradation. The grain size distribution of the basecourse at a point 2 feet from the abutment, as shown in Figure 68. A grain size curve is shown for the basecourse in Figure. The basecourse materials at this location consisted of 8.2 percent fines, had a D_{30} of 1.5 mm, and appeared to meet the WSDOT specification for crushed surfacing.

2. Densities. The dry unit weight of the basecourse at the location of the grain size distribution sample was 120.0 pcf. The subgrade soils consisted of cobbly material and, therefore, could not be tested for density.

3. Moisture Content. Moisture contents at several locations in the test section were as shown in Figure 69. The moisture content increased with distance from the abutment backwall and had an average value of 5.7 percent. An additional sample taken at

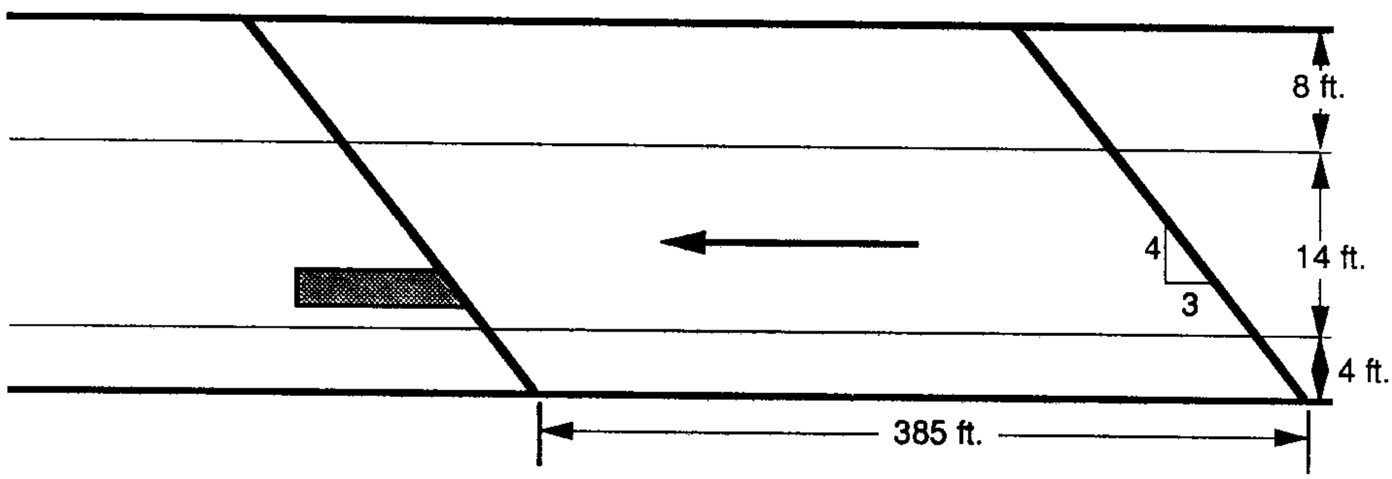


Figure 65. Location of Test Section for West-West Ramp SR 82

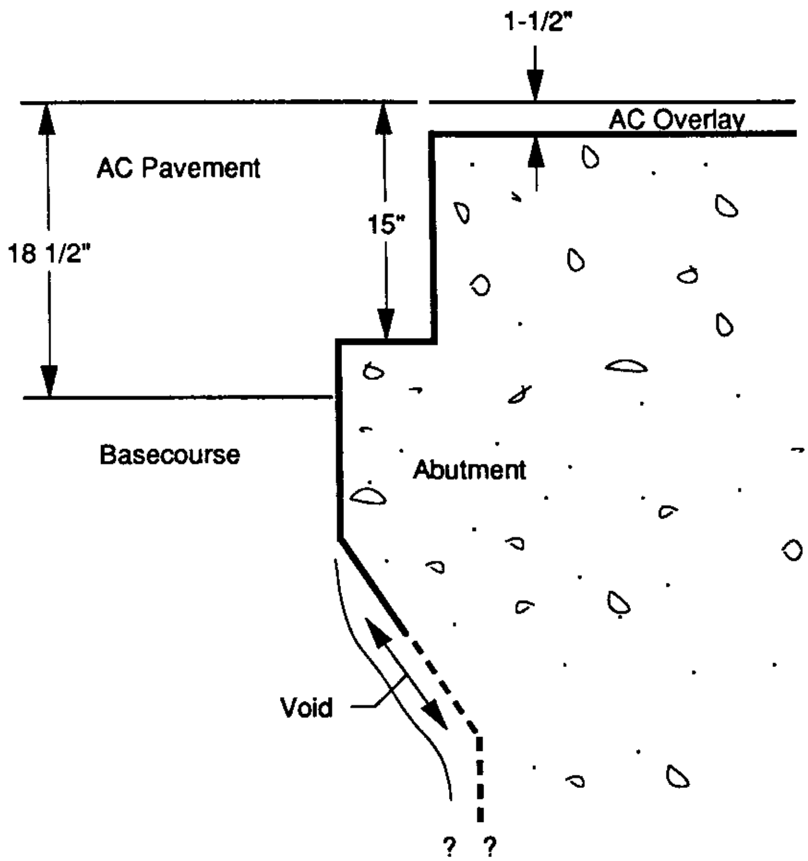


Figure 66. Soil Profile Behind Abutment

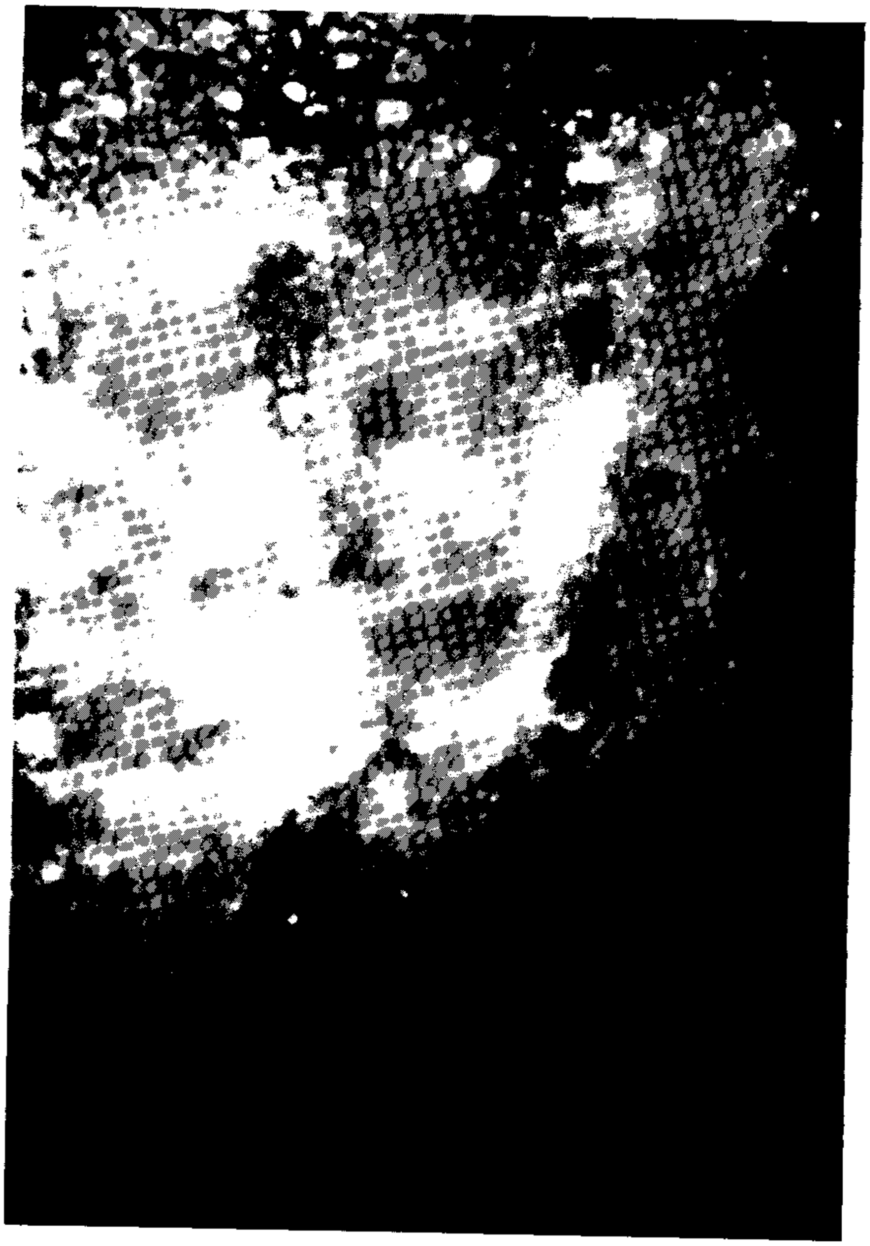


Figure 67. Void Under Corbel at the West-West Ramp Bridge on SR 82

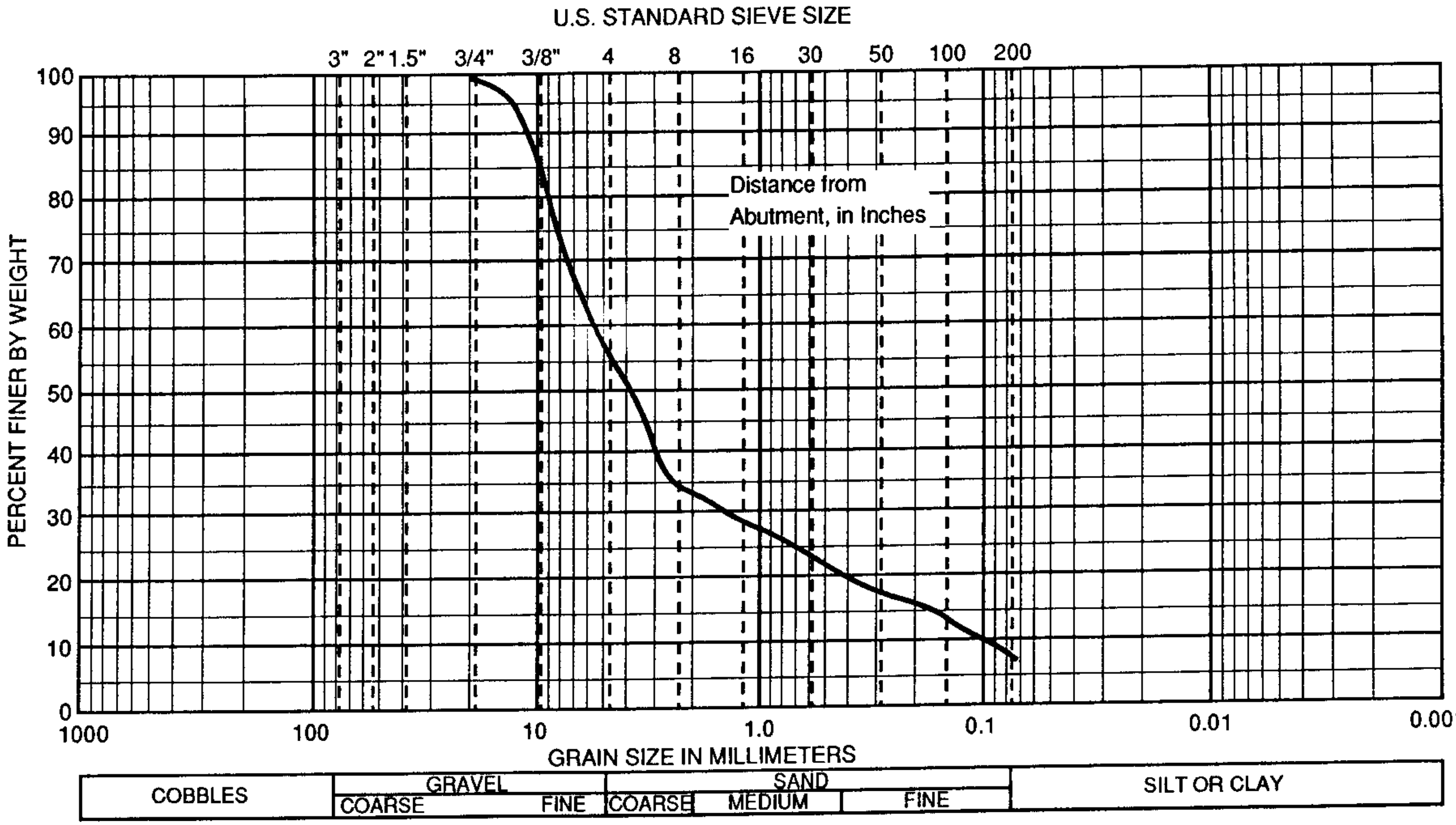


Figure 68. West-West Ramp 82 Overcrossing G.S.P. for Basecourse

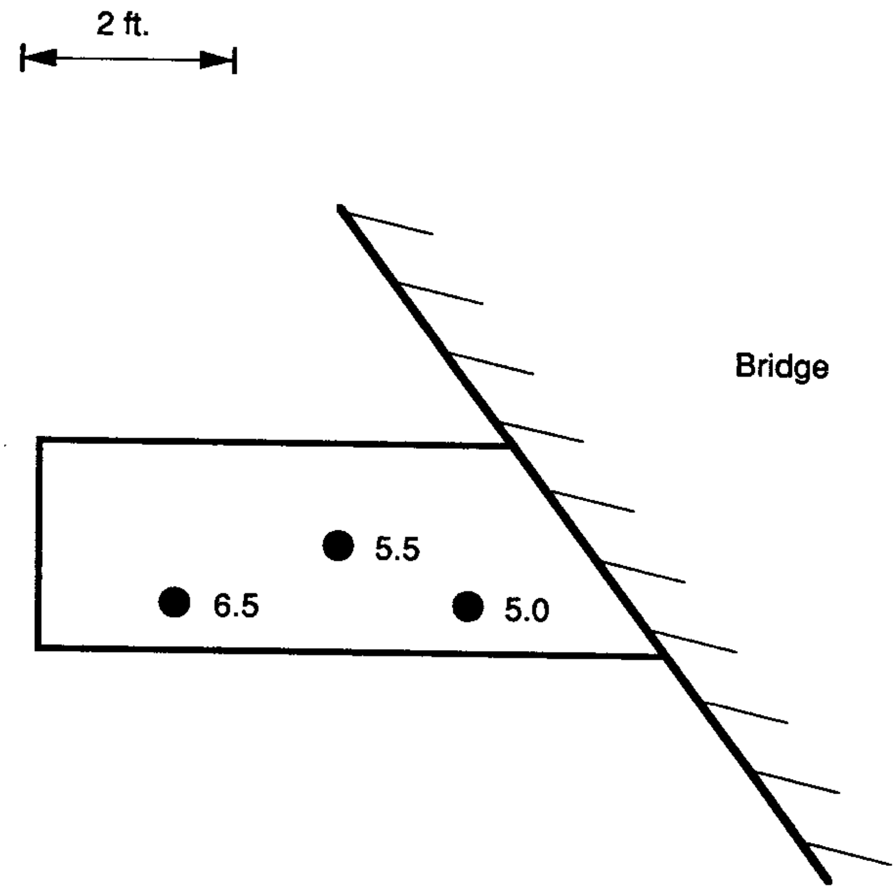


Figure 69. Moisture Content (in percent) of West-West Ramp SR82 Basecourse.

a depth of 27 inches and directly adjacent to the pavement seat had a moisture content of 5.3 percent.

Conclusions

As with many of the previous structures, a significant void appeared to have developed under the abutment corbel. This void created a weak spot and a potential water channel in the subgrade. The void alone could be responsible for the observed approach distress, but could have been complemented by insufficient compaction, thermal bridge movements, and/or lack of positive drainage. The numerous large cobbles in the embankment fill may have strongly contributed to the poor compaction.

9. JOHNSON ROAD UNDERCROSSING

Location

The Johnson Road undercrossing is approximately 3.7 miles east of the Yakima County line at mile post 250.90 and is 2 miles north of Prosser, Washington. The bridge undercrossing carries local traffic over Interstate 82. The bridge has exhibited continued approach problems.

File Search

Documents pertaining to the Johnson Road undercrossing were located at the Materials Laboratory in Tumwater, Washington. The development plans and the foundation report detailed the bridge layout, foundation conditions, and footing capacities. Since this bridge is 12 years old, files were no longer housed at the main engineering records office in Olympia. Neither construction nor maintenance records were available from any of the state record offices.

History

The Johnson Road undercrossing structure was built in 1978 as part of the Interstate 82 freeway system. The local traffic route was maintained by the construction of this undercrossing. Highway 82 passes below this bridge in a basalt rock cut, as shown in Figure 70.

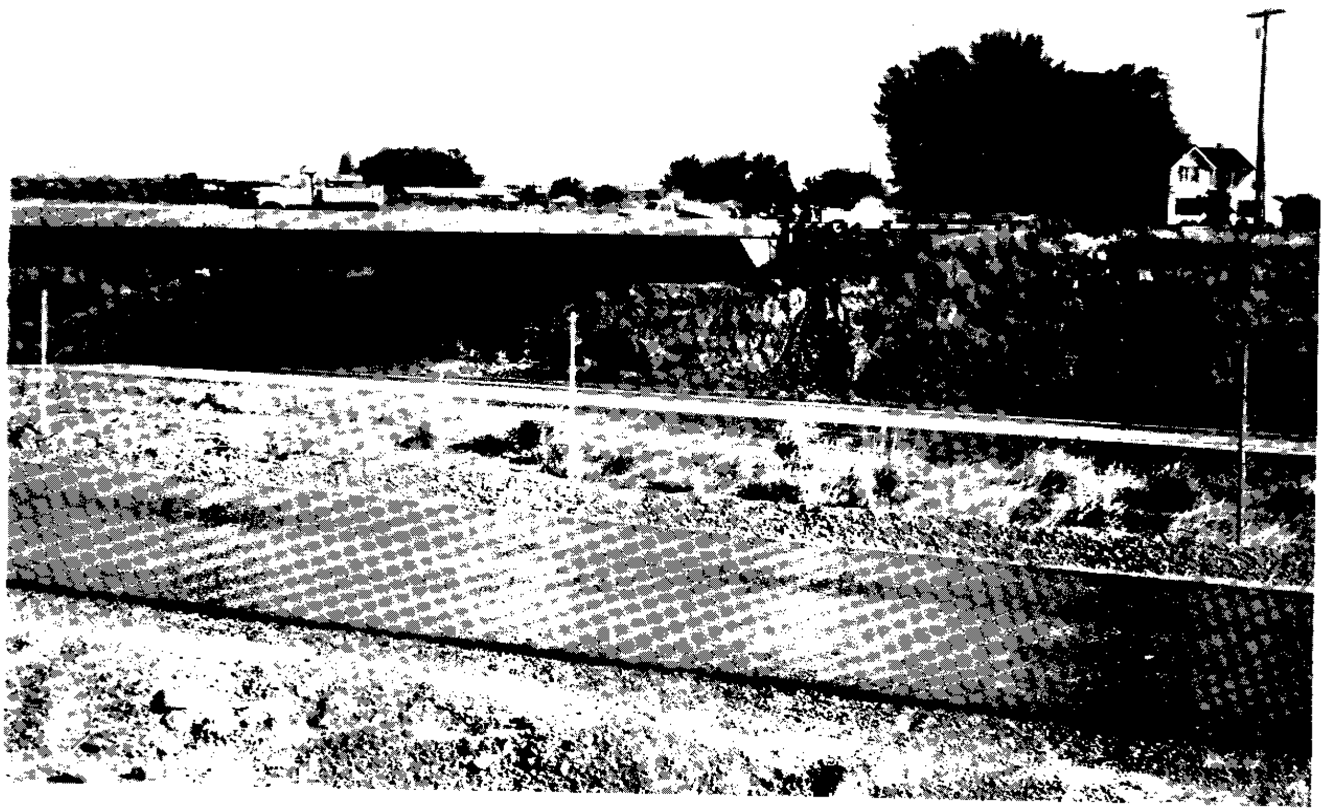


Figure 70. Basalt Rock Cut at Johnson Road Bridge

Soils, Foundation, Structure, and Pavement Structure

As part of the foundation investigation, three test holes were drilled to determine the type of foundation support required. In the test holes, vesicular basalt rock was encountered within 2 ft. of the ground surface. The basalt was highly fractured and of poor quality above elevation 772. In January and February 1976, water was noted within a few feet of the ground surface. At that time, the water appeared to come from nearby irrigation ditches and may have been perched on the more massive rock. Water was encountered at pier 2 during construction, and pumping was required to control its level.

The Johnson Road Undercrossing is a 317-foot-long, two span, post-tensioned, concrete box girder bridge with a 45 degree skew. Apparently, small gaps were left between the girders, and the girders extended somewhat beyond the abutments. No approach slabs exist on this structure, since both approaches are essentially at grade. SR-82 passes below the bridge in a cut section with 3/4:1 slopes at the bridge. Rock beneath the bridge ends was excavated with pre-split methods. All three piers are supported by spread footings on rock as shown in Figure 71. In addition, because of the poor quality of the rock at the bridge ends, the front of the footings had to be located at least 10 ft. behind the face of the slopes.

Approach Distress

Approach distress problems at this bridge have been so bad that maintenance work to improve the rideability of the undercrossing has been repaired six times in the last 10 years. The condition of the pavement at the west abutment was as shown in Figure 72. In one maintenance "fix," the entire east abutment was dug out to a depth of 4 feet and replaced with compacted maintenance sand.

Department of Transportation engineers, who have studied the bridge and its approaches, noticed that sand and gravel had been spilled out underneath the structure like "mini-alluvial fans." Yet the amount of material that spilled through the gaps was not

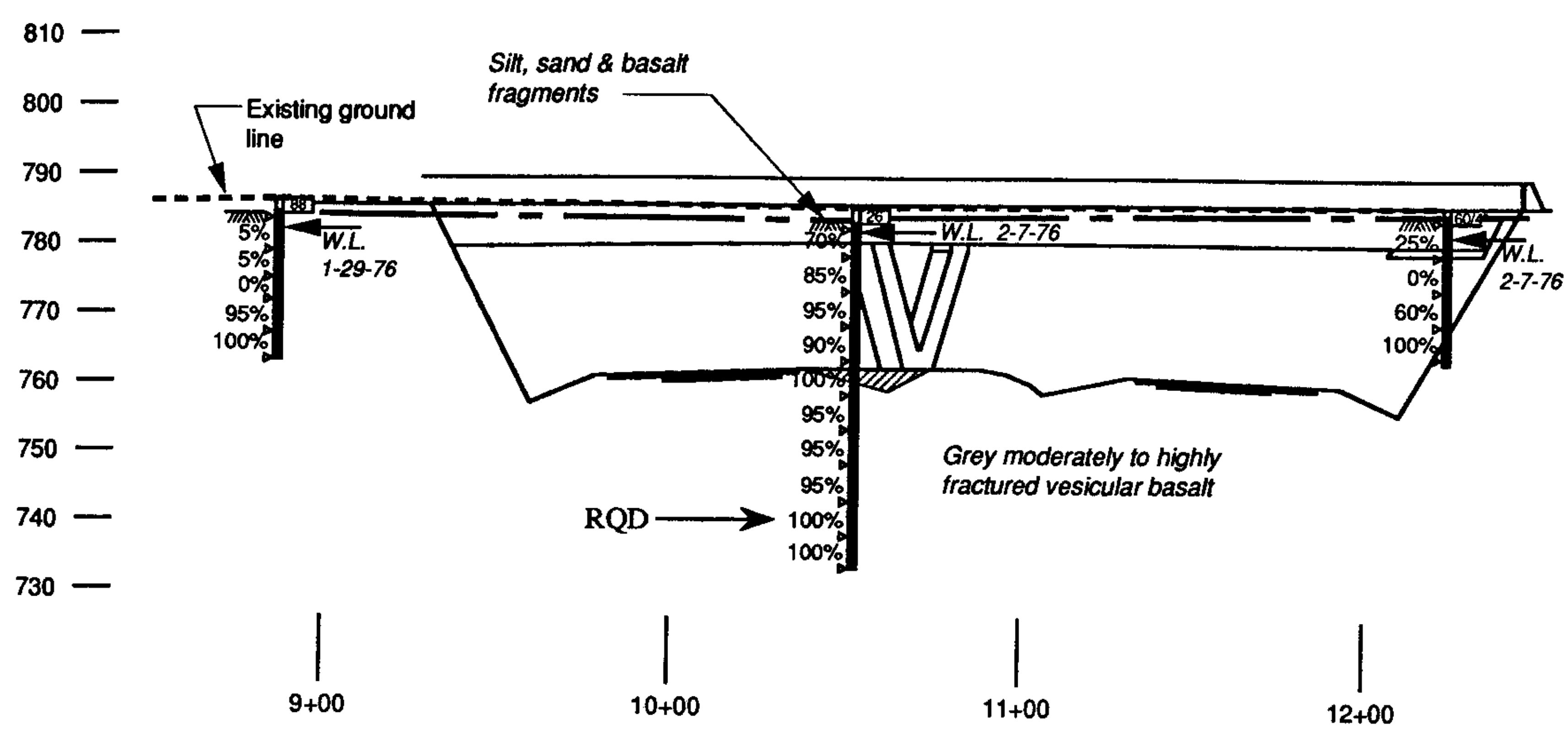


Figure 71. Profile View of Johnson Road Undercrossing

Figure 72. Approach Cracking at the Johnson Road Bridge

enough to explain the magnitude of the observed settlement. The engineers also discovered that the catch basins began leaking as long as 5 years ago. The catch basins were repaired, but settlement problems have continued.

Site conditions

The field investigation of the northbound lane, exit side of the structure was conducted under clear skies on the morning of Wednesday, September 26, 1990. A state maintenance crew of two men controlled traffic by closing the southbound lane. The state crew also recompacted and patched the test section after the field investigation had been completed.

Two test sections were removed in the northbound lane one in the wheel path and the other closer to the shoulder as shown in Figure 73. The first test section was centered on the painted shoulder line, where a sinkhole was forming in the surface pavement. The second test section was excavated to observe the underlying soil profile and obtain samples.

The groundwater is apparently high and irrigation trenches are located nearby.

Observations

Removal of the pavement in the first test section exposed the basecourse, but no evidence of depressions or voids was found. The pavement in the second test section was then removed, exposing maintenance sand below the pavement. This maintenance sand, which was placed during the most recent repair work, was well compacted and had consistent gradation and water content. Excavation of the maintenance sand continued to a depth of 18 inches below the pavement seat. No corbel was discovered below the pavement seat.

Test Results

1. Gradation. A grain size sample of the maintenance sand 12 inches from the abutment showed that roughly 75 percent of the sand was within the #4 and #200 sieves, as shown in Figure 74. The sample consisted of 9.4 percent fines and had a D_{30} of

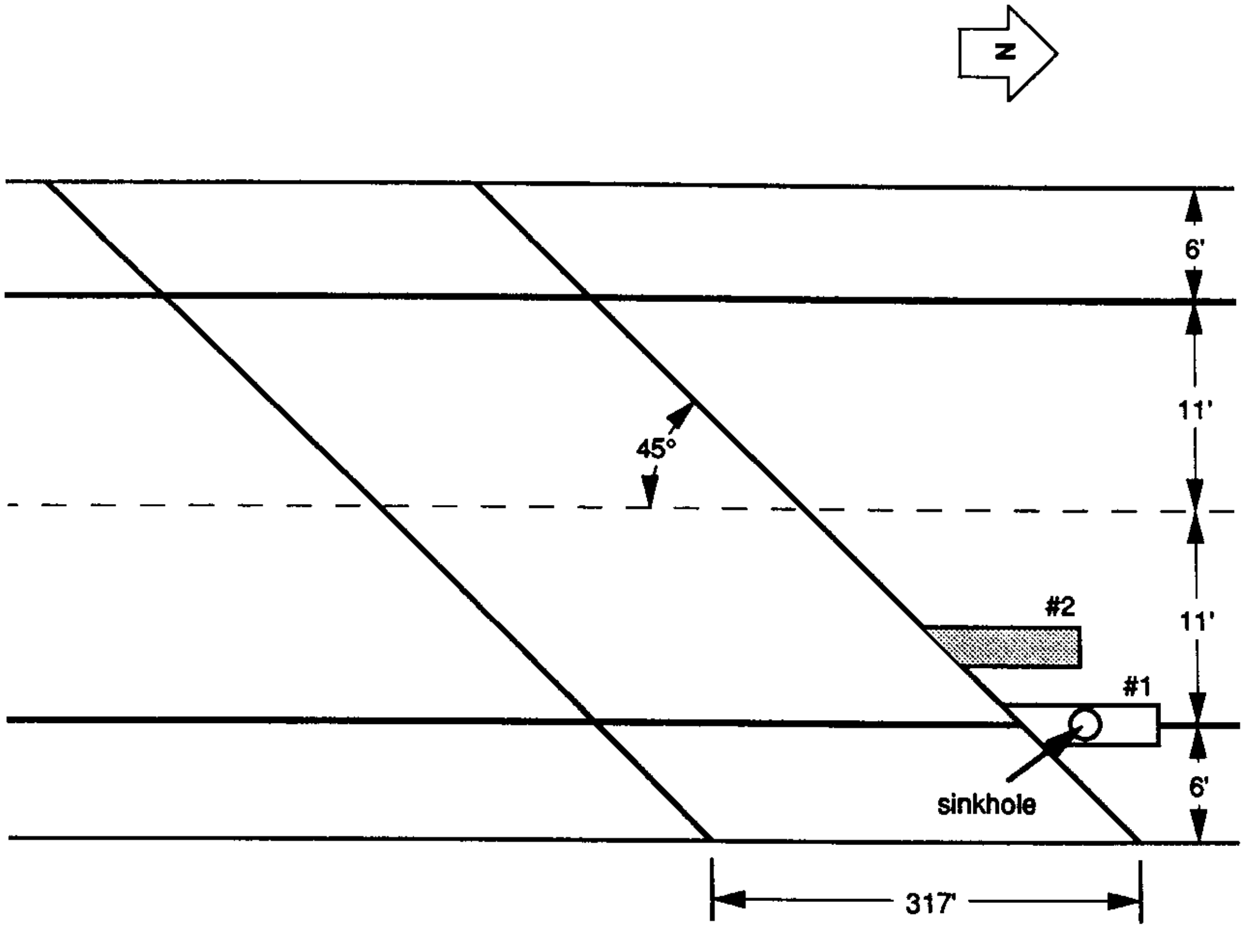


Figure 73. Location of Test Section in Southbound Lane of the Johnson Road Undercrossing

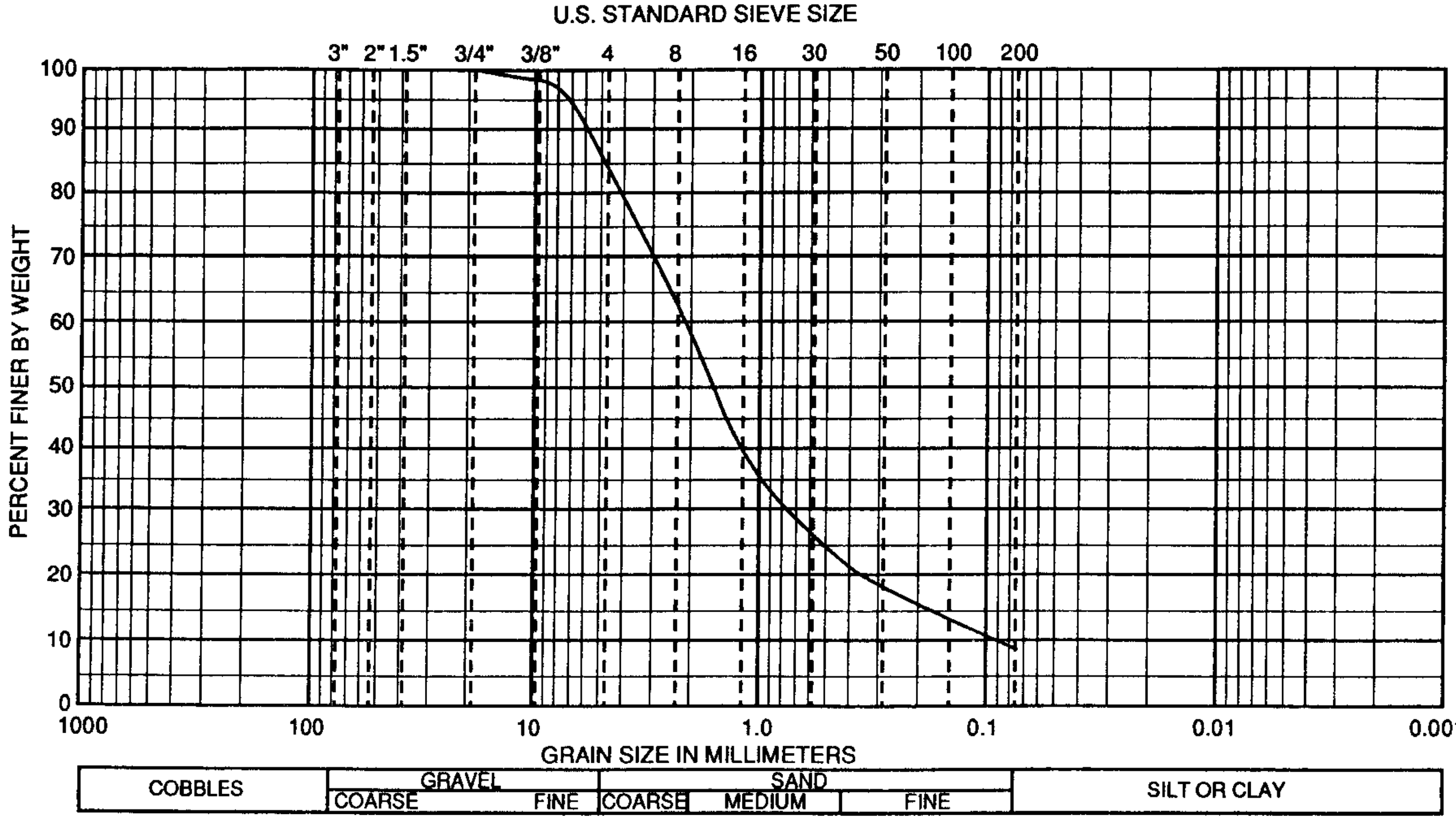


Figure 74. Grain Size Distribution of Johnson Road Undercrossing

0.7 mm. This material would generally be considered too fine for use as a basecourse, but it is a good free-draining backfill.

2. Densities. The dry density of the maintenance sand at the location of the grain size distribution sample was 129.2 pcf, indicating that the sand was well compacted.

3. Moisture Content. The results of moisture content tests on the samples from the maintenance sand are shown in Figure 75. The moisture contents were very close, with an average of 4.9 percent.

Conclusions

Approach problems at the Johnson Road Undercrossing have occurred from the settlement of approach fills. Bridge settlement is unlikely, since the bridge is founded on competent basalt. The cause of the localized approach fill settlement is not apparent, but is most likely the result of poor compaction, which may have resulted in part from the bridge design. The bridge girders, which rest on the abutment, extend beyond the abutment backwall; hence, areas under and adjacent to each of the protruding girders would have been very difficult to compact. Densification of the under-compacted soils is the most likely cause of the observed distress. Thermal bridge movements may have exacerbated the problem.

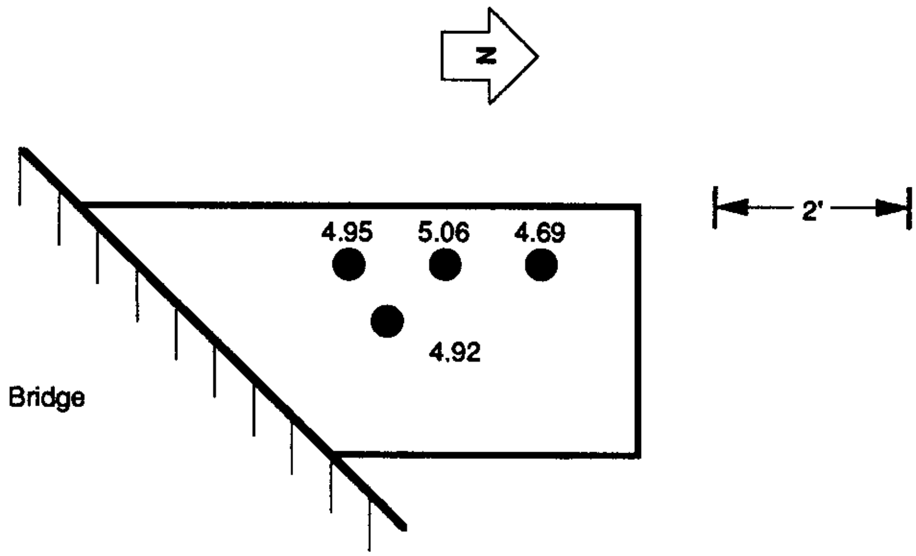


Figure 75. Moisture Content (in percent) of the Johnson Road Undercrossing

INTERPRETATION, APPRAISAL AND APPLICATION

SUMMARY OF EXISTING STATE OF KNOWLEDGE

The consensus of previous research is that there is no general cause of, and consequently no general solution to, bridge approach settlement, and that cases must be evaluated site by site. Concepts and techniques that perform satisfactorily for one site may not even be considered for another site. However, from the information procured from the literature search, the present knowledge of bridge approach settlement can be synthesized. The causes of bridge approach problems can be most conveniently described by one or more of three distinct components of bridge distress. These problems can be classified as differential settlement between the pavement and bridge deck, movement of abutments, and/or design-construction related problems. In this chapter, the current knowledge concerning each of these contributions to bridge approach distress is summarized.

Differential Settlement

Differential settlement between the pavement and bridge deck may be caused by compression of foundation soils, compression of embankment soils, and/or local compression near the pavement/bridge interface. This differential settlement is generally regarded as the most common source of bridge approach problems and may have many causes. A number of these are discussed in the following paragraphs.

Compression of Foundation Soils

One of the most common sources of approach embankment settlement is compression of the foundation soils beneath the approach embankment. Since many bridges necessarily cross soft, compressible soil deposits, such settlement is often inevitable. The movements resulting from the foundation soils can be caused by primary consolidation, secondary compression, and creep.

1. Primary Consolidation. Consolidation of the natural foundation soils beneath the approach embankments is often a prime contributor to bridge approach settlement. In permeable, granular foundation soils, initial and primary consolidation of the embankment foundation may occur before the pavement is placed. In such cases, the effect of primary consolidation on bridge approach problems is likely to be small. In other cases, however, settlement due to consolidation of foundation soils may continue long after bridge construction. In these cases, approach settlement due to primary consolidation of foundation soils is important, particularly when the bridge is pile-supported. Previous research has indicated that predicted ultimate settlement magnitudes are usually in reasonable agreement with observations when conventional settlement computations are performed on the basis of properly characterized soil conditions.

The predicted rates of settlement, on the other hand, have generally proved more difficult to accurately predict. The factors that affect the rate of foundation soil consolidation include the soil properties, layer thicknesses, embankment dimensions, surcharges, length of drainage paths, rate of construction of embankments, and stress history. The rate of embankment settlement due to primary consolidation has often been observed in the field to be much faster than predicted. Observed rates of settlement may be faster in the field than predicted rates because two-dimensional effects are generally not considered in settlement or because of sand lenses or sand seams undetected during the boring program. Test specimen disturbance during sampling and handling may influence the rate of consolidation observed in the lab. Uncertainty in stress history, initial void ratio, and consolidation parameters represent other problems in predicting rate of settlement. Settlement rate can also be difficult to predict if the deposit is thick and singly drained. In this situation, the top layer may experience secondary compression while the middle to lower portion is still undergoing primary consolidation. All of these factors can create difficulties in predicting consolidation rate behavior.

2. Secondary Compression. Another time-dependent component of settlement is secondary compression, which causes settlement to occur at constant effective stress. Secondary compression can be important for soft clays and silts and dominant for organic soils and peats. Data suggest that foundation soils with a coefficient of secondary compression as low as 0.007 are important in long-term foundation settlement. Secondary compression of foundation soil has been a significant factor in bridge approach settlement. Repeated, costly approach patching is the only alternative to settlement in the approach area because of continuing secondary compression.

3. Creep. Creep results in slow, continuous distortional straining under constant levels of shear stress. Soils that are subject to significant secondary compression often exhibit significant creep characteristics. Such soils typically exhibit a threshold shear stress below which creep deformations do not usually occur. When sustained shear stresses above the threshold shear stress are required for long-term equilibrium, creep strains may develop. Even though these strains occur relatively slowly, they can lead to significant deformations. Foundation soil creep is most important near the toe of the approach embankment, simply because shear stresses are usually highest in that area. Creep-induced lateral and vertical movements of the foundation soils underlying the approach embankment cause the approach pavement surface to settle.

Compression of Embankment Soils

Bridge approach settlement has also been attributed to movements within the approach embankment soils themselves. Such movements can result from either volumetric compression or distortional movements occurring at essentially constant volume.

1. Volume Changes. Compression of the embankment itself can result from volume changes within the soil mass. Factors known to influence embankment soil volume change include gradation, density, degradability, plasticity, organic content, strength and stiffness, environmental effects, and even traffic loading.

The gradation of embankment soils influences their strength, stiffness, and permeability. Well-graded soils have generally produced stiffer, stronger approach embankments, with a low potential for volume change.

Large volume changes have been observed in embankments constructed of degradable soils and rock. Degradable soils are deposited with weaker mineral grains, bonds, and structure. After these materials break down to finer portions, they are susceptible to erosion and compression. Volume changes from degrading soils or rock and reduced shear strength from the high content of plastic soil minerals can create large surface settlements. Significant creep is not uncommon in areas where shale or soils derived from shales are used for embankment construction.

Many times a borrow source has not been clearly identified as a troublesome soil. Volume changes from expansive clays may be erratic from cycles of swelling and shrinkage. Another soil type that may be contained in a borrow source is organic material. Soils of high organic content also have high void ratios and water contents. Volume changes in organics typically produce large compression, secondary compression, or creep in the embankment, depending on their proportion. Failure to recognize shrink-swell clays or organics in borrow material can lead to unexpected embankment settlements.

Volumetric changes due to frost action can also create serious problems in highway pavements. Volumetric expansion of water upon freezing is usually not uniform, and differential heave is likely to occur. Differential heave may be explained by the difference in thermal conditions that exist at material boundaries such as at bridge ends. Volume changes from frost action are believed to have been responsible for a number of bridge approach problems.

2. Distortional Movements/Creep of Embankment Soils. Distortional movements and creep within approach embankments have contributed to differential settlement at approaches to bridges. The potential for distortional movements of an embankment may be related to stability parameters such as embankment height, side slopes, shear strength, and compaction characteristics. Since the factor of safety is inversely related to the proportion of shear strength mobilized for equilibrium, the potential for creep induced distortional movements in an embankment increase as the design factor

for safety against stability failure decreases. One approach to minimizing the potential for such problems is to specify a minimum factor of safety sufficiently high to keep shear stresses below the threshold shear stress.

Local Compression at Bridge/Pavement Interface

Bridge approach settlement problems may be local, resulting from compression of materials close to the bridge abutment. Such local effects may result from inadequate compaction, drainage and joint sealing problems, rutting or distortion of the pavement section itself, or thermal bridge movements.

1. Inadequate Compaction Near the Abutment. Inadequate compaction near bridge abutments is one of the most commonly cited causes of bridge approach problems. The difficulty of operating large equipment close to abutments, particularly when the bridge is skewed to the roadway centerline, often results in inadequate, nonuniform embankment compaction. Large compaction equipment is driven as close as possible to the abutment when the approach embankment is compacted, but hand compactors are used to finish the remaining compaction next to the abutment and in other inaccessible areas. The large difference in compactive effort can cause nonuniform density patterns. One theory suggests that nonuniform compaction from traffic induced vibrations can lead to further densification and settlement of the granular soils underlying the approach slab. Hence, inadequate compaction near the abutment can lead to localized bridge approach settlement.

2. Drainage and Erosion Problems. Drainage of water from pavements and bridges has long been an important consideration in highway design. Water may enter from either the surface or subsurface, and when trapped, can lead to a number of problems for approach pavements and bridge abutments. Poor drainage in pavements and underlying basecourses can cause fines to be pumped into the basecourse layers, with a resulting loss of support. Lack of drainage around the bridge abutments and approach embankment slopes has been known to create serious erosion and piping problems. Significant erosion can undermine approach slabs or bridge abutments and cause very large movements. Piping problems often become evident when subsurface exploration encounters voids and weakened subgrade soils. If adequate drainage is not provided, pressures behind abutments may increase significantly, leading to abutment tilting and expansion joint closures.

3. Rutting/Distortion of Pavement Section. Many rough approaches to bridges have resulted from pavement rutting, cracking, and distortion near bridge abutments. In A.C. pavements, poor mix design and/or mix components can prematurely age the pavement surface. In P.C.C. pavements, poor aggregate, job mix and materials, insufficient curing, or overfinishing can accelerate surface deterioration.

Cracking is often caused by failed basecourse materials or differential subgrade movements in reaction to saturation. Edge cracking can often be explained by a lack of lateral support, while the larger longitudinal cracks in pavement often result from weak seams or poor bonds between adjacent pours or spreads.

Rutting, shoving, and channelizing can cause rough transitions onto bridge decks. Rutting generally develops from an accumulation of lateral movement of one or more of the underlying courses or further compaction and distortion of A.C. pavement under traffic loading. Plastic movements due to rutting is often indicated in front of approaches to bridges by localized depressions in the wheel paths with adjacent bulges in the pavement surface. Sunken approaches or approach faulting in rigid pavements can develop from inadequate load transfer and pumping of fines caused by the up-and-down motion of the slab under traffic loading.

4. Traffic Loading. Of great concern in highway engineering is traffic loading from wheel loads, tire pressures, and load repetitions. Frequent, heavy loads have been related to approach slab distress, from cracked slabs to slabs dropped off their paving notch. Problems arise when traffic volume exceeds the originally intended service design.

Traffic growth has accounted for numerous cases of failed approach pavements in highways throughout the the country.

5. Thermal Bridge Movements. The performance of approach pavements can be seriously impaired when expansion joints do not function as designed. The primary function of the expansion joint is to relieve stress from expansive, contractive, or frictional forces initiated by temperature, moisture, and slab-base interface properties. Joints are most susceptible to infiltration of incompressible materials during the colder season, when joints tend to open up. Incompressible materials can also infiltrate through poorly sealed joints and/or with base materials pumped into the joint. Joint sealing is influenced by the movement of the joint, the bonding between the side walls of the pavement slabs, and the characteristics of the joint. During warmer weather, clogged joints inhibit movement of the neighboring slabs and large pressures develop at the abutments. These large pressures displace the pavement and soil adjacent to the abutment, creating a gap when the bridge retracts. Subsequent traffic loading pushes the pavement and underlying soil downward and toward the abutment to fill in the gap. This movement results in abrupt differential settlement at the pavement/abutment interface. Maintaining a properly functioning expansion joint is imperative in reducing temperature-induced pressures and, consequently, pavement distress in bridge approach areas.

Movement of Abutments

Vertical abutment movements have been caused by settlement of supporting soils under abutments or downdrag on deep foundation elements. Vertical abutment movements can also occur from soil erosion beneath and around the abutment. Horizontal abutment movements have usually been attributed to excessive lateral pressures such as water pressure, thrust pressures from thermal bridge movements, or from swelling pressures caused by expansive soils, and by lateral deformation of embankment and foundation soils.

Design/Construction Problems

Approach settlement can also result from problems or errors in the design and construction of approaches and abutments. These problems can be related to the activities of the engineer, the contractor, and the inspector.

Engineer-related

Many bridge approach problems have resulted from specification of the wrong materials or techniques for a project. Most state highway departments have a set of standard specifications for bridge construction. These guidelines are general and are not tailored for a particular job. This generality is particularly likely to lend to problems in bridge abutment and foundation construction, where design is very site specific. Specifications should be updated and tailored for a particular project whenever possible to be most effective in producing satisfactory long-term performance of bridge structures and approaches.

1. Improper Materials. The choice and proper specification of embankment and basecourse materials can significantly influence the overall performance of approach pavements. Most highway departments specify certain ranges of material gradation for embankment construction. The material gradation should limit the percentage of fines to minimize embankment fill plasticity, which can influence the strength and overall stability of an embankment and lead to creep deformations. Basecourse and backfill materials should meet a higher standard of gradation, since drainage, as well as strength, is an important consideration. Acceptable subgrade and basecourse materials can reduce the incidence of unsatisfactory approach pavements. Material durability has been shown to influence approach embankment performance and should be addressed in material specifications.

2. Lift Thickness. Specification of a maximum lift thickness discourages excessive rates of fill placement, which can lead to poor compaction. The maximum lift thickness depends on the size and type of compaction equipment and on the maximum grain size of the fill. Typically, lift thicknesses range from 6 inches near abutments to 1 foot in the main embankment. Rock fills are generally placed in thicker lifts. The proper procedures for achieving maximum lift thicknesses for the abutment backfill and the main embankment should be specified to achieve suitable compaction within the embankment fill.

3. Compaction Requirements. Problems with relative compaction and the moisture content under which compaction is conducted are often primary causes of approach embankment settlement. If the compaction water content is not carefully controlled, the required relative compaction may be nearly impossible to achieve. Other problems can occur if the amount of compactive effort applied is not monitored. Overcompaction, or application of excessive compactive effort to a single lift, can make certain materials weaker. Better overall embankment performance may be achieved by specifying a range of placement water contents, as well as some minimum relative compaction. The desired approach embankment performance requires appropriate specifications for moisture contents and compactive effort.

Contractor-related

Bridge approach problems may also result from actions of the contractor. Inexperience with local conditions or with bridge construction can lead to construction difficulties for even the most conscientious contractor.

1. Improper Equipment. Adequate compaction of either the abutment backfill or approach embankment may be difficult to achieve when improper compaction equipment is used. A variety of compaction equipment is available to apply energy by one or more of the following methods: kneading action, static weight, vibration, or impact. The choice of equipment used in a compaction operation depends on the type of soil to be compacted. In general, cohesionless soils are efficiently compacted by vibration, whereas cohesive soils are most efficiently compacted by kneading compactors. Another important aspect is to utilize the appropriate size and mass of compactor for the desired work. Optimum construction procedures, such as number of passes, towing speed, and frequency of vibrator operation, can be determined in the field. Adequate compaction using appropriate compaction equipment can be crucial in achieving the desired performance of approach embankments and is particularly important for abutment backfill.

2. Overexcavation for Abutment Construction. Bumps on bridge approaches have even been observed when bridge ends were located in cut sections. The most likely cause of settlement in a cut section is overexcavation to facilitate abutment construction, with subsequent insufficient compaction of the backfill soils. Also, overexcavation can create a sloping plane from the bottom of the excavation, which can potentially become a slip surface. Overexcavation for abutment construction should be avoided when possible.

3. Survey/Grade Errors. Construction errors have been responsible for many reported cases of sudden grade change early in a bridge's life. Elevation differences can result from surveys that were not checked against "as constructed" structure elevations. Unsatisfactory start of the paver from the bridge deck, grade errors on guides used by paving machines, or manual screed adjustments can leave irregularities in the final grade. Built-in errors produce immediate bridge approach problems that my be difficult to correct.

Inspector-related/Poor Quality Control

One of the keys to construction quality for bridges and other structures is the availability of skilled inspectors during construction. The absence of such inspection has been linked, on many occasions, to poor bridge approach performance.

1. Lack of Inspection Personnel. Bridge approach problems are often associated with a lack of quality control and quality assurance activity during construction. Most state highway departments have field inspection crews on all highway construction projects; however, budget constraints often cause these crews to be understaffed. Thus, aggressive or continuous inspection may not always be possible to ensure quality control. A few interstate projects in recent years have subcontracted inspection duties to private inspection firms, in which case the inspector becomes an employee of the contractor. This conflict of interest has led to problems in aggressive inspection. Generally, quality assurance by experienced personnel is needed to ascertain that the construction is preformed to specification.

2. Improper Inspection Personnel Training. The fact that inspectors are on the job does not ensure that the project is built to specification. The inspector must be provided with the proper tools and equipment and must be thoroughly trained by an experienced inspector. Inspectors must be trained in communication skills and to act responsibly and reasonably. Inspectors must realize that no set of construction documents is perfect, and that errors exist in every set of construction documents. The department must support the inspector, and lines of communication must be kept open between the project engineer and the inspector. Properly trained inspection crews can reduce the number of "built-in" construction errors, which result in both immediate and delayed bridge approach problems.

MITIGATION OF BRIDGE APPROACH SETTLEMENT PROBLEMS

Introduction

The review of technical literature regarding bridge approach problems revealed a number of measures that can be used to reduce the occurrence of such problems. In their attempts to minimize bridge approach problems, highway departments have relied heavily on traditional aspects of bridge approach construction, occasionally incorporating newer concepts and materials into approach design. Although these new concepts have been employed with some success, bridge approach problems continue to occur throughout the country. A summary of standard methods and more recently developed techniques for mitigating bridge approach problems is presented in the following paragraphs. These methods and techniques are largely drawn from the experience and conclusions of previous investigators of bridge approach problems. Methods for mitigating bridge approach problems can be roughly divided into four categories: improvement in general operational methods, foundation soil improvement methods, embankment improvement methods, and abutment interface improvement methods.

General Operational Methods

Experience has shown that general operational methods can strongly influence bridge approach performance. When time or budgetary constraints do not limit their use, the operational methods described below have been shown to be effective in mitigating bridge approach problems. Many of these items may appear obvious; however, their neglect has resulted in many examples of bridge approach distress and has led to significant maintenance costs for many highway departments

Improved Site Investigations

One of the most universally recommended methods for reducing bridge approach problems is the use of more extensive and detailed site investigations. Many cases have been recorded in which actual subsurface conditions differed from those anticipated because of limited site investigations, and the result has been undesirable movements in the bridge approach area. Thorough site investigations can reduce the occurrence of bridge approach problems resulting from unanticipated or unknown subsurface conditions.

Strict Material and Construction Specifications

The potential for bridge approach settlement can, in many cases, be reduced by the use of strict material and construction specifications. Material characteristics such as gradation and durability can strongly influence the performance of an embankment fill. Strict adherence to proper construction specifications, such as lift thickness and compaction measures, can improve the overall strength and stiffness of an approach embankment and reduce its compressibility. Approach embankment settlement is less likely to occur where good quality fills are placed in appropriately sized lifts and are properly compacted to a specified percentage of the maximum dry density.

Construction Inspection

Construction inspection by competent field personnel can eliminate many bridge approach problems and contribute greatly to the satisfactory construction and performance of approach embankments. Many investigations of distressed bridge approaches have revealed problems due to construction errors that would have been caught by trained construction inspectors. Quality control is also important in soil improvement operations, since the adequacy of treatment should be verified before construction is allowed to continue. It is difficult to overemphasize the importance of proper inspection during the construction of bridge approaches.

Field Monitoring

Field performance monitoring can be very useful for evaluating the stability and settlement of approach embankments during and after construction. Stability can be monitored to avoid detrimental embankment failures, whereas settlement monitoring can be used to determine when final paving can proceed.

Foundation Improvement Methods

Poor foundation soil conditions have been responsible for unsatisfactory bridge approach performance in many locations across the country. A number of techniques have been used to improve foundation soil conditions. Appropriate use of these methods has been shown to improve bridge approach performance.

Removal and Replacement

A common method of foundation soil improvement involves removing incompetent soils and replacing them with compacted backfill that meets desired material specifications. Removal and replacement are typical for organic deposits and very soft clays, where stability and excessive settlements are a common problem. If the basement topography slopes, benches should be constructed to provide a horizontal contact for the newly placed fill. The depth of excavation, excavation methods, disposal of the excavated materials, and environmental factors often influence the feasibility of this treatment.

Improvement by Densification

Physical densification techniques are sometimes utilized where loose granular deposits are anticipated to settle when loaded. Physical densification techniques include dynamic compaction, blasting, and vibroflotation. Dynamic compaction densifies loose deposits by repeatedly dropping heavy weights on the ground surface above them. Blasting densifies sands by vibrating them with detonations from buried explosives. Vibroflotation densifies granular materials by inserting a vibrating probe, which contains water jets, into the deposits. Highway departments have used all of these techniques to produce good results under the appropriate conditions.

Improvement by Grouting

Compressible soils can be stabilized with grout injections or soil mixing techniques, but highway departments do not normally used these techniques. Grouting can be performed with different types of grout and a variety of techniques: slurry, compaction, chemical, or jet grout. Each of these techniques improves the performance of a deposit by altering their physical properties.

Improvement by Inclusions

Improving foundation soil with inclusion usually involves constructing vertical stone/sand or lime columns. Improvement by inclusion increases shear strength, reduces settlements, and may reduce the liquefaction potential. To construct lime columns, engineers drill into the compressible deposit and refill it with a lime-soil mixture as the auger is withdrawn. To construct sand/stone columns, a vibratory probe with water jets is inserted into the ground, and a sand/stone mixture is introduced into the depression that forms at the ground surface. Timber piles, another type of inclusion, have been driven into foundation soils to support approach embankments. Pile supported approach embankments may provide smooth transitions but do not always eliminate the differential settlement at bridge ends. The availability of timber piles or installation costs for construction of piled embankments make this solution prohibitive in many cases.

Improvement by Surcharging

Compressible soils may be improved by preloading or surcharging the foundation soils to allow time for consolidation settlement. Consolidation settlement of soft clays may be accelerated with prefabricated vertical drains. Vertical drains allow the water forced from the voids of a saturated soil to drain more readily, thus shortening the time required for consolidation.

Embankment Improvement Methods

Many cases of unsatisfactory bridge approach performance have been attributed to problems within the approach embankment itself. Many of these cases could have been avoided by use of embankment improvement methods such as those described in the following sections.

Selection of Embankment Materials

The characteristics of embankment materials can strongly influence the overall behavior of the approach embankment. Important material characteristics that contribute to the behavior of the embankment include plasticity, durability, gradation, and shrink-swell. The plasticity of fill soils controls the potential for creep. Creep movements, although very slow and often not perceptible, can lead to approach distress over time. If the fill soils are not durable, they slowly weather, break down, and compress, leading to substantial surface settlement. Fill containing rocks such as shales and weaker sandstones are susceptible to deterioration. As with the importance of durability, the gradation also contributes to the performance of an embankment fill. Gradation controls the density, strength, and stiffness of the fill soils in a compacted embankment. A well-graded soil compacts more densely since smaller particles fill the voids around the larger particles. The gradation also controls the susceptibility to frost action. Fill soils must also be tested for shrink-swell capabilities. Since embankments are exposed to both wetting and drying conditions, soils that display shrink-swell behavior must not be included as a borrow source for embankment fill. Lightweight embankment fills are considered for some projects where a reduction in the embankment load may reduce settlements and increased stability. Lightweight fills come from a variety of sources and are effective, since their compacted unit weights are less than that of ordinary embankment fills.

Embankment Slopes

Embankment slopes contribute to approach distress when their factor of safety is less than 1.5, particularly when highly plastic embankments or foundation soils are involved. In such cases, sustained shear stresses may be large enough to induce creep deformations in susceptible materials. These creep deformations have been responsible for approach settlement in a number of cases cited in the literature. Stabilizing embankments may involve flattening their side slopes and thus reducing shear stresses below creep threshold values. However, total settlements may also be increased, and, in some cases, flatter slopes may not be practical because of right-of-way or economic constraints.

Surface Water Control

Surface water must be controlled to avoid detrimental erosion in and around the abutment and embankment slopes. Many approach embankments are designed with catch basins to catch water runoff before it enters the bridge or to catch water as it leaves the bridge deck. Catch basin, culverts, and trenches must be maintained so that they function as designed. Side slopes and end slopes should be protected by concrete slope protection, retaining walls, geotextiles-woven fabrics or geogrids, rip-rap, or vegetation. Some state agencies have developed more elaborate drainage schemes where surface drainage is particularly important. Even though the crown or super-elevation of the highway may have changed from years of rehabilitation work, water must be trapped and effectively drained away from the abutment and the approach embankment.

Embankment Reinforcement

Reinforced earth embankments have worked well in controlling differential settlement in a limited number of studies of their use in bridge approaches. Reinforced earth embankments can be used with either deep or shallow foundations. Geosynthetic reinforcement, which has usually been used for temporary embankment construction in the past, has now been adopted for permanent highway functions in some states. The Wyoming Highway Department has been investigating the feasibility of this type of approach embankment since 1987. In their design, a free-standing, geotextile reinforced wall is constructed behind the abutment. This geotextile wall reduces the lateral forces imposed on the abutment backwalls by the embankment soils and subsequently reduces differential settlement of approach slabs. Reinforcing fabric may also be placed on top of soft foundation soils before embankment construction to distribute embankment loading and prevent the underlying low strength soils from heaving.

Abutment Interface Improvement Methods

Poor bridge approach performance resulting from localized soil and pavement movements in the area near the interface between the pavement and the bridge abutment has long been known to cause bridge approach problems. Even in cases where foundation and embankment soil movement can be eliminated, localized movements can lead to significant problems. In many respects, such localized movements are among the most important of the mechanisms leading to bridge approach distress because they can cause approach problems for which expensive improvement methods, e.g., approach slabs, may not be necessary. Careful attention to design and construction considerations at the bridge/soil interface can eliminate the need for approach slabs in many cases.

Expansion Joints

Expansion joints have frequently been associated with poor approach slab performance. Infrequently maintained expansion joints easily become clogged with debris and lose their effectiveness. Many of today's newer, compression seal expansion joints allow contraction and expansion of the bridge superstructure while effectively sealing the joint from debris.

Drainage Provisions

Drainage provisions are imperative for preventing erosion and avoiding embankment saturation that can lead to pumping, piping (removal of finer grained particles), and increased frost action in cold weather climates. These conditions can weaken the subgrade and increase lateral earth pressures on abutment backwalls. Consequently, embankments should be designed with careful consideration of both surface and subsurface drainage. A free draining backfill is crucial and may be modified with piping systems and/or geosynthetics. A Caltrans design specifies select granular backfill enclosed in a geotextile filter fabric for use behind, under, and in front of the abutment. Perforated pipe is installed in the select backfill region and is drained outside the limits of the embankment.

Abutment Detail

Abutment design detail dictates the constructibility and function of the bridge-approach/embankment unit. The design and shape of many abutments, including those of WSDOT bridges, hamper compaction in some areas. Compaction in constricted areas, for example areas near structural elements such as abutments and wingwalls, is generally accomplished with small, hand-operated equipment. The difficulty of achieving good compaction under such conditions is well-recognized by geotechnical engineers. Abutments that are constructed with a protruding pavement seat and supporting corbel leave areas inaccessible to compaction equipment and invite localized settlement problems.

Specific abutment designs can alleviate the bridge approach settlement problem. The use of shallow foundations with simply supported end spans has reduced differential settlements. Integral abutments have resisted abutment movements more readily than standard abutment configurations. Cellular, or hollow, abutments can be used to reduce the weight of the abutment on the approach fill. Longer bridges, with deep foundations, are an expensive option for crossing problem soil deposits. Bridges designed with cantilevered end spans have been used to eliminate the need for abutment support, but they do not address the embankment settlement problem.

Approach Slabs

Reinforced concrete approach slabs are used by most states to alleviate the occurrence of bridge approach distress due to changes in the underlying soils. The concept of the approach slab is to provide a transition between the earth approach embankment and the bridge structure. One end of the slab rests on the abutment, while the other end is situated on the embankment. The rigid approach slab is designed to "bridge" materials that may be inaccessible to large compaction equipment because of their close proximity to the abutment. Some researchers have noted that approach slabs have not always solved the problem of differential settlement.

Approach slabs constructed by state departments of transportation are not uniform in length, thickness, reinforcement, hinge detail, footing detail, or bridge skew. Surveys conducted by various investigators show that the length of single slabs range from 10 ft. to (in one case) 120 ft., with an average of 40 ft. One investigation (Stuart, 1985) concluded that slab length does not have an affect on the overall settlement. However, shorter slabs (10-15 ft.) are more cost effective and do exhibit better vehicle ride characteristics than bridges without approach slabs. A 1989 Caltrans design specified a 30-ft. long approach slab, which rests on a sleeper slab and is integrally cast with a 15-ft. long pavement section. Another concept that has been developed by a few transportation offices incorporates two separately cast approach slabs joined by an underlying sleeper slab. The newest modification implements inflatable bladders between the two approach slabs and the sleeper slab. The slabs can be inflated or deflated in accordance to grade requirements. A more flexible approach slab is in the design stages at several highway offices. Precambered approach slabs have been designed to achieve a smooth ride after smaller post-construction settlements have taken place. Some states realize that approach slab settlement is inevitable and design preformed grout holes and keyways to safely assist in slab lifting via mudjacking. Still others cast threaded holes in the slabs for subsequent mechanical lifting.

Special Abutment/Approach Slab Treatments

Special compaction measures and materials have been utilized adjacent to abutment backwalls to provide extra strength in this area of potential deformation. Some state agencies have used special backfill materials such as stabilized bases or asphalt beams with geomembrane linings adjacent to the abutment. Ground anchors have been used in the embankments to minimize approach slab movement. A 1989 Caltrans design used select backfill extending 150 ft. from the end of the structure plus a fabric "bond breaker" between the approach slab and a 6-inch treated permeable base.

Jacking Systems

A number of interstate bridges have been designed with concrete jacking pads built along side the beam seats. These jacking systems have been typically attached to spread footing abutments where bridge settlements have been greater than the adjacent approach embankment settlements. Jackable abutments are useful when settlement is expected to continue after construction. When such bridge approaches become too rough, hydraulic jacks are brought in to lift the bridge back to a grade at which it can be shimmed. Design changes such as simple spans or larger girders can also be incorporated to accommodate for larger bridge settlements.

Temporary Pavements

Some highway departments have used temporary pavements to eliminate post-construction settlements that have occurred at bridge approaches. The idea is to pave across the bridge deck and then rotomill the pavement flush to grade when the settlement has stabilized. Replaceable precast concrete approach sections have been proposed but have not been used by any state transportation office.

Maintenance

When differential settlement between the bridge deck and approach slab occurs, the easiest and sometimes the only solution is to place a smooth patch of asphalt concrete over the deformed area. Approach patching is effective and provides a smooth transition onto and off of the bridge. Overlays are smoother when extended the full length of the bridge and a sufficient distance ahead of the bridge ends. Sometimes, when primary or secondary settlements are not complete, periodic patching is needed to maintain the smoothness of the driving conditions. Asphalt cement overlaying is the most commonly used method of patching approaches, but it may be costly if repeated maintenance is required to maintain a smooth approach. Bonded concrete overlaying has also been utilized in some states.

Slab jacking (mudjacking) of approach slabs is widely practiced and is usually successful in restoring the original grade. Slab jacking involves inserting grout tubes into holes drilled in the approach slab and pumping the grout under pressure to lift the slab to the desired elevation. Polyurethane foams have also been proposed for lifting slabs. If approach slabs are cast separately for each lane, work can proceed without shutting down the entire roadway. Hydraulic rams or mechanical jacks can also be used to temporarily raise the slab until undergrouting can be performed. Complications do occur, however, and slab jacking is not a flawless procedure. Reports indicate that slabs have been cracked and broken from mudjacking operations.

Research indicates that some short bridge spans do not generate enough movement to justify the construction of an expansion joint. On some short span bridges where the slabs have remained intact and the expansion joints have been identified as the source of problems, the joints have been completely removed and concrete has been poured in its place.

In addition to the actual maintenance performed at a given site, meticulous maintenance records should be logged for each approach slab maintenance activity. This maintenance log can be used for cost comparisons and statistical purposes, or it can be used to determine which foundation soil treatment might have been applied with better results.

CONCLUSIONS AND RECOMMENDATIONS

The use of approach slabs has proved to be an effective method of improving vehicle ride characteristics in areas where differential settlement between bridge approaches and bridge abutments has developed. Approach slabs have been used by nearly every highway department in the United States and are very commonly used overseas as well. They have a long record of generally successful performance in reducing the detrimental effects of even very large amounts of differential settlement. Clearly, approach slabs are, and will continue to be, an important consideration in the design and construction of highway bridges.

However, approach slabs can add to the constructed cost of a highway bridge. Construction costs associated with design, materials, labor, and construction delays must be considered in evaluating their overall effectiveness. In addition, not every approach slab works as well as intended. Structural damage to approach slabs has been reported and can be difficult and expensive to repair. Occasionally, the installation of an approach slab simply moves the bump from the edge of the bridge to the edge of the approach slab. Approach slabs are not needed on all bridges, and their elimination on certain bridges may reduce costs considerably.

CONCLUSIONS

Development of a rational policy for the use, or non-use, of approach slabs requires an understanding of the potential causes of differential settlement between bridge approaches and abutments and knowledge of the methods available to reduce that settlement. Numerous previous investigations have shown that, while bridge approach distress is very common, there is no single cause to which it can be attributed. Rather, each reported case has had its own peculiar characteristics or combinations of characteristics which have contributed to the observed distress. Consequently, it is unreasonable to expect that a standard bridge approach design will perform satisfactorily in all cases, or that a rigid state-wide or even district-wide policy on the use of approach slabs will lead to the best use of resources.

CAUSES OF BRIDGE APPROACH DISTRESS

Bridge approach distress generally results from differential settlement between bridge approaches and abutments. The causes of such differential settlement are numerous and were summarized in a previous section of this report. The sources of differential settlement can be broadly divided into three categories: compression of foundation soils, compression of embankment soils, and local compression of materials near the approach/abutment interface.

Compression of foundation soils occurs in many bridge approach conditions highway designers encounter. Methods for predicting such settlement are fairly well developed, and past experience has shown that, given the resources necessary to adequately characterize geotechnical conditions at such sites, engineers are generally able to predict settlement magnitudes with adequate accuracy. Because rates of settlement are influenced by many factors that may be difficult to detect at a particular site, prediction of settlement rates is more difficult than settlement magnitudes. Distortion of foundation soils because of secondary compression and creep can also lead to bridge approach distress, particularly when the foundation soils are highly plastic or contain organics. Nevertheless, it is possible at most bridges to evaluate the contribution of foundation soil compression to differential settlement at the approach/abutment interface. Differential settlement at bridge abutments supported on footings founded in the approach embankments is usually not strongly influenced by foundation soil compression.

Embankments are generally constructed of compacted fill that has been subjected to some material quality specification. Desirable embankment soils are generally well-graded, non-plastic granular soils composed of durable materials. These soils should be placed in relatively thin layers, moisture conditioned, and adequately compacted. Years of experience across the country have shown that embankment fill materials with these desirable characteristics may not be readily or economically available at all bridge sites, and that the actual construction process often deviates from the process that has been specified. The use of undesirable embankment fill materials can contribute to differential settlement in a number of ways, as can insufficient compaction of the materials that are used. Typical embankment and compaction specifications tremendously reduce the incidence of embankment compression. Therefore, unexpected differential settlement due to embankment compression is often predominantly an inspection or quality control problem. Again, the influence of embankment compression on differential settlement at abutments supported on footings in an embankment is generally lower than for pile-supported abutments.

Local compression of soil and pavement materials is a common cause of differential settlement at approach/abutment interfaces and is thus a critical issue in the formulation of a rational approach slab policy. The soils immediately beneath the pavement adjacent to the abutment are generally placed after the main portion of the embankment and the abutment have been constructed. These soils are placed in a local excavation too small for compaction with conventional compaction equipment and are therefore compacted with small, hand-operated compactors. They are generally placed and compacted directly against the abutment. Many previous studies, and the field investigation conducted as part of this research project, have shown that inadequate compaction of the soils adjacent to bridge abutments causes densification and distortion that manifests itself as abrupt differential settlement at the approach/abutment interface. This mechanism has been observed in conjunction with compression of foundation and embankment soils, but also by itself at sites where no foundation or embankment soils are present. Elimination of local compression near bridge abutments is an important consideration in the development of a rational approach slab policy.

Mitigation of Bridge Approach Distress

Prevention of bridge approach distress generally involves elimination, or at least minimization, of differential settlement at approach/abutment interfaces. Available measures for reducing such settlement were summarized in a previous section of this report.

The mechanisms that cause foundation soil settlement are generally well understood. Foundation soil settlement can be reduced with a number of proven, conventional soil improvement methods, and these methods are used regularly by WSDOT and other highway departments. One source of approach/abutment differential settlement that often goes unrecognized is creep deformation induced by sustained shear stresses imposed on the foundation soils by the overlying approach embankment. Creep occurs most commonly in highly plastic and/or organic soils subjected to shear stresses greater than about one-half their strength. By recognizing the creep problem and reducing creep-inducing shear stresses by flattening embankment slopes, highway agencies can reduce the incidence of bridge approach distress at many sites.

Differential settlement at approach/abutment interfaces resulting from compression of embankment soils can most easily be reduced by strict adherence to embankment fill material and compaction specifications. Communication between the embankment design engineer, the construction inspector, and the contractor is a key to achieving satisfactory embankment performance. The design engineer must emphasize the importance of material and compaction requirements to the inspector before construction. The inspector must inform the design engineer of deviations from specifications in a timely manner so that their influence on the performance of the embankment may be evaluated. Unfortunately, staffing and organizational constraints may hinder such exchanges on some projects.

Compression of soils near abutments is a problem whose solution is both easy and difficult. The results of previous investigations, and of the field investigation undertaken in this research, strongly indicate that inadequate compaction of soils placed as backfill in abutment construction excavations is the primary cause of localized approach/abutment settlement. The difficulty of compacting soil in such tight quarters has long been known, but in many cases is exacerbated by the design of the abutment itself. Many WSDOT bridges, for example, are constructed with abutments featuring pavement seats consisting of 6- to 8-inch-wide ledges located at depths of 12 to 15 inches below the pavement surface. These pavement seats are probably effective in reducing the abruptness of differential settlement, but many are configured in a manner that renders adequate compaction of abutment backfill soils virtually impossible. Elimination of the sloping face of the abutment backwall below the pavement seat, with which a void was associated in every case found in this field investigation, would go far toward reducing local compression effects on approach/abutment settlement. Modification of this abutment detail represents an easy solution to one of the problems contributing to local settlement. The solution for the other main aspect of the local settlement problem is more difficult. It requires improved inspection and testing of abutment backfill compaction and/or the modification of material and compaction specifications for abutment backfills. Improvements in the availability of construction inspectors may be hindered by budgetary constraints, but modification of construction specifications can be undertaken.

APPROACH SLAB USE AND EFFECTIVENESS

The differential settlement conditions leading to the use of approach slabs vary from site to site; however, some general conclusions on their use can be drawn. Approach slabs have been shown to provide improved vehicle ride characteristics at bridge approaches subject to differential settlement at the approach/abutment interface. The level of differential settlement required to cause poor vehicle ride characteristics is so small that it is nearly impossible to prevent at most sites. Thus, at these sites, geotechnical conditions virtually require that approach slabs be used.

Sites with approach embankments on compressible foundation soils experience settlement, and approach slabs are very likely to be required, particularly when the bridge abutments are pile-supported. Sites at which marginal embankment fill materials are used or at which they are inadequately compacted are very likely to require approach slabs. In fact, the results of this research investigation indicate that approach slabs should be strongly considered for almost all bridges, but that there are some exceptions.

Elimination of bridge approach slabs should be considered for sites with certain geotechnical conditions. These sites should have foundation soils for which long-term settlement due to consolidation, secondary compression, and creep will be insignificant. They should also require a very low approach embankment or no approach embankment at all. At such sites, two of the primary sources of approach/abutment differential settlement are not significant, leaving local compression of the soils near the abutment as the remaining source of approach distress. When these soils are placed properly, differential settlement should be small enough to render approach slabs unnecessary. However, with the abutment detail encountered in a number of bridges in the field investigation of this research abutment backfill soils could not be placed properly and approach distress was observed. Modification of this particular abutment detail, and increased attention to the abutment backfill process, should allow bridges at such sites to perform successfully without approach slabs.

RECOMMENDATIONS

The long history of successful approach slab performance in Washington and other states suggests that their regular use should be continued; indeed, there are many situations in which geotechnical conditions virtually require their use. However, absolute requirement of approach slabs on all bridges is unwarranted and would result in excessive design and construction costs on a number of bridges. To avoid wasting construction funds on approach slabs, it is necessary to develop a rational policy for their use or non-use. The authors of this report do not pretend to know all about the administrative, contractual, personnel, and other factors that will undoubtedly influence an eventual approach slab policy. The results of the research, and the recommendations that follow, are intended to provide a technical perspective for guidance in the development of such a policy.

The following recommendations should be considered in the development of policy for the use, or non-use, of approach slabs.

1. Approach slabs should not be specified for all bridges. There are sites at which approach slabs are necessary, but also sites at which they are not necessary and at which their use adds excessive cost to the bridge.
2. The successful performance of bridge approaches depends on good engineering design and good construction quality, whether or not approach slabs are used.
3. The decision to use approach slabs should be made on a site-specific basis.
4. The design geotechnical engineer should recommend whether approach slabs are required at a specific bridge site on the basis of the geotechnical conditions present at the site. Construction, maintenance, or other considerations may eventually overrule the geotechnical recommendation, but the initial evaluation should be made on the basis of geotechnical information.
5. The design geotechnical engineer should be provided with adequate resources to properly evaluate the need for approach slabs at a particular site.

6. For bridges at which approach slabs are determined to be unnecessary, the following steps should be taken:

   a. The abutment detail should be reviewed to ensure that no overhanging ledges are present to produce zones in which compaction will be impeded. An abutment detail similar to that shown in Figure 76 should be specified.

   b. The abutment backfill should be constructed of select granular fill material that meets the standard WSDOT specification for good quality granular material. The use of controlled density fills for abutment backfill should be investigated.

   c. The abutment backfill should be compacted to at least 95 percent relative compaction.

   d. Continuous inspection of the placement and compaction of abutment backfill should be provided.

Some of these measures may add to the cost of bridges without approach slabs; however, the additional cost should be minimal in comparison to the cost of approach slabs or the cost of repeatedly repairing a distressed bridge approach over the life of the bridge.

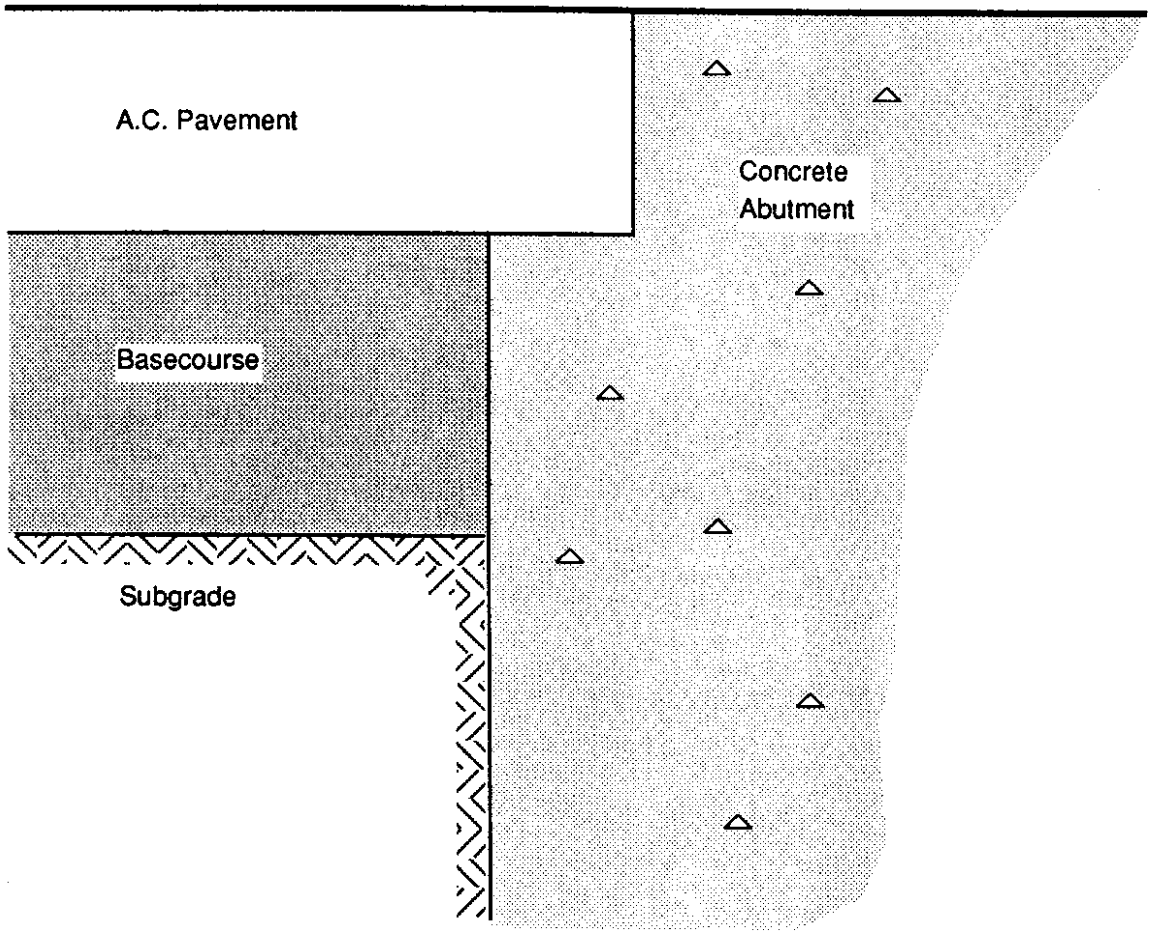


Figure 76. Schematic Illustration of Suitable Abutment Profile

IMPLEMENTATION

The research investigation described in this report identified geotechnical conditions for which the use of approach slabs may not be warranted. It also found that many existing cases of approach distress at bridges constructed without approach slabs could likely have been prevented by the use of a modified bridge abutment detail. Abutments with overhanging ledges were found in field investigations to have aided the development of voids between the abutment and the abutment backfill soil below the ledge. These voids resulted in sufficient differential settlement at the approach/abutment interface to cause poor vehicle ride characteristics and require repeated maintenance.

Recommendations for modification of bridge abutment details to eliminate overhanging ledges, and to modify abutment backfill material and compaction specifications, should be addressed as soon as possible. Implementation of these recommendations will allow the successful performance of bridges constructed without approach slabs and should result in substantial design, construction, and maintenance cost savings.

ACKNOWLEDGMENTS

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A great deal of assistance was provided to the researchers by various WSDOT personnel. Al Kilian of the Materials Laboratory and Keith Anderson of the Research Office were instrumental in developing the research plan and monitoring of the project progress. Invaluable assistance in organizing and conducting the field investigations was provided by Chuck Vanderpool of District 3, Jack McLean of District 2, Guy Couture of District 5, and their crews. Grateful acknowledgment is also extended to those who provided the researchers with the benefit of their experience and perspective by completing the bridge approach survey.

REFERENCES

Jones, C.W. (1959). "Smoother Bridge Approaches," *Civil Engineering*, Vol. 29, June, pp.407-409.

Stermac, A.B., Devata, M., Selby, K.G., (1968). "Unusual Movements of Abutments Supported on End-Bearing Piles," *Canadian Geotechnical Journal*, Vol.2, No.2, May, pp.69-79.

Hopkins, T.C., Deen, R.C. (1969). "The Bump at the End of the End of the Bridge,"*Highway Research Record,#302*, Division of Research, Kentucky Department of Highways, June, pp.72-75.

Hopkins, T.C., Scott, G.D. (1969). "Estimated and Observed Settlements of Bridge Approaches," *Highway Research Record #302*, Division of Research, Kentucky Department of Highways, June, pp. 76-86.

Timmerman, D.H. (1976). "An Evaluation of Bridge Approach Design and Construction Practices," *Report No. OHIO-DOT-03-77*, University of Akron, Akron, December, 58 pp.

Dimillio, A.F. (1982). "Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," *Report No. FHWA/RD-81/184*, Federal Highway Administration, Washington, D.C., October, 54 pp.

Hopkins, T.C. (1985). "Long-Term Movements of Highway Bridge Approach Embankments and Pavements," *Report No. UKTRP-85-12*, Kentucky Transportation Research Program, Lexington, April, 149 pp.

Stewart, C.F. (1985). "Highway Structure Approaches," *Report No. FWHA/CA/SD-85-05*, California Department of Transportation, Sacramento, California, July, 158 pp.

Allen, D.L. (1985). "A Survey of the States on Problems Related to Bridge Approaches," *Report No. UKTRP-85-25*, Kentucky Transportation Research Program, Lexington, October, 188 pp.

Laguros, J.G. (1986). "Evaluation of Causes of Excessive Settlements of Pavement Behind Bridge Abutments and Their Remedies - Phase I," ODOT study No. 84-12-2, Item 2140 ORA 155-857, Office of Research Administration, University of Oklahoma, Norman, January, 76 pp.

Hopkins, T.C. (1986). "Stability of Embankments on Clay Foundations" *Report No. UKTRP-86-8*, Kentucky Transportation Research Program, Lexington, April, 72 pp.

Ardani, A. (1987). "Bridge Approach Settlement," *Report No. CDOH-DTP-R-87-06*, Colorado Department of Highways, Denver, April, 53 pp.

Wolde-Tinsae, A.M., Aggour, M.S., Chini, S.A., (1987). "Structural and Soil Provisions for Approaches to Bridges," *Report No. FHWA/MD-89/04*, Maryland Department of Transportation, Baltimore, July, 105 pp.

Edgar, T.V., Puckett, J.A., Sherman, W.F., Groom, J.L., (1987). "Utilizing Geotextiles in Highway Bridge Approach Embankments," *Geotextiles and Geomembranes* (5) 0266-1144/87, pp. 3-16.

Wahls, H.E. (1989). "Bridge Approach Design and Construction Practices," Project 20-5, Topic 18-03, Transportation Research Board, Washington, D.C., July, 127 pp.

Edgar, T.V., Puckett, J.A., Rodney, B.D. (1989). "Effects of Geotextiles on Lateral Pressure and Deformation in Highway Embankments," *Report FHWA-WY-89-001*, Wyoming Highway Department and The University of Wyoming, Laramie, July, 229 pp.

Tadros, M.K., Benak, J.V., (1989)."Bridge Abutment and Approach Slab Settlement," Final Report-Phase I, University of Nebraska, Lincoln, December, 89 pp.

APPENDIX I

BRIDGE APPROACH SURVEY

Appendix I

Bridge Approach Survey

This questionnaire is being distributed as part of a WSDOT-sponsored research project investigating the effectiveness of bridge approach slabs in Washington. We would appreciate your filling out the survey and returning it in the stamped, addressed envelope as soon as possible.

Approach slabs are generally used to minimize "the bump at the end of the bridge" in order to improve driver comfort and safety. The objectives of this project are to develop a rational decision process for the use of approach slabs and to develop improved approach slab designs. An important part of the project is to determine the extent of existing bridge approach settlement problems in the state. To do that, we've decided to ask the experts by means of this survey. We would like to get some examples of cases where the use of approach slabs has worked and also of cases where they haven't worked. We'd also like to get examples of bridges where approach slabs haven't been used with both good and bad results. Please give your honest answers and opinions. If you are uncomfortable with or are unable to answer any of the questions, just leave them blank and go on.

Individual responses will be kept confidential, so feel free to express any opinions you have on any topic relating to the survey. In order to judge how well the survey is covering the state, though, we need the following information:

Name (optional): ______________________

Title: ______________________ District: __________

Geographic areas your responses cover: ______________________

---

1. Is differential settlement between approach pavements and bridges a common problem in your district? Please rate from 1 (very common) to 5 (very unusual).

   Rating _______

2. In your district, what percentage of bridges use approach slabs? (Circle best answer)

   0-20% 20-40% 40-60% 60-80% 80-100%

3. In your district, what percentage of bridge approaches need periodic maintenance to reduce differential settlement problems?

   Bridges without approach slabs

   0-20% 20-40% 40-60% 60-80% 80-100%

   Bridges with approach slabs

   0-20% 20-40% 40-60% 60-80% 80-100%

4. On the average, how long does it take for significant differential settlement to develop after completion of construction?

   0-1 yr 1-2 yrs 2-3 yrs 3-5 yrs other _____

5. On the average, how often is maintenance necessary for bridges which develop problems?

Bridges without approach slabs

0-1 yr   1-2 yrs   2-3 yrs   3-5 yrs   other _____

Bridges with approach slabs

0-1 yr   1-2 yrs   2-3 yrs   3-5 yrs   other _____

6. How is maintenance typically performed?

___ Pavement overlays
___ Slab jacking
___ Other - please specify: ____________________
____
____

7. The type of abutment foundation can influence the amount of differential settlement at bridge approaches. In your district, what percentage of bridges with the following types of bridge foundations develop differential settlement problems?

Shallow foundations (footings)

0-20%   20-40%   40-60%   60-80%   80-100%

Deep foundations (piles, drilled shafts)

0-20%   20-40%   40-60%   60-80%   80-100%

• Don't know foundation types

8. There are a number of possible causes of differential settlement at bridge approaches. Based on your experience with bridges in your district, rank the following possible causes from 1 (very likely) to 5 (very unlikely). Since there can be more than one cause of differential settlement at a site, more than one can be rated as very likely.

Settlement of foundation soils beneath approach embankment ____
Overall compression within approach embankment fill ____
Local compression (within 3-4 ft of bump) of embankment fill
    Due to poor compaction ____
    Due to poor embankment fill soils (excessive fines, etc.) ____
    Due to water infiltration ____
Distortion of pavement section (rutting, etc.) ____
Erosion of embankment soils ____

Comments: ____________________
____
____

9. In your opinion, what measures could be taken to improve compaction in tight quarters adjacent to abutments, approach slabs, etc.?

____
____
____

In the next few sections we would like to get examples of actual WSDOT bridges both with and without approach slabs where approach settlement problems (bumps) have and have not developed. Bridges can be identified by bridge number, milepost, exit number, cross-street, etc. We will use this information to select some of these bridges for further study.

No Approach Slabs - Good Performance

10. Please list any bridges you are aware of where approach slabs were not used and where the resulting performance has been good (significant maintenance not required).

No Approach Slabs - Poor Performance

11. Please list any bridges you are aware of where approach slabs were not used and where the resulting performance was poor (significant maintenance <u>was</u> required).

Approach Slabs Used - Good Performance

12. Please list two or three bridges you know of where approach slabs were used with good success.

Approach Slabs Used - Poor Performance

13. Please list any bridges you are aware of where approach slabs were used but where their performance was poor.

14. In what percentage of bridges with approach slabs does the use of the approach slab simply move the bump from the end of the bridge to the end of the approach slab?

0-20% 20-40% 40-60% 60-80% 80-100%

15. In your opinion, what is the cause of bumps at the end of approach slabs?

____

____

____

____

16. What factors do <u>you</u> think should be used to determine whether approach slabs should be used at a particular bridge site?

____

____

____

____

____

17. Please list any other comments you might have regarding the use and effectiveness of bridge approach slabs.

____

____

____

____

____

____

____

Thanks very much for your cooperation and assistance in completing this survey. The completed survey should be placed in the stamped, addressed envelope and returned to:

Professor Steve Kramer
265 Wilcox Hall, FX-10
University of Washington
Seattle, WA 98195

APPENDIX II

ROAD VIDEO LOG

APPENDIX II
ROAD VIDEO LOG

Preliminary work for this research project involved identification of problematic bridge approaches on Washington State Highways that could warrant further investigation. One simple, yet very effective method of reviewing a number of highway bridge approaches without physically driving to each individual site, was to analyze the bridge crossings by viewing a road video log. The WSDOT materials laboratory houses a library of road video logs that cover a significant portion of the highways it manages. By viewing the videos of the highway sections, pertinent information, such as the presence of approach slabs or approach patching and the approach pavement type, could be readily obtained. In addition, the amount of camera deflection-produced when the vehicle-mounted camera crosses the bump at the bridge ends-provides a good indication of the severity of the bump. On this basis, a rating system was established in which a bump could be classified. The bump was rated on a scale of 1 (no observable bump) to 5 (significant bump). It was not possible to rate all bridge crossings. Some bridges were paved over and others were filmed in the early morning hours on an overcast day; these conditions made it difficult to interpret approach slabs and patches.

The table below is the result of watching several road video logs. Columns of classification are listed under both headings: "Approach Slab" and "Patches." In the columns, "Y" for yes and "N" for no, is the convention used. Any additional comments were also noted in the comments column. The bridges analyzed here were suggested by DOT personnel due to known soil conditions at the bridge site and the related maintenance activities:

- SR16-Tacoma to Bremerton
- SR167-Renton to Puyallup
- SR 82-Tri-Cities to Yakima
- SR 5-Vancouver to Kelso
- SR 90-Ritzville to Grant/Adams Co. Line

SR 16 EB (south) Bremerton -> Tacoma

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.38	Nalley Valley Viaduct	Y	2	N	
0.47	S-N Ramp & SR16	Y	2	N	
0.62	Cedar St.	Y	3	N	
1.20	Union St.	Y	2	N	
1.67	Snake Lake Br.	Y	4	Y	patched entrance
3.12	S. 12th St.	Y	3	N	
3.53	Pt. Defiance Ferry Terminal	Y	3-4	N	
3.62	Tahlequah Ferry Terminal	Y	4	N	exit rough
5.70	6th Ave				
5.83	Pearl St.				
7.28	Tacoma Narrows	Y	2	N	
12.76	Rosedale	N	3	Y	patched on and off
15.75	N-W Ramp 302	N	2	N	
25.06	Sidney Rd.	N	3	?	broken pavement
26.70	Tremont St.	Y	3	N	

SR 16 WB (north) Tacoma -> Bremerton

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.38	Nalley Valley Viaduct	Y	2	N	
0.47	S-N Ramp & SR16	Y	3	N	
0.62	Cedar St.	Y	3	N	
1.20	Union St.	Y	3	N	
1.67	Snake Lake Br.	Y	4	Y	patched exit
3.12	S. 12th St.	Y	3	N	
3.53	Pt. Defiance Ferry Terminal	Y	3-4	N	
3.62	Tahlequah Ferry Terminal	Y	4	N	exit rough
5.70	6th Ave				
5.83	Pearl St.				
7.28	Tacoma Narrows	Y	2	N	
12.76	Rosedale	N	5	Y	patched on and off
15.75	N-W Ramp 302	N	2-3	N	patching
25.06	Sidney Rd.	N	3	Y	entrance & exit
26.70	Tremont St.	Y	2	N	
27.80	Jct. 160	?	4	Y	rough

S.R. 167 S.B. Renton-> Puyallup

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.64	B.N.R.R.	Y	2	N	
3.69	Clark Cr.				
6.40	Puyallup R.	Y	3	Y	
6.85	Milwaukee Ave	N	2	N	
7.19	Valley Ave. & U.P.R.R.	N	2	N	
7.56	West Valley Hwy.	N	2	N	
10.62	8th Ave	N	3-4	N	
11.70	3rd Ave	N	3	N	
12.26	Ellingston Rd.	N	4	N	
12.69	1st Ave N.	N	2	N	
14.27	S.R. 18	N	3-4	Y	short patches on and off
19.04	Green River	N	2	N	
19.60	S.R. 516 (Willis)	N	2	N	
19.83	Meeker St.	N	2	N	
20.20	James St.	N	2	N	
20.40	U.P.R.R.	N	2	N	
20.70	4th Ave. N	N	1	N	
20.96	B.N.R.R.	N	4	N	significant bump on exit
21.31	84th Ave.	N	2	N	

S.R. 167 N.B. Puyallup -> Renton

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.64	B.N.R.R.	Y	3	?	
3.69	Clark Cr.				overlay
6.40	Puyallup R.	Y	5	Y	large bumps from joints
6.85	Milwaukee Ave	N	3-4	N	overlay
7.19	Valley Ave. & U.P.R.R.	N	3	N	
7.56	West Valley Hwy.	N	2	N	
10.62	8th Ave	N	3	N	
11.70	3rd Ave	N	1	N	overlay
12.26	Ellingston Rd.	N	1	N	paved on passing lane only
12.69	1st Ave N.	N	2	N	paved on passing lane only
14.27	S.R. 18	N	2	Y	patch on exit, entrance o.k
19.04	Green River	N	3-4	N	
19.60	S.R. 516 (Willis)	N	2	N	
19.83	Meeker St.	N	2	N	
20.20	James St.	N	3	N	
20.40	U.P.R.R.	N	2	N	
20.70	4th Ave. N	N	2	N	
20.96	B.N.R.R.	N	2	N	
21.31	84th Ave.	N	2	N	

S.R. 82 W. B. Tri-Cities -> Yakima

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
23.89	Selah Cr.	?	2	?	overlay
25.15	E Pomona Rd.				
26.22	Roza Canal				
29.02	East Selah				
30.77	Yakima R.				
30.90	Naches R.				
30.99	Overflow Channel				
32.47	N.P.R.R. Moxee				
33.83	Beech St.				
36.29	Valley Mall				
38.48	Yakima R.				
40.34	Gangle Rd.	Y	2	N	
41.53	B.N. & U.P.R.R.	Y	2	N	
42.52	Mellis Rd.	Y	1	N	
55.63	Old S.R.12	Y	1	N	
58.50	S.R. 223	Y	3	N	
58.78	B.N.R.R.	Y	1 off 4 on	N	
61.27	Dekker Rd.	Y	2	N	
65.85	Snipes Mtn.	Y	1	N	
66.93	Midvale	Y	3	N	
67.19	U.P.R.R.	Y	1	N	
70.99	Tear-Forsell	Y	2	N	
72.60	B.N. & S.R. 12	Y	3 on 4 off	N	
74.25	Sunnyside Canal	Y	2	N	

S.R. 82 W. B. Tri-Cities -> Yakima (Continued)

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
77.03	King Tull Rd.	Y	2	N	
78.33	U.P.R.R. -Sunnyside	Y	1-2	N	
81.86	Chandler Canal	Y	3	N	
81.91	Yakima R.	Y	2-3	N	
96.02	B.N.R.R.	Y	2	N	
102.51	S.R. 182	Y	1	N	
104.51	Goose Gap	Y	2	N	
108.93	Badger Rd.	Y	2	N	

S.R. 82 E. B. Yakima ->Tri-Cities

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
23.89	Selah Cr.	?	1	?	overlay
25.15	E Pomona Rd.	?	3		
26.22	Roza Canal	?	2	?	
29.02	East Selah	?	3	?	
30.77	Yakima R.		1		
30.99	Naches R.		2		
30.99	Overflow Channel		3		
32.47	N.P.R.R. Moxee		3		
33.83	Beech St.		1		
36.29	Valley Mall		1		
38.48	Yakima R.		2		
40.34	Gangle Rd.	Y	1	N	
41.53	B.N. & U.P.R.R.	Y	3	N	
42.52	Mellis Rd.	Y	2-3	N	
55.63	Old S.R.12	Y	3	N	
58.50	S.R. 223	Y	1	N	
58.78	B.N.R.R.	Y	1 -2	N	
61.27	Dekker Rd.	Y	2	N	
65.85	Snipes Mtn.	Y	2	N	
66.93	Midvale	Y	1	N	
67.19	U.P.R.R.	Y	2	N	
70.99	Tear-Forsell	Y	1	N	
72.60	B.N. & S.R. 12	Y	1	N	
74.25	Sunnyside Canal	Y	2	N	

S.R. 82 E. B. Yakima ->Tri-Cities (Continued)

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
77.03	King Tull Rd.	Y	3 off 4 on	N	
78.33	U.P.R.R. -Sunnyside	Y	2 off 3 on	N	
81.86	Chandler Canal	Y	3	N	
81.91	Yakima R.	Y	2-3	N	
96.02	B.N.R.R.	Y	3	N	
102.51	S.R. 182	Y	2-3	N	
104.51	Goose Gap	Y	2	N	
108.93	Badger Rd.	Y	3	N	

S.R. 5 S.B. Vancouver -> Kelso

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.34	Mill Plain Blvd.	Y	2	N	
2.35	McLoughlin	Y	1	N	
4.38	78th St.	?	1	?	overlay
5.40	99th St.	?	1	?	overlay
6.32	Salmon Cr.	?	1	?	overlay
7.48	S.R. 205	?	1-2	?	overlay
9.51	S.R. 502	Y	2	N	
18.2	E Fk. Lewis R.	Y	1	N	
19.83	Lewis R.	Y	3 off 4 on	Y	patched exit
21.08	S.R. 503	Y	2	N	
22.72	Log Dump Rd.	Y	3 on 4 off	Y	both ends patched
26.01	B.N.R.R.	Y	2	?	overlay
27.70	Todd Rd.	Y	3	Y	patched entrance travel lane patched on exit
29.85	Elm St.	Y	3	N	
31.82	Kalama Rd.	Y	2 on 3 0ff	Y	entranced patched
35.81	Owl Cr.	Y	4-5	Y	entrance & exit patched
38.48	Coweman 135	Y	4-5	?	
39.26	Coweman R. Br.	Y	3-4	Y	patched exit
39.29	Coweman 185	Y	2	N	
39.82	Allen St.	Y	4-5	Y	patched entrance
40.72	S.R. 431	Y	3	?	

S.R. 5 N.B. Kelso -> Vancouver

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
0.34	Mill Plain Blvd.	Y	2	N	
2.35	McLoughlin	Y	4	?	
4.38	78th St.	?	2	?	overlay
5.40	99th St.	?	2	?	overlay
6.32	Salmon Cr.	?	3	?	overlay
7.48	S.R. 205	?	2	?	overlay
9.51	S.R. 502	?	2	?	
18.2	E Fk. Lewis R.	Y	3	N	
19.83	Lewis R.	Y	4	N	
21.08	S.R. 503	Y	3	?	
22.72	Log Dump Rd.	Y	1	N	both ends patched
26.01	B.N.R.R.	Y	2	?	overlay
27.70	Todd Rd.	Y	4	Y	patched on exit
29.85	Elm St.	Y	4	?	
31.82	Kalama Rd.	Y	4	Y	patched exit & entrance
35.81	Owl Cr.	Y	5	Y	entrance & exit patched
38.48	Coweman 135	Y	3	N	
39.26	Coweman R. Br.	Y	3	Y	patched exit
39.29	Coweman 185	Y	3	N	
39.82	Allen St.	Y	5	Y	patched entrance
40.72	S.R. 431	Y	3	Y	

S.R. 90 E.B. Grant/Adams Co. line -> Ritzvlle

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
194.82	Moody Rd.	?	2	?	overlay
196.61	Farrier Coulee	?	2	?	overlay
196.91	Deal Rd.	?	2	?	overlay
199.91	Batum Rd.	?	1	?	overlay
202.14	Damon Rd.	?	3	?	
206.18	S.R. 21	?	3 off 4 on	?	
210.03	Wahl Rd.	?	3	Y	patched/boken pavement
215.24	Paha Rd.	?	4	Y	patched or broken
221.95	S.R. 261	?	4-5	?	overlay

S.R. 90 W.B. Ritzvlle -> Grant/Adams Co. line

M.P.	Bridge	Approach Slab	Bump	Patches	Comments
194.82	Moody Rd.	?	1	?	overlay
196.61	Farrier Coulee	?	2	?	overlay
196.91	Deal Rd.	?	2	?	overlay
199.91	Batum Rd.	?	2	?	overlay
202.14	Damon Rd.	Y	4	?	bump on exit
206.18	S.R. 21	?	2	N	
210.03	Wahl Rd.	?	3	Y	patched/boken pavement
215.24	Paha Rd.	?	3	?	patched or broken?
221.95	S.R. 261	?	2 on 3 off	?	overlay