AN ABSTRACT OF THE THESIS OF

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Abstract Approved: _____
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The undermining of bridge foundations can lead to either costly repairs or a bridge collapse. These foundations must be designed to counter the effects of scour. Current practice does not allow for accurate estimates of scour in erodible rock. Scour in rock can be related to geotechnical and hydraulic properties. A field study of eleven bridge sites provided samples of the bedrock where the abrasive resistance of the rock was determined and hydraulic properties of the channel were calculated. Laboratory abrasion resistance values from a modified slake durability test and hydraulic variables such as stream power were compared to recent and past stream channel cross-sections. A preliminary model has been proposed wherein the degradation of the stream channel is related to the abrasive resistance of the bedrock and the area under the daily stream power. This method provides an estimate of the degradation of the stream bed due to abrasion by bedload and flood events, not necessarily local or contraction scour.

SCOURABILITY OF WEAK ROCK IN THE OREGON COAST RANGE

by

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SCOURABILITY OF WEAK ROCK IN THE OREGON COAST RANGE

1. INTRODUCTION

1.1 STATEMENT OF THE PROBLEM

In a study of 823 bridge failures in New York between 1950 and 1991, hydraulics accounted for 494, or 60% of the failures (Shirhole and Holt, 1991). These include failures due to the impact of scour, debris and ice flows on footings and abutments. Scour can be defined as "the result of the erosive action of flowing water, excavating and carrying away loose material from the bed and banks of streams" (Richardson, et. al., 1993). On April 5, 1987, two spans of the I-90 crossing over Schoharie Creek near Amsterdam, New York collapsed and fell about 24 m (80 ft) into the flooding stream. The cause of the collapse was an undermining of a pier due to local scour in cohesionless material. Ten people were killed as a result of the scour induced collapse. A short time after the first collapse, another pier was undermined and collapsed, bringing down the third span. After this event the Federal Highway Administration (FHWA) expressed a heightened interest in the research and improvement of the state-of-practice of bridge design and scour, and implemented a comprehensive bridge inspection program for existing bridges to determine their safety (Landers and Mueller, 1995). The ensuing research efforts led to the development of design methods for scour in cohesionless materials.

At sites underlain by weak rock, foundation design has traditionally relied on deep foundations in order to obtain secure bearing beneath the potential zone of scour. Current FHWA guidelines for scour require assessments and/or monitoring programs. Prioritizing the potentially hazardous bridge sites could be made, provided a method existed for evaluating the scour potential in weak rock foundations. Current methods are highly overconservative when applied for rock formations such as shales and weak sandstones that erode due to water and bedload abrasion.

The Oregon Department of Transportation (ODOT) accounts for 2,640 bridges in Oregon, of which, about 65% are over water. Figure 1.1 summarizes a survey of all the state bridges over water in Oregon. Of these bridges, 44% are pile supported, 16% are spread footings on non-erodible material, and 40% are spread footings on erodible material (Bryson, 1998).

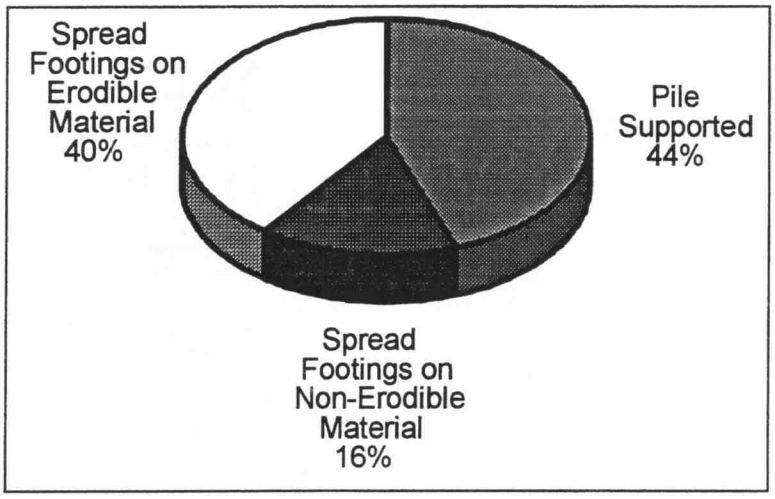

Figure 1.1: Summary of Oregon Department of Transportation Bridge Foundations of Bridges over Water

From Figure 1.1 it is evident that a large percentage of bridges have spread footings on erodible material. Spread footings on erodible material could fail before the design life of the structure due to problems such as undermining of foundations and subsequent collapse. This type of failure is not immediate, which makes the problem of scour time dependent. Scour in non-cohesive material is relatively immediate in comparison to scour in an erodible rock formation such as shale. Under constant flow conditions, maximum scour depth in sands and other non-cohesive materials takes hours, while rocks will reach the same scour depth in years or centuries depending on hardness (Richardson, et. al., 1993). Therefore, the problem is not whether the rock will scour, it is how much time is required under particular flow conditions to achieve the critical scour depth.

With regard to bridge foundations on potentially scourable rock, two items need to be addressed. First, to establish guidelines for evaluating the scour susceptibility of weak, jointed rock masses, and estimate the rate of erosion in these types of materials. Second, to establish improved guidelines for evaluating the safety of existing bridges founded on various types of rock. This study focused on evaluating the scour susceptibility and estimating the erosion rate. From this information, the guidelines are also improved for evaluating the safety of existing bridges.

1.1.1 Rock Scour

A relevant example of scour in rock is the flow of water through unlined spillways located in bedrock. In locating a dam site on rock, the foundation must be investigated for geologic properties such as jointing, bedding, cleavage, and size and direction of cracks. The rock quality of a dam site is very important as it will be subjected to large hydraulic forces at the spillway which could result in dam foundation instability.

A conceptual model based on observations of spillways and available literature on river bed scouring suggests that scour in rock involves three phases. The first phase involves removal of rock fragments due to pressure from turbulent flow. During this phase the jointing and discontinuities in the rock mass are the prevailing geologic characteristics that contribute to scour. The uniaxial strength properties of the rock are insignificant. The pressure gradient created by turbulent flow must overcome the fragment's weight and the cohesive resistance in order to pull it out of the surrounding deposit. Assuming constant flow out of the spillway during a single event, the increase in scour depth causes a larger flow area. Therefore the flow velocity and consequentially, the bedflow energy decreases leading to phase two (Akhmedov, 1988). In this phase, flow energy is still enough to remove fragments through vibration induced by pressure fluctuations, but now abrasive forces are evident, reducing the size of the fragment to a point where it is dislodged and removed. The third phase is where flow energy does not remove the rock fragments and scour is due purely to intensive abrasion from non-cohesive particles (Akhmedov, 1988).

Relating a general flow condition to those typically observed in the Oregon Coast Range streams investigated in this research, most streams would fall into "phase three" streams, where bedrock scour is due predominately to abrasion. However, on the higher gradient streams (slope > 0.6 %) located in the Coast Range and the Cascades that have bedrock with a low Rock Quality Designation (RQD), there is a transition from a "phase three" stream to a "phase two" stream during high power flood events. This means that scour is now caused by both abrasion and pressure induced removal of rock fragments.

A method of predicting the potential for scour based on geologic properties of the rock and hydraulic parameters has been proposed (Annandale, 1995). Annandale's model is based on observations of clear water scour of emergency spillways. The conceptual model is similar to phase one scour as described by Akhmedov in that the rock is scoured due to jacking, dislodgment, and displacement. However, Annandale does not address abrasion. The Erodibility Index (K_h), first introduced by Kirsten in 1988, represents the rock's ability to resist erosion based on rock fragment removal. This involves geologic parameters such as RQD, joint spacing and roughness, and unconfined compression. In Figure 1.2 the maximum stream power was calculated for the emergency spillways and related to K_h (Annandale and Kirsten, 1994). The method defines a threshold stream power required to induce scour in non-cohesive materials and rock based on the Erodibility Index.

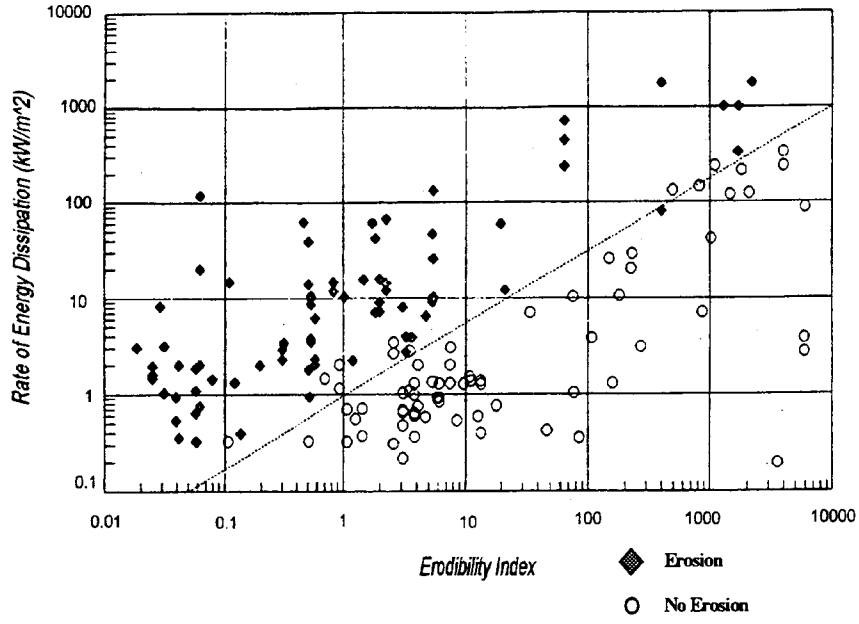

Figure 1.2: Erodibility of Rock and Complex Earth Materials (Annandale, 1995)

The Erodibility Threshold in Figure 1.2 represents the stream power required to remove rock fragments based on their Erodibility Index. The Erodibility Index will not change for a particular rock, so once it has been calculated the only factor controlling scour is the stream power. As the stream power increases, the Erodibility Threshold is approached and eventually surpassed, at which point the rock will scour due to jacking and dislodgment. This is useful in estimating the power required to remove fragments, however the rate of scour is not addressed.

The Colorado Department of Transportation used the Erodibility Index to develop a method of predicting scour depths in layered soil and rock profiles. The depth of the scour (independent of time) is estimated by comparing the stream power and Erodibility Index to the Erodibility Threshold. Erosion of each layer will occur sequentially as long as the stream power exceeds the Erodibility Threshold of the exposed material. The depth of scour is a function of the depth of erodible geologic bedding. Scour will occur until a more resistant layer is met, with no rate of scour provided. If the layer is deep then

erosion will continue to a depth where the resulting stream power is less than the threshold stream power due to changes in channel morphology (Smith, 1994).

The streams in this study were consistently below the threshold power represented in Figure 1.2, yet each site showed signs of bedrock erosion. This demonstrates the need for enhancing the scour model developed by Annandale and CDOT. When referring to Akhmedov, when the stream power is too low for dislodgment, then erosion must be related to abrasion forces. While the CDOT method provides useful information on properties of the rock and an estimate for scour depth, it is desirable to develop a method for predicting the rate of scour in rock at streams such as those in the Oregon Coast Range. This would provide bridge engineers with a screening tool for prioritizing scour mitigation measures at potentially hazardous stream crossings.

1.1.2 Current Design Procedures

Scour can be defined as "the result of the erosive action of flowing water, excavating and carrying away loose material from the bed and banks of streams" (Richardson, et. al., 1993). The current design procedure in predicting scour in soil bed materials is presented in Hydraulic Engineering Circular No. 18 (HEC 18) (Richardson, et.al., 1993). Procedures for predicting scour in cohesionless materials are outlined with several equations based on laboratory flume studies. These methods provide estimates for local scour, contraction scour and degradation/aggradation in cohesionless materials using grain size, sediment transport and hydraulic properties such as flow and stream velocity. In this investigation, traditional methods of evaluating scour in non-cohesive soils were not addressed. Instead, fundamental research on the rate at which sedimentary rocks in the Oregon Coast Range erode was performed. The research efforts focused on the average rate of scour across a natural stream channel due to the erosive forces of abrasion from bedload, or degradation of the bedrock. Therefore, local and contraction scour were not investigated. Although local scour, which involves the amplification of stream power due to pier geometry, and contraction scour, which involves stream power changes due to the geometry of the channel were not investigated, the average stream power can be

modified with factors based on pier or channel geometry. The CDOT method outlines some of these factors (Smith, 1994).

In order to better understand scour in rock, a thorough literature review was performed. As of December 1997, a consensus on the scour resistance of various rock types had not been established and design methods have yet to be proposed. HEC 18 addresses the issue of scour in highly resistant rock and spread footings on erodible rock. Spread footings on highly resistant rock need only be laterally restrained with dowels embedded into the rock, while erodible rock involved evaluations by engineering geologists, supplemented with analysis of intact rock cores (Richardson, et. al., 1993). In a memorandum issued by the Bridge Division of the FHWA (Gordon, 1991), the scourability and rock quality should be assessed using the following geotechnical parameters: (1) Rock Quality Designation (RQD) (Deere, 1963), (2) Unconfined Compressive Strength of the material (q_u), (3) Slake Durability Index, (4) Sulfate Soundness and (5) the LA Abrasion test (Gordon, 1991).

Evidence of erosion and partially undermined footings existed in a study that included a bridge founded on the shale bedrock of the Canadaway Group in New York. The five tests recommended by Gordon were performed on the bedrock in overall agreement with the memorandum and some minor changes were recommended for unprotected footings on shale (Avery and Hixon, 1993). The geotechnical parameters given in the memorandum and recommended changes from Avery and Hixon are summarized in Table 1.1. Even with these changes, there is no existing method to predict the depth to which scour will occur and the time required for this streambed degradation. These values represent an evaluation of the bedrock quality, which is important in locating and inspecting bridges and determining the susceptibility of scour.

Table 1.1: Summary of Existing Geotechnical Parameters for Evaluating Scour Potential

Test	ASTM (1996)	FHWA Memorandum (1991)	Avery and Hixon Modifications (1993)
RQD (Deere, 1963)	--	> 50%	> 40%
Unconfined Compression (q_u)	D2938	> 1724 kPa (250 psi)	> 1724 kPa (250 psi)
Slake Durability Index	D4644	> 90	>92
Sulfate Soundness (Sodium) (Magnesium)	C88	> 12 > 18	> 12 > 18
LA Abrasion (Loss %)	C131	< 40	< 40

1.2 SCOPE OF THE PROJECT

In a joint effort between the Oregon Department of Transportation and Oregon State University, a method to estimate scour in different types of rock was investigated. Eleven sites in the Oregon Coast Range were selected based on evidence of erosion, historical stream information and gauging, site geology, and bedrock exposure across the channel. Rock consisted of sedimentary lithology with hardness varying from very soft siltstones to hard tuff. The resulting model relates abrasion resistance of the rock with stream power.

1.3 THE PREMISE BEHIND THE RESEARCH

In developing the research for this problem, geotechnical factors needed to be combined with hydraulic factors. The rock properties are as important to the problem of bedrock scour as the hydraulic variables. Since this research is focusing on the problem of abrasion, some strength properties are required. Abrasion is the wearing away of the bedrock (and bedload) caused by friction of the moving bedload against the bedrock. Therefore, the geotechnical parameters must involve abrasive resistance. Axial strength properties such as the Unconfined Compression test are important to footing design, however, they have been found to be poorly related to the problem of abrasion. Different Geotechnical tests and their relevance are described in Section 3.2.

In addition to the geotechnical parameters, the hydraulic factors must also be assessed. The hydraulic factors govern bedload movement and provide pressure fluctuations that wear down the bedrock. Factors such as velocity and stream power are necessary for sediment transport, and the sediment transport in the form of bedload is important to scour due to abrasion. Hydraulic variables are discussed in further detail in Chapter 4.

By combining geotechnical factors and hydraulic factors, a relationship between abrasion and stream power over time can be established. From daily flow records, daily stream power can be calculated and plotted in a form similar to a flow hydrograph. If it is assumed that the rate of rock scour is related to the stream power and the area under the stream power curve is calculated over time, then the average erosion over a period of time, or the erosion rate can be calculated. Chapter 5 describes the results of combining the geotechnical and the hydraulic data.

1.4 INTEGRATED STREAM POWER

The variables given by the Army Corps of Engineers HEC-RAS version 2.0 River Analysis System included shear, velocity, and stream power. Annandale has used the rate of energy dissipation per unit width of flow to describe the erosive power of water during turbulent flow for emergency spillways and weirs (Annandale, 1995). He presented the stream power as follows:

$$P = \gamma q(\Delta E) \tag{1-1}$$

Where; γ is the unit weight of the fluid, q is the unit discharge, and ΔE is the energy loss.

Costa and O'Conner (1995), studied geomorphically effective floods, or flood events that alter the channel and overbank areas. Using investigations from floods in the Northwest, they considered the importance of flood-flow duration with the geomorphic effectiveness. While Annandale looked at the maximum flow, and therefore the maximum power as the preferred variable due to turbulence, Costa and O'Conner looked at the role of duration of the flood as the critical factor. This means it is important to know both the size of the flood and the duration of the flood (Costa and O'Conner, 1995). This idea

relates well to a daily power, which tracks the time and size of any flood over many years. Figure 1.3 shows a conceptual model for explaining how the flood duration and power affect the destructive ability. Flood "A" is a long-term, low power flood that would cause insignificant scour. Similarly, flood "C" is a short-duration, high power flood with small destructive ability. However, Flood "B" shows a high-intensity, long duration flood that is highly destructive. From this, an average energy per unit area expended during a flood (Ω) is represented as:

$$\Omega = \int \gamma QS/w \, dt \tag{1-2}$$

Where; Q is the discharge, S is the energy gradient, *w* is the water surface width, and *t* is the time.

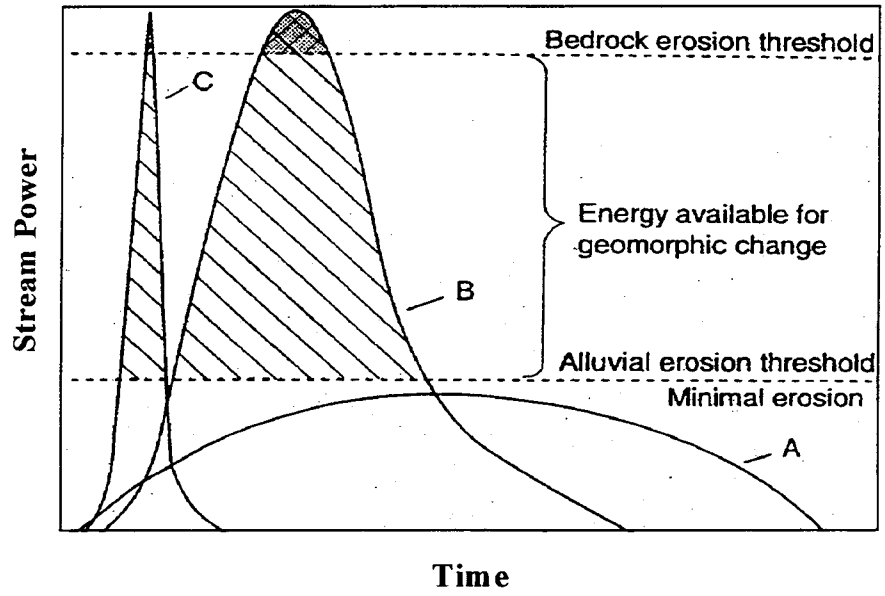

Figure 1.3: Conceptual Stream Power Graphs for Different Floods (Costa and O'Conner, 1995)

This concept was expanded over the duration of the study (i.e. from the date of the first cross-section to the date of the second cross-section) and the area under the daily stream power plot above zero stream power was calculated. This is termed *integrated stream power* and denoted as Ω. The daily power value came from the relationship of channel stream power with flow which is described in Section 4.2. The resulting units are

expressed in power per area, or stream power per unit area. Costa and O'Conner also stated that time integrated flood power when compared with some quantitative measure of resistance would be useful in determining geomorphic effectiveness (Costa and O'Conner, 1995).

2. STREAM STUDY SITES

2.1 SITE INVESTIGATION

Approximately 40 to 50 candidate sites were selected from the files of ODOT and the Siuslaw National Forest. Each site was visited and photographed. From these observations, candidate sites were narrowed down on the basis of geology, availability of historical cross-sections and stream data, visible bedrock, and accessibility to the site. Ten sites at which requisite data was available were selected as appropriate for the study. The field investigation at each site consisted of drilling for rock cores and surveying the channel. The rock cores were taken to the geotechnical lab at Oregon State University (OSU) for a variety of strength and durability tests. The channel cross-sections taken during the survey were compared to historic cross-sections and used to create computer models for calculating the hydraulic variables. Information on the tests performed on the rock cores and the hydraulic study will be discussed in more detail in Chapters 3 and 4. Additional sites were drilled with a hand coring drill. All of the geotechnical and hydraulic information data obtained are detailed in Appendix A.

2.2 SITE SELECTION

The sites for this study were all located in or near the Coast Range of Western Oregon. The purpose of this was due to the geology of this region. Weak sandstones and siltstones are the predominate rock type with more volcanic rocks towards the Cascades (i.e. Basalt and Andesite). Sites in the Cascades were not used because this type of rock is very hard and abrasion resistant. The varying hardness of the sandstones and siltstones provided a range of erosion and geologic properties useful to the study. Combined with some high gradient streams, which are capable of producing high stream powers, many bridges in the Coast Range are susceptible to scour. Some sites were selected over short periods of time but bracketed two major flood events in Oregon. The Nestucca Site is from 1995 to 1997

and the Rosenbalm site is from 1990 to 1996. This is important because it is a clue to the destructiveness of a single flood event. Figure 2.1 displays the location of all the sites in the study.

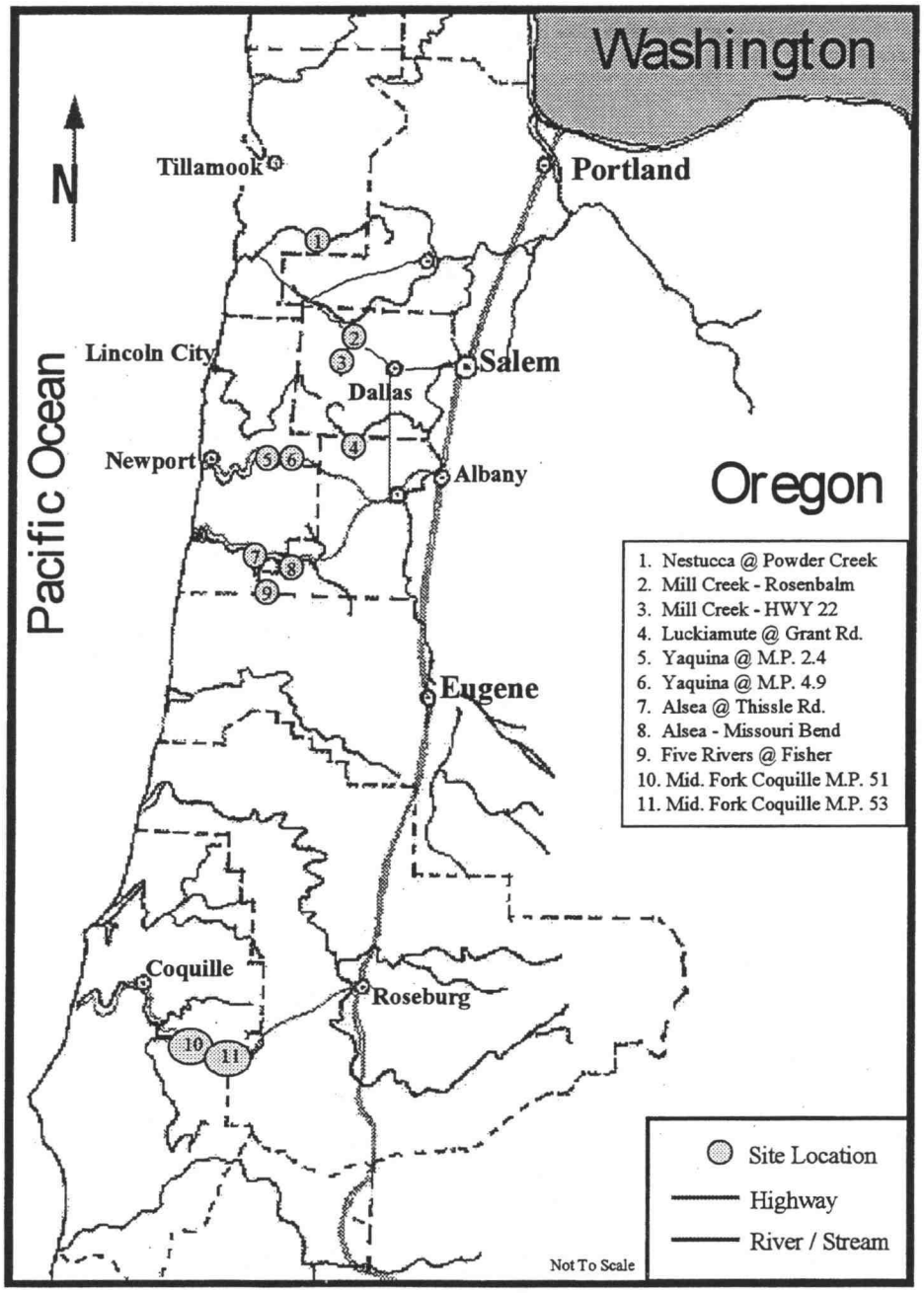

Figure 2.1: Site Locations

2.2.1 Mill Creek at Rosenbalm

This site is located in Polk County approximately 20 miles west of Salem on Highway 22. The footings on the west side of the bridge over Mill Creek were exposed with obvious signs of bedrock erosion.

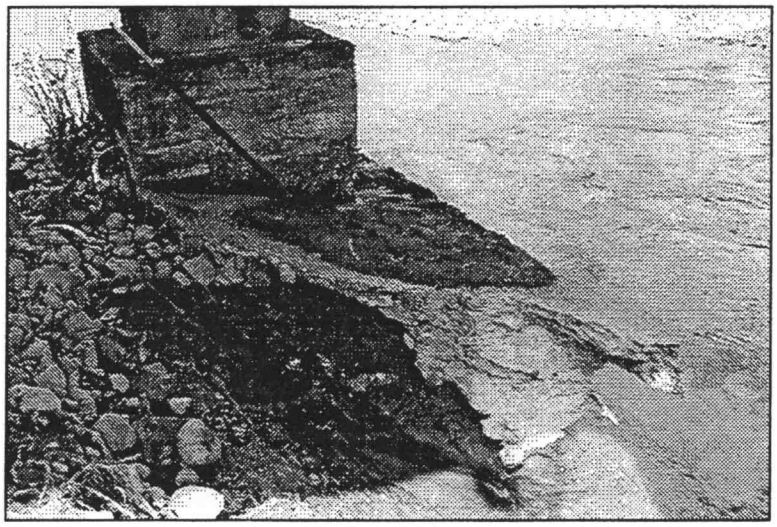

Figure 2.2: Evidence of Bedrock Erosion at Mill Creek - Rosenbalm

The bridge at this site was replaced during the summer of 1997 by a single span structure. The bedrock is from the Eocene age and is a dark-gray, faintly bedded shale and siltstone from the northern Coast Range Yamhill Formation (Peck, 1961). At the time of investigation, this site had some bedload, but is mostly exposed bedrock. The cross-sections were performed by soundings and compared with bridge inspection soundings from 1990.

Daily water flow information came from USGS Stream Gauge 14193300 (Mill Creek near Willamina, OR). Recordings were only obtained until 1973, therefore daily flow values were synthesized from a correlation with flows measured on the South Yamhill River. The flows were reduced to account for the different drainage areas by a

method outlined in the USGS report "Magnitude and Frequency of Floods in Western Oregon" (Harris et.al., 1979). This site was selected because of the obvious erosion and weak rock. A bridge crossing Gooseneck Creek (a tributary of Mill Creek) also has visible erosion, however due to the unusual flow conditions and repairs made to the bridge, this additional site was not selected.

2.2.2 Mill Creek at Highway 22

This site is located approximately three miles upstream of Rosenbalm and is a triple span bridge on Highway 22 crossing Mill Creek in Polk County. The bedrock is the same siltstone and the flow is about one-third that of Rosenbalm. This site has two footings on the edges of the stream with some visible undermining of the footing, but not as dramatic as Rosenbalm.

Figure 2.3: Evidence of Erosion under a Bridge Footing at Mill Creek – HWY 22.

The survey consisted of soundings and compared to cross-sections from plans created in 1983. The elevation used for the survey came from the plans. The hydraulic information is from the same stream gauge as Rosenbalm and adjusted to account for tributaries and the different drainage area. This site does have an armor layer on the downstream side of the bridge, however the upstream side does have exposed bedrock with some bedload.

2.2.3 Yaquina River at Mile Post 2.4

This bridge spanning the Yaquina River is located on the Eddyville/Blodgett Highway 2.4 miles north of Eddyville, and about 25 miles west of Corvallis. This bridge was affected by the February flood of 1996 as lateral migration of the stream during a high water event washed away the rock and fill material exposing the piles used in the foundation. The bridge is a two span structure with one footing placed in the middle of the stream. There is visible bedload in the stream, and a small bar is being formed downstream of the pier in the water.

The geology of the area is of the Tyee and Burpee formations consisting of Feldspathic and Micaceous massive-bedded sandstone and subordinate siltstone from the Middle-Eocene period. The bedding of these formations consists of coarse graded sandstone at the bottom to fine sandstone and siltstone at the top (Peck, 1961). This site consists of siltstone with weak, parallel planes. The geometry and engineering data was obtained from ODOT and the cross sections are from 1976 plans for the bridge. The elevation used in surveying the bridge is from the plans, and the stream channel was surveyed using soundings. Daily stream values come from the USGS, stream gauge number 14306030 (Yaquina River near Chitwood Oregon).

2.2.4 Yaquina River at M.P. 4.9

About 2.5 miles upstream of Yaquina at M.P. 2.4, a triple span bridge crosses the Yaquina River. The flow is about a half of the value for the other Yaquina site and the

slope is less. The geology consists of the same formations, however the material at this particular site is fine grained sandstone with layers of weak siltstone. The bedload is fine grained silt grading into coarser material. The soundings during low flow conditions encountered this material in the channel. This had to be accounted for when compared to the ODOT cross sections from 1976. This site had a bench mark on one of the bents.

2.2.5 Alsea River at Thissel Road

This site is located about 30 miles west of Corvallis along Highway 34. The three span bridge belongs to Siuslaw National Forest along Thissel road and crosses the Alsea River. The site has exposed bedrock across the channel with rock outside the channel. The bridge has two spread footings into the rock on the edges of the stream. During high water, the water flows around the piers creating some local effects.

The geology is Tyee and Burpee Sandstone similar to the Yaquina sites. However, instead of fine sandstone, the material is coarser, with some jointing. High recovery and Rock Quality Designation (RQD) values were obtained during the geologic exploration. This site is particularly good because of the exposed bedrock and the stream gauge (USGS gauge 14306500, Alsea River near Tidewater) is nearby. Since the gauge is so close and currently in operation, no adjustments were made to the flow, and no correlations were made to synthesize flow. The survey consisted of soundings and compared to soundings performed in 1989 for a bridge inspection performed by Siuslaw National Forest.

2.2.6 Alsea River at Missouri Bend

About 10 miles upstream from the Alsea at Thissel site, there is a three span bridge located off of Highway 34 on Benner Creek Road. This bridge is also from Siuslaw National Forest and is founded on the same Tyee sandstone formation as the Thissel site. The difference is a longer period of record, and a shallower slope, creating more bedload

due to lower velocity. The footings are located up the bank from the water and will encounter water only in unusually high flood events. The survey consisted of soundings and compared to soundings performed in 1978 for a bridge inspection performed by Siuslaw National Forest.

2.2.7 Five Rivers at Fisher

This site is a single span bridge across Five Rivers located next to a covered bridge in Fisher, Oregon about 6 miles south of the Alsea at Thissel site. The geology is the Tyee sandstone, but a little finer and harder than the Alsea sites. Recovery was near 100 percent and RQD values were between 75 to 100 percent. The water information comes from USGS gauge number 14306400 (Five Rivers near Fisher, Oregon) up until September 1990, when the gauge was closed. Since Five Rivers is a tributary of the Alsea, the Alsea near Tidewater gauge was correlated with the Fisher gauge and synthetic data was produced until October 1996. The survey consisted of soundings and compared to ODOT plans from 1973. This site has the longest record of all the sites.

2.2.8 Middle Fork Coquille River at Mile Post 51 (M.F.C. 51)

This site is located about 30 miles southwest of Roseburg, Oregon on Highway 42 at a single span bridge crossing the Middle Fork Coquille. The geology is marine sedimentary rocks consisting of thin-bedded, alternating dark-gray mudstone and sandstone with massive micaceous and tuffaceous sandstone beds from the lower Eocene (Peck, 1961). This site has coarser sandstone that erodes easier than the finer sandstones.

The bridge is located near the beginning of the river, with the USGS stream gauge 14326500 (Middle Fork Coquille River near Myrtle Point, Oregon) located about 10 miles away. This gauge was only active until 1946, so the entire record is based on synthetic data from averaging flows in or near the basin (South Umpqua and Rogue Rivers) and correlating the average with the Middle Fork Coquille record. The flow was adjusted to

account for the smaller drainage area using the ratio of drainage areas as outlined in the USGS report. The survey is from surveying and subtracting the soundings and comparing with elevations from 1981 plans.

2.2.9 Middle Fork Coquille River at Mile Post 53 (M.F.C. 53)

This site is located about 2 miles upstream from the M.F.C. 51 site. The geology consists of harder fine grained sandstone with some darker mudstone. The site is similar to M.F.C. 51 except the flow is lower. The flow is calculated the same way and then adjusted to account for different drainage areas. The bedload consists of approximately one inch minus material on both sites with exposed bedrock visible in parts of the stream. There is no visible bedrock scour under the footings at this site, but over the period of study some has occurred. This site is influenced by contraction effects during high flows. However to limit these effects, the cross-sections were taken upstream of the bridge.

2.2.10 Nestucca River at Powder Creek

This site is located about 10 miles east of Beaver, Oregon, on Powder Creek Road, at a single span bridge crossing the Nestucca River. This same site was used by Oregon State University for a research project to estimate flows for the north coast range. The research project included a survey of the channel and the installation of a staff gauge. The soundings were taken in December of 1995 on the downstream side to allow for visibility of the weight being pushed downstream. This allowed for a more accurate adjustment of the depth due to the string not being vertical. In June of 1997, the stream was resurveyed using a fiberglass survey rod, physically measuring depth from the water surface to the bedrock and comparing the height of water with the staff gauge. During these two years this site was exposed to two 100-year flood events. The difference in elevations could also have been influenced by some contraction effects. The geology of this site consists of dark gray tuffaceous shale, siltstone, and thin-bedded sandstone from

the Nestucca Formation of the Upper Eocene (Peck, 1961). This site was drilled by hand, with RQD and recovery values coming from a bridge further downstream founded in the same material. The bedrock is visible across the entire channel with little visible bedload during low-flow conditions. Water information came from correlating the average flow of the Wilson, Alsea, and Siletz Rivers with gauge data from USGS gauge 14303600 (Nestucca River near Beaver, Oregon). The flow was adjusted to account for different drainage areas.

2.2.11 Luckiamute River at Grant Road

This site is located on Grant Road off the Kings Valley Highway, about 20 miles northwest of Corvallis, Oregon at a single span bridge crossing the Luckiamute River. The bedrock is visible in the channel, with small amounts of bedload visible during low flow. The geology consists of Tyee sandstone similar to that in the Alsea River sites. The soundings were compared to ODOT plans from 1984. Flow information came from the USGS gauge 14190000 (Luckiamute River near Pedee, Oregon) up until 1970, the rest of the flow information is synthesized from the Luckiamute at Suver and the South Yamhill River. This site was drilled by hand, so no drilling information is available.

2.2.12 Other sites

Other sites were investigated but not used for several reasons. Some sites were eliminated because stream information was either not available or not able to be effectively synthesized. These sites include bridges over Euchre Creek, Deep Creek, and Slick Rock Creek. Difficulties in evaluating past cross-sections did not allow for a third bridge over Mill Creek. A bridge over the North Yamhill River was not used because bedrock was exposed in only half of the channel, while the other half was silty sand. This created difficulties in stream modeling. Even though data points could not be obtained for these sites in the final model, the rock could still be evaluated for the scour potential.

2.3 HISTORICAL CROSS SECTIONS

Historical cross-sections were obtained from ODOT and Siuslaw National Forest. The sections were established between 1940 and 1995 with a majority of the site surveys performed in the 1980's. The recent cross-sections were performed by the authors using soundings on 10 sites and survey rod readings on the other. ODOT and Siuslaw provided data from earlier cross-sections. When stream velocity is low, there are only minor differences between rod readings and soundings. However, higher velocity streams can make the soundings appear deeper than they really are due to the water pushing the weight from the vertical position and corrections need to be made for this. When compared to the recent sections, serious differences in channel shape were not noticed, however localized differences were observed. These changes could have been caused by several of the following factors: First, bedload could have altered readings taken on bedrock. For example, if initial readings were taken when there was significant bedload, then later readings were taken when bedload thickness over the bedrock was small, the difference in elevation would not be entirely due to bedrock erosion. Another difference in readings could be caused by switching between soundings and rod readings. As described earlier, soundings can produce an errant reading if the water is moving fast enough to move the tape (if no correction is made). Furthermore, if the reading is not in the same location, there might be a different reading which could lead to errant results. There is no way of knowing precisely where the measurements were taken in 1970 or 1980. Therefore, the only way to counter this is to assume that the bedload has not changed significantly over the duration of study and that the readings were in the same location as previously recorded. This assumption allows the observer to assume that any drop in elevation is now a result of erosion. In selecting sites, the authors looked for visible bedrock in the channel, without any armor layering.

Average Erosion is the average difference of individual points, of the two cross-sections over the width of the year-round saturated channel. This definition allows for observation of only the saturated bedrock with no wetting and drying effects as described

in Section 3.3.1. This definition does not account for lateral stream migration that may have occurred over the study period.

3. GEOTECHNICAL STUDY

3.1 OBTAINING SAMPLES

Samples for the laboratory study were obtained either by triple barrel coring or a hand coring drill. Due to restricted access, drilling occurred on both sides of the stream, away from the individual bridges with a truck mounted drilling machine. Samples ten feet into the bedrock were collected and wrapped in cellophane to protect them from drying. The depth of coring is to collect saturated rock samples that are below the weathered zone. Geologic and geotechnical aspects of the rock were recorded, such as percent recovery, joint locations and angles, and RQD. The other method for obtaining samples was a motorized hand coring drill provided by the Bureau of Mines. This allowed for the collection of samples directly out of the stream bed at a considerably lower cost, producing a sample appropriate for the laboratory abrasion test. By sampling directly from the bedrock in the stream, weathering profiles from exposure to air are avoided. This hand coring method does have drawbacks. The hole can only be drilled approximately one foot deep. Any deeper and the rock samples become difficult to retrieve. In addition RQD or percent recovery can not be determined, and the sample is not the specified size or shape for some of the other lab tests (e.g. unconfined compression).

3.2 LABORATORY TESTS

A variety of geotechnical lab tests were performed on the samples in order to evaluate the strength and abrasion resistance of the rock. They include: LA Abrasion (ASTM C131), Unconfined Compression (ASTM D2938), Density (ASTM D2937), and Slake Durability (ASTM D4644). A modification to the Slake Durability test (ASTM D4644) was developed and is explained in section 3.2.3.

3.2.1 Unconfined Compression

The unconfined compression strength (q_u) is a useful parameter for describing the strength and cementation of intact rock specimens. The test consists of uniaxially loading an intact sample from a rock core to failure and representing the strength of the rock as an ultimate compressive stress. The test, however, does not account for jointing or fracturing. The unconfined compression strength describes the strength of the rock and it is a very useful parameter for establishing design loads for foundations, however, it was not found to correlate well with the abrasive resistance of the rock. Results are listed in Table 5.1. As shown in Figure 3.1, the Unconfined compression test shows no significant trend with Average Erosion.

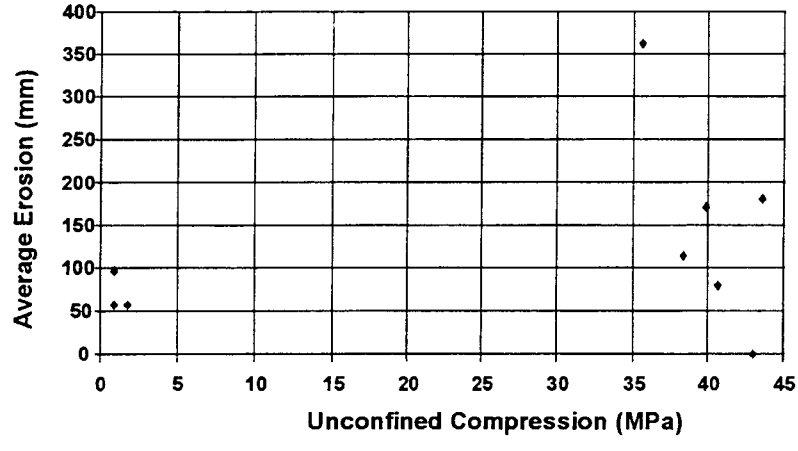

Figure 3.1: Average Erosion as related to Unconfined Compression

3.2.2 Continuous Slake Durability

A test for evaluating the wetting and drying effects on the slaking effects of clay bearing rock and siltstones has been devised by Franklin and Chandra (1972). They developed a standardized test which consists of placing 500g ± 50g of oven dry material

(10 pieces about 50g each) into a standard mesh cage (Figure 3.2) with water just below the axis of the rotating cage.

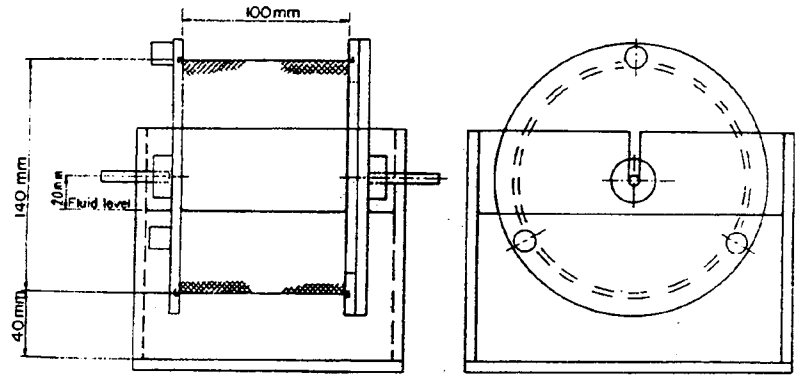

Figure 3.2: Standard Dimensions for Slake Durability Cage and Water Level

The cage is then rotated at 20 RPM for 10 minutes, removed from the apparatus and put back into the oven. After about 16 hours of drying, the cage and sample is weighed then the process is repeated. The slake durability index (I_d) is defined as the percentage ratio of final weight after two cycles to initial dry weight of the material (Franklin and Chandra, 1972). This test is important to the bridge scour problem.

As seasonal water fluctuations occur, the bedrock is exposed to yearly wetting and drying cycles along the banks. From field observations, as the stream recedes, the exposed bedrock undergoes significant drying and the surface starts to crack and flake due primarily to capillary stresses caused by desiccation. When winter rains bring the streams near or past flood levels the weakened, weathered rock is easily eroded. The amount of rock that is washed away is dependent on the depth into the bedrock that the drying occurred. This is a steady process that occurs every year but cannot be quantified due to the variability of wetting and drying, and the inherent variability of the rock mass.

When a rock mass remains permanently saturated or at it's natural water content, then the rock strength increases significantly from the same material that has been dried in

the past. Furthermore, rocks that normally slake from wetting and drying remain intact (Morgenstern and Eigenbrod, 1974). If the bedrock in the channel is saturated, then scour resistance is higher than the rock exposed to wetting and drying. The ASTM Slake Durability Test is representative of the bedrock exposed to wetting and drying, but not representative of the bedrock saturated year round. Therefore, the rate of scour in the stream channel cannot be accurately estimated with the current ASTM Slake Durability Test. A modification to the ASTM test has been made to account for the rock continuously saturated in the channel. The procedure utilized herein involves the following:

1. The sample size should be 500g ± 50g and the pieces are between 12.5 to 25 mm (1/2 to 1 in) in diameter, similar to the ASTM method, however the sample is now kept in a saturated state until the test. Prolonged soaking could be detrimental to the sample because of softening effects, so soaking should not be more than 24 hours. (Morgenstern and Eigenbrod, 1974).
2. Wet the cage and dry off excess water on the top and bottom. Weigh the wet cage for a tare value. Lightly dry off the excess water from the rock pieces, place them into the cage and weigh.
3. Fill the reservoir with tap or distilled water to the same levels as prescribed in ASTM for the Slake Durability test. Turn the cage for 500 ± 20 minutes at 20 RPM.
4. After 30 minutes turn the motor off and take the cage out of the water. Place at an angle to let the water inside the cage drain for 30 to 60 seconds. Remove the lid and lightly hand dry the cage the same way as in the beginning of the test when the cage was first weighed. Repeat the procedure and subject the rock fragments to 30 minutes of rotation. Take weights of the cage and rock every 30 minutes for the first two hours then every hour until 480 to 500 minutes.
5. Calculate the percent weight loss (equation 3-1) and plot against time.

$$\% \text{ Weight Loss} = \frac{C - B}{B - A} \tag{3-1}$$

A = Initial weight of wetted cage with no material

B = Initial weight of wetted cage with rock sample, time = 0 minutes.

C = Weight of wetted cage and rock sample at time > 0 minutes.

Instead of expressing final weight to initial weight as described in the ASTM standard, the result is plotted as a percent weight lost versus time. This continuous slake test represents the abrasion resistance of the rock as the fragments are abraded against the cage and bouncing off other rock fragments. From this plot the Continuous Slake Number (β) developed for use in this study can be calculated.

3.2.3 Continuous Slake Number β

The Continuous Slaking Test allows for the calculation of an index property for rock. Looking at the shape of the plot for Tyee Sandstone at Alsea-Thissel (Figure 3.3), the curve increases quickly due to the wearing off of sharp edges, then the plot begins to level off at around 120 to 200 minutes as the edges become smooth.

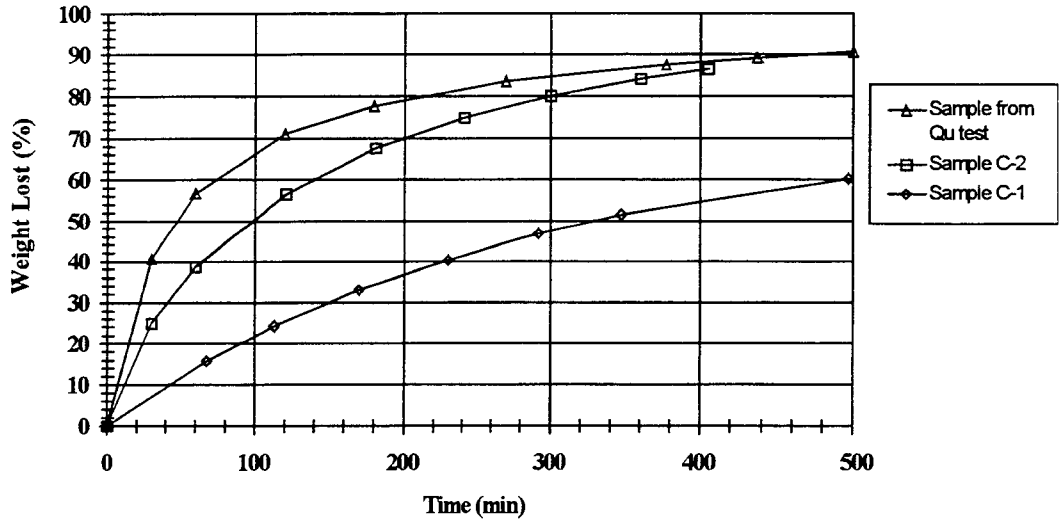

Figure 3.3: Plot of results of Continuous Slake Test for Alsea-Thissel

After about 120 minutes, two of the three curves become essentially parallel. The top curve has a smaller slope due to varying hardness within the core. This is why it is recommended to do multiple tests on the same core. Similarly, for different rock types, the curves are essentially parallel after 120 to 200 minutes. The weight loss data obtained after 120 minutes plotted against time (logarithmic) yields a straight line. This slope of the straight line portion of the test is defined as the *Continuous Slake Number* (β), as shown in Figure 3.4.

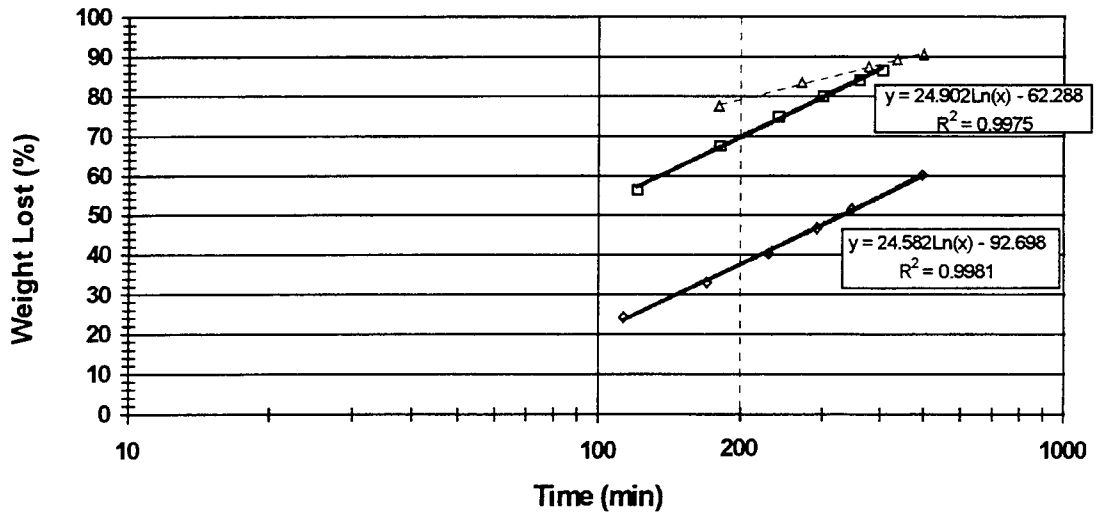

Figure 3.4: Semi-log plot of Continuous Slake Test for Alsea-Thissel

The lines are of the form:

$$Y = \beta * Ln\,(X) + B \qquad (3\text{-}2)$$

Where β is the Continuous Slake Number and B is the Y-intercept. Since the regression is for points after 200 minutes, B is not equal to zero. The B value represents the initial changes in weight loss, with large values for pieces that abrade quickly, and low values for highly abrasion resistant materials whose edges do not chip off easily (i.e. unweathered basalt). The rocks generally fall into particular values with basalts and very hard rocks varying from 1 to 10, sandstones from hard to soft, from 10 to 20, and soft siltstones and shales, 20 to 30 or more. This test can also be used to determine weak materials that may

appear strong. For example, the Tyee Sandstone exposed at the sites investigated herein is classified as weak rock. Field investigations give high recovery and RQD values from 70 to 100%. Based on the unconfined compression testing, it ranked about the same as other sandstones and the ASTM Slake Durability lists the material as high to very high. However, the β-value is between 20 and 25, indicating that the abrasive resistance of the sandstone is low, which could be potentially hazardous. This demonstrates that the material would provide good foundation support ($q_u \sim 40$ MPa). However, over time, enough high flow events could undermine the footing through scour due to abrasion.

It is acknowledged that the slake durability apparatus is not commonly contained in standard geotechnical labs. In order to assist bridge engineers, a simple relationship between β and the saturated density of the rock cores has been developed (Figure 3.5). This plot demonstrates that as the saturated density increases, the abrasive resistance increases.

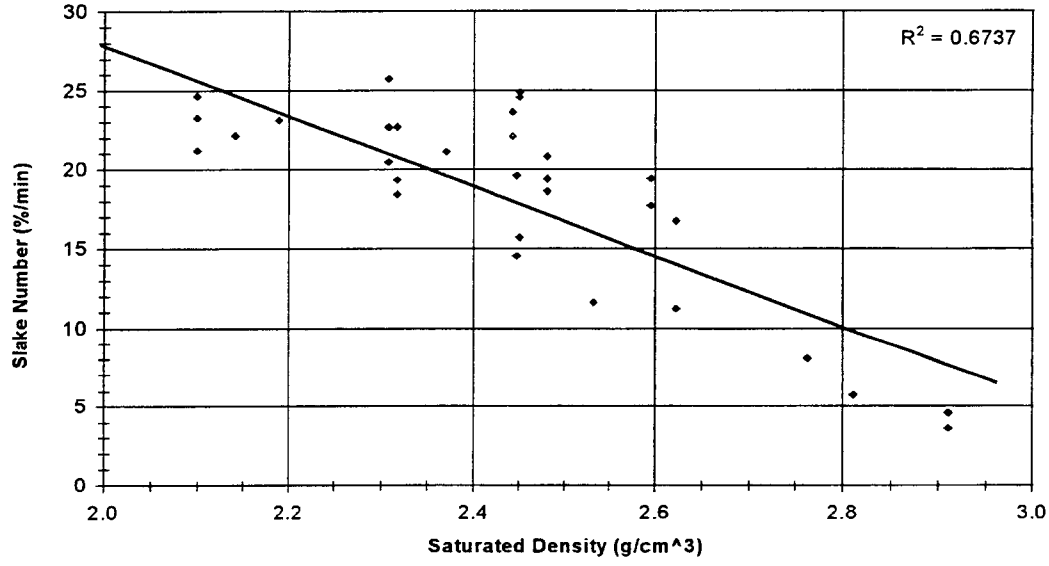

Figure 3.5: Relationship of Continuous Slake Number (β) and Saturated Density

3.2.4 Density

The density of the material is important to its abrasive resistance. The abrasion resistance has been demonstrated to vary linearly with density (Goodman, 1989). Therefore, densities were measured for all the rock samples. These values ranged from 2.0 to 3.0 g/cm^3 (125 to 190 pcf). All the densities are saturated densities which relate well with the continuous slaking test that uses saturated materials.

3.2.5 LA Abrasion Test

The objective of the LA Abrasion test is to determine the durability of gravel or crushed rock. The procedure includes taking a representative sample of a known gradation of aggregate or gravel (about 5 to 10 kg per gradation size), insert it into a large steel drum along with 10 steel balls of known size and rotate the drum 500 revolutions. The steel balls will wear down the rock to a point where the gradation is changed. The rock is deemed acceptable if the gradation has not changed beyond a certain percent. This test is difficult to perform on rock core samples for several reasons. First, the sample is a cylinder that needs to be pulverized with a hammer before insertion into the drum. Second, the sample is too small for the size of the drum and the number of balls. Without drilling another 10 to 15 holes, the sample size will be around 500 g per gradation size. Third, oven drying some of the samples makes the rock extremely brittle. This means at the end of the 500 revolutions, some of the rocks become completely destroyed to a point where none of the original gradation is left. This tells a little about the effect of wetting and drying, but the slake durability test is a better indicator of that property. Overall this test shows a difference between rock hardness, but is difficult to quantify and reproduce.

3.3 WETTING AND DRYING EFFECTS

Everything else being equal, the ASTM Slake Durability test has been found to provide very useful material properties for the determination of the amount of erosion that will be seen in a channel. For the purpose of this study, only the measurements in the saturated rock were used. As the plot for Mill Creek - HWY 22 shows (Figure 3.6), in twenty minutes the specimen tested per ASTM D4644 completely wore down so that nothing remained in the cage. This corresponds to a Slake Durability Index of 1.0.

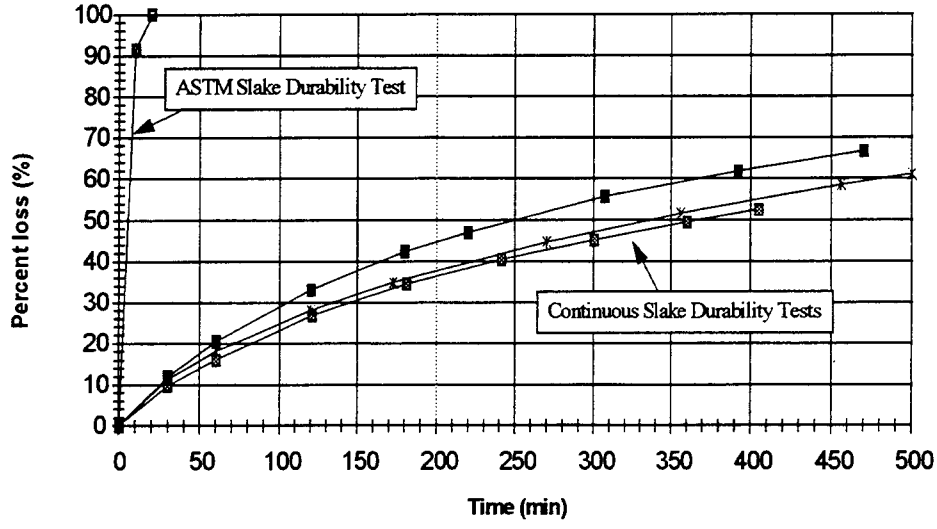

Figure 3.6: Continuous Slake Results for Mill Creek @ HWY 22

This is a marked difference between dry and wet material. On a bridge site, the seasonal fluctuations of the stream will cause wetting and drying effects along the edges of the channel. At the Rosenbalm site, the bridge footings were located on rock within this wetting and drying zone, therefore, the footing was progressively exposed due to bedrock erosion. The solution to this problem involved installing drilled shafts into the

bank where the siltstone is not exposed to drying, and the footing is not exposed to the stream. The bank was also protected with a rip-rap and shotcrete armor to reduce lateral stream migration.

4. HYDRAULIC INVESTIGATION

After selecting and surveying the sites, and sample collection and laboratory tests were performed, a hydraulic study of each site was required. The study included evaluation of cross-sections and computer modeling using the Corps of Engineers HEC-RAS backwater program (USACE, 1997). Stream gages near the sites provided flow information during the time interval of interest (i.e. from the date of the first cross-section to the date of the most recent survey).

4.1 STREAM GAGES

Stream gage data was obtained from the United States Geological Survey via the Internet (USGS, 1997). In several cases, available gage data was supplemented with correlations to other stream data for the time interval between the two surveys. One of the more difficult tasks in site selection was due to the location of stream gages to the site. Several potential sites had to be discounted because daily stream flow values could not be obtained or synthesized with confidence.

4.2 ADJUSTING AND SYNTHESIZING DAILY FLOW VALUES

Adjustments to the flow data obtained from stream gages were required for sites located far enough away to make a significant change in the drainage area. The adjustments were made based on the procedures outlined in the USGS/ODOT "Magnitude and Frequency of Floods in Western Oregon" (Harris, et.al., 1979). From this report, design flow or peak discharge is estimated using one of three methods based on the drainage area of the site and the drainage area of the gage. The first method is for a difference in drainage area less than 5 percent. In this case stream gage data is not

adjusted. If the difference in drainage area is within 5 to 25 percent the flow adjustment is:

$$Q_u = Q_g*(A_u/A_g)^a \qquad (4\text{-}1)$$

Where; Q_u is the ungaged discharge, Q_g is the gaged discharge, A_u is the ungaged drainage area, A_g is the gaged drainage area, and *a* is a drainage area exponent from the regression equations in the USGS/ODOT report.

When the drainage area difference is greater than 25 percent, the report recommends using the regression equations that are based on precipitation intensity, forest cover, areas of lakes, and drainage area. This method gives estimates of flow for the 2, 5, 10, 25, 50, and 100 year events with 30 to 40 percent standard error (Harris, et.al., 1979). Unless all the sites were located close enough to the stream gage to allow for less than 25 percent difference, then site selection would be extremely limited. Therefore, an adjustment to the gages using drainage area ratio (equation 4-1) provided the best estimate for daily flow. It is understood that the stream flows are estimates only and many sites are clearly outside the recommended 25 percent limit in drainage area difference. However, the necessity for daily values justified using this method.

Most site's gaps in the flow data (e.g. the stream gage was closed prior to the end of this study) necessitated the synthesis of flow data. In several instances, flow data was synthesized from a local gage, or from averages of two or more local gages within the same basin. This was achieved by comparing the available stream records with other stream records located in the same drainage basin or nearby, where there are similar features as described in the USGS regression equations (Harris, et. al., 1979). The site flow data was then plotted against the nearby stream data and a linear regression produced a best fit line with an equation and R^2 value. An example is provided Figure 4.1 for Five Rivers and Alsea River. If the R^2 value was too low ($R^2<0.70$), or in other words, if the scatter was too great then another stream record or an average of stream records in the basin was used. Most streams where synthesis was required produced R^2 values greater than 0.87 and all were greater than 0.70.

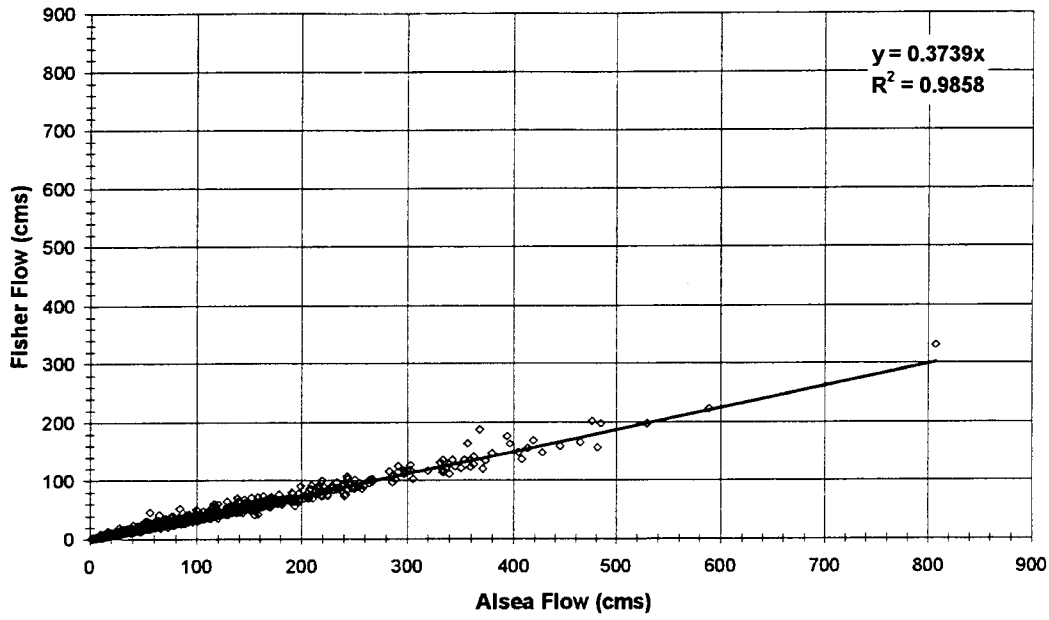

Figure 4.1: Regression of Alsea River at Tidewater and Five Rivers Near Fisher Daily Flows

4.3 EVALUATION OF HYDRAULIC VARIABLES

Hydraulic variables that were studied included flow volume, stream power, shear, and velocity. The variables were calculated using the Army Corps of Engineers HEC-RAS version 2.0 backwater analysis program (USACE, 1997). The stream model was based on the channel cross-sections surveyed for this study and longitudinal profiles obtained from USGS 7.5 minute quadrangle maps. Some of the rivers studied were large enough to make cross-sectioning away from the bridge very difficult without the use of a boat. To evaluate the flow through the bridge in the numerical model, the section on the upstream side of the bridge was copied and placed 100 feet upstream and downstream at an elevation difference that correlated with the slope. For consistency, only 7.5 minute quadrangle maps were used to estimate the slopes of the streams. This is similar to the procedure commonly used by ODOT engineers during the analysis and design of bridge foundations. Although smaller streams could easily be surveyed for longitudinal bed profiles, difficulty in surveying the slopes of larger rivers made this approximation necessary.

It is assumed that the slope of the stream is equal to the energy gradient of the stream. This is a common assumption, although at times erroneous in the following cases (a) non-uniform, subcritical flow where the energy slope is larger than the bed slope and (b) non-uniform supercritical flow where the energy grade is less than the bed slope (Annandale, 1995). After the cross sections were entered at the adjusted elevations, and Manning's ***n*** values were estimated for the channel and the overbank sections, different flows ranging from the lowest observed value to the highest observed value were run through the model. From this suite of analysis, a table of variables including stream power, shear, velocity, and Froude number was created. Each variable was plotted against the representative flow and relationships were calculated. Figure 4.2 demonstrates this by plotting stream power against flow. All of these plots created relationships with an R^2 of 0.95 to 1.0. The equation of the best fit allowed a conversion of daily stream flow (Figure 4.3) to daily stream power (Figure 4.4), or daily velocity, or daily shear. From these daily values, sediment transport can be evaluated, and by using the daily power, scour can be estimated. This will be explained in more detail in Chapter 5.

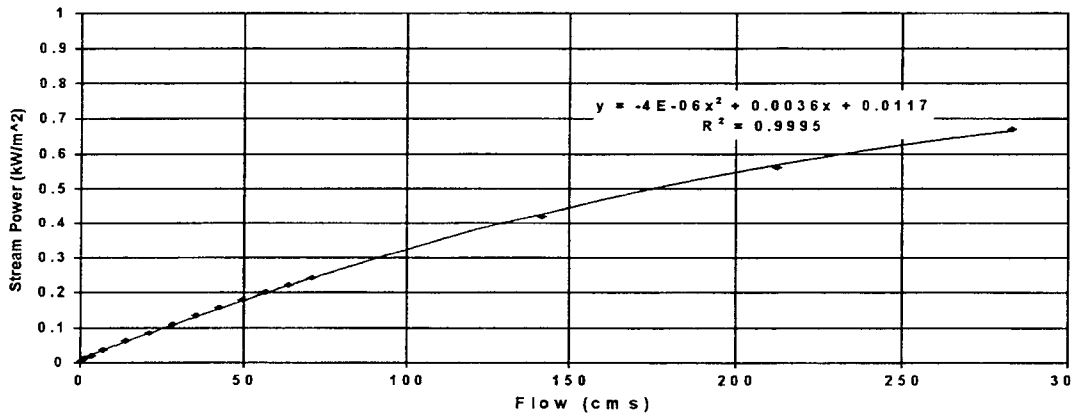

Figure 4.2: Flow versus Stream Power for Five Rivers Near Fisher

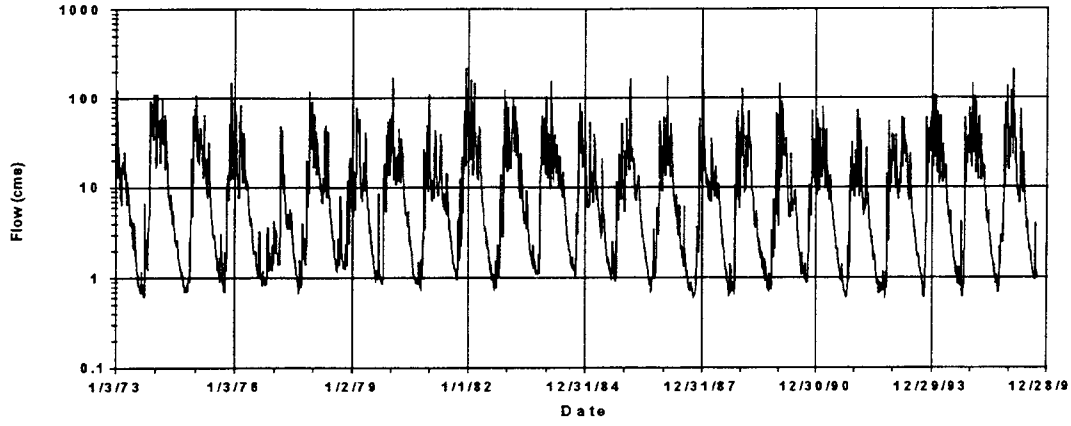

Figure 4.3: Daily Flow at Five Rivers Near Fisher

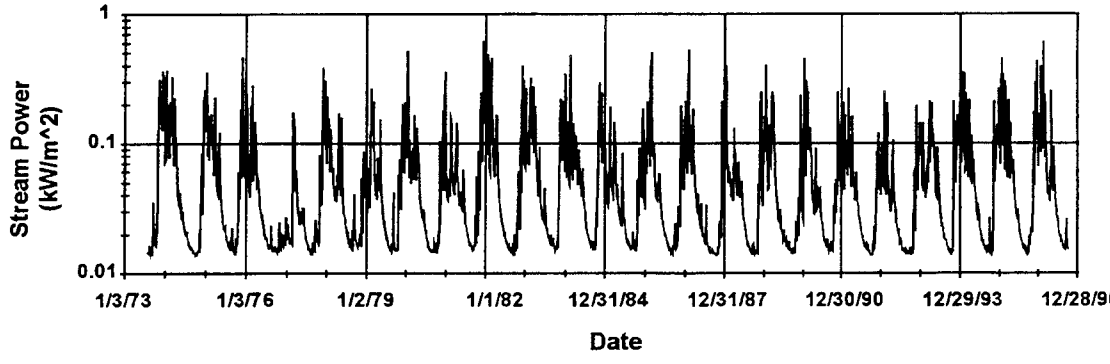

Figure 4.4: Daily Stream Power for Five Rivers Near Fisher

4.4 EFFECT OF SLOPE ON STREAM POWER

As shown in Figure 4.2 to Figure 4.4, stream power is calculated by correlating the stream flow to stream power. From Annandale (1995), the stream power equation is as follows:

$$P = \gamma q s_f L \tag{4-2}$$

where P is the unit stream power γ is the unit weight of water, q is the unit discharge, s_f is the energy grade and L is unit length. Since it is assumed that s_f is equal to the slope of the bed (s_o), a substitution of variables in equation 4-2 yields:

$$P = \gamma q s_o L \tag{4-3}$$

From this equation it is now evident that the slope is an important variable in determining the stream power. Figure 4.5 shows the variations that can occur between surveyed slopes and scaled slopes.

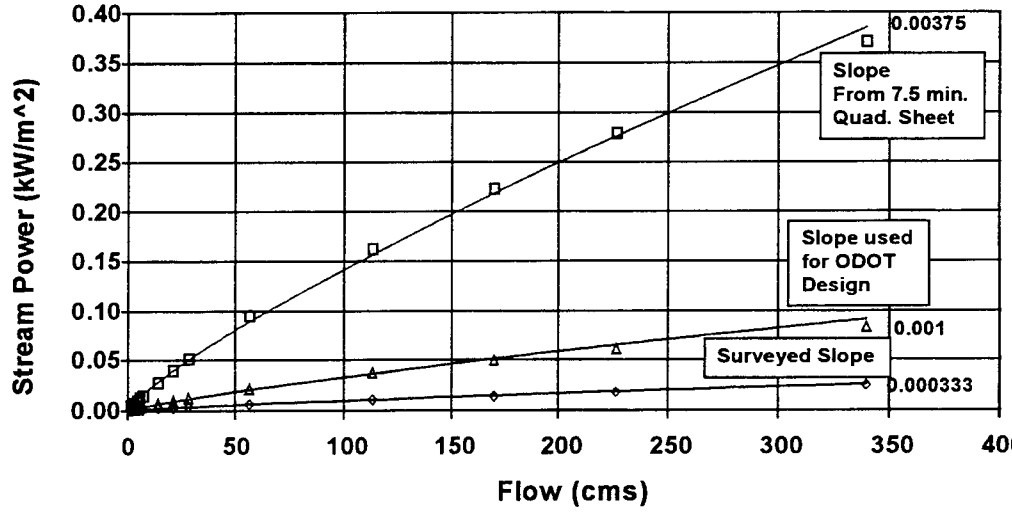

Figure 4.5: Effect of Various Slopes on Stream Power at Luckiamute

From Figure 4.5 it is evident that the slope can effect the stream power at high flows, where the most scour is likely to occur. In this case the use of the 7.5 minute quadrangle sheet for estimating the slope provides a higher value for stream power. For the sites included in this study, it appears that this overprediction of stream power is consistent at most or all of the sites. Again the use of the 7.5 minute quadrangle sheets is adopted for use herein for the following reasons: (a) consistency in channel slope estimates and (b) this follows the methods sometimes employed by ODOT engineers performing hydraulics calculations. Surveying the channel bottom can lead to surveying errors that can change the slope enough to overestimate or underestimate the power. An

elevation difference of 2.75 mm (0.11 inches) in 1000 meters can change the stream power from the middle curve in Figure 4.5 to the top curve. This sensitivity leads the engineer to use judgment in deciding the appropriate slope for design. The authors used the 7.5 minute quadrangle sheets for consistency in measurement.

4.5 DISCUSSION OF HYDRAULIC STUDY

In order to efficiently and consistently perform the hydraulic calculations, the following simplifications were required in the hydraulic study:

- modeling the bridges with prismatic sections upstream and downstream
- only the most current cross-sections were used to model the stream
- using the 7.5 minute quadrangle sheet to estimate the slope
- the slope is equal to the energy gradient

The stream model was based on a copied cross-section of the upstream side of the bridge adjusted for slope. This was checked against a smaller stream (Mill Creek - Rosenbalm) where a full cross-section of upstream and downstream was taken. This was modeled with HEC-RAS and the resulting regression equations were compared to the equations produced using copied sections. The results were not significantly different, therefore the copied sections from one cross-section on the upstream side of the bridge were used. This comparison was performed on a single span bridge where pier and contraction effects are not seen. However, if there are piers and contraction effects, a full cross-section should be used. Furthermore, all sections used in the model were from the most recent cross-sections. The model section should be changing with time, since that is what is this study is trying to prove. However, the only available sections were before and after. Therefore, the final section was used as the researchers are confident of the cross-section measurements and it most represents current conditions. The other two assumptions about the slope are addressed in Section 4.2.

5. STATISTICAL ANALYSIS OF GEOTECHNICAL AND HYDRAULIC DATA

5.1 INTRODUCTION

In an attempt to develop a useful, practical method to estimate the depth of scour in bedrock, all geotechnical and hydraulic data required a statistical analysis. In order to establish a viable model of scour, a significant population of site data is required. However, the small sample size (i.e. eleven data points) limited the statistical significance of the relationship. While the incorporation of additional sites would significantly improve the confidence in the model, the current study of eleven sites forms a useful start towards the development of a screening tool for bridge engineers.

The values summarized in Table 5.1 represent the geotechnical and hydraulic variables used in the statistical study. The final model uses average erosion as the response variable, and the independent variables are Integrated Stream Power (Ω) and the Continuous Slake Number (β).

5.2 BEDLOAD

Depending on the time of the year that the stream is investigated and the flow conditions, most of the sites contained visible bedrock with bedload. It is assumed that bedload of a similar gradation is being transported across the bedrock of all the sites during the year. Although bedload samples from adjacent point bars were collected for comparative studies of gradation, the characteristics of the bedload as a function of the stream power were not investigated. Assuming a similar gradation on all sites, the differences in stream power and velocity at individual sites determined which particles were in suspension and which were abrading.

Table 5.1: Variables Used in the Statistical Study

Site	Dates of Observation	Average Amount of Erosion (mm)	Density (g/cm^3)	Slake Durability ASTM (%)	Slake Durability β	Unconfined Compression (MPa)	Integrated Stream Power (Ω) (kN/mm)	Average Power (kW/m^2)	Average Flow (cms)
Mill Creek									
HWY22	9/18/80 to 10/13/96	57.2	2.10	0.0	23.1	0.9	5170.2	0.010	50.7
Rosenbalm	4/4/90 to 4/18/97	96.5	2.17	0.3	24.8	0.9	5038.0	0.023	83.3
Yaquina									
M.P. 2.4	7/27/76 to 10/13/96	57.6	2.31	3.2	23.0	1.8	4021.4	0.005	40.3
M.P. 4.9	7/27/96 to 10/13/96	0.0	2.32	73.6	20.2	43.0	1070.2	0.001	34.7
Alsea									
Missouri Bend	12/11/78 to 10/15/96	170.9	2.44	95.0	22.9	39.9	10915.7	0.019	241.2
Thissel Rd.	9/1/87 to 10/18/96	181.2	2.45	73.6	21.9	43.6	9856.1	0.031	400.8
Five Rivers									
Fisher	8/1/73 to 10/1/96	362.6	2.45	96.5	16.3	35.6	44921.7	0.061	146.6
Nestucca									
Powder Creek	12/12/95 to 6/26/97	171.9	2.81	99.8	5.1	N/A	807.6	0.016	345.9
Mid. Fork Coquille									
M.P. 51	11/2/81 to 10/14/96	79.8	2.59	72.2	18.6	40.7	8032.9	0.017	40.2
M.P. 53	11/2/81 to 10/14/96	114.0	2.62	97.9	14.0	38.3	12027.3	0.022	26.1
Luckiamute									
Grant Rd.	6/20/84 to 9/12/97	134.6	2.37	N/A	21.6	N/A	5440.3	0.013	84.2

β - Continuous Slake Number

5.3 COMPARISON OF CROSS-SECTIONS

The erosion value obtained from the cross section is the most important variable, as it is the response variable, or the value that will dictate the results. The recent cross-sections were plotted against the initial cross sections, and an the change in channel elevation was used to determine the amount of erosion. The resulting erosion was calculated using two different methods. The first method was to find the area of the displaced material between the two sections, then divide it by a width of the stream. This would give the result as a depth. The second method was to take the average depth over the width of the stream. The width used for both methods was the distance across the stream where the rock remains saturated. This correlated to the low flow condition for the stream. The saturated width is important because it takes away any wetting and drying effects that would give a bias interpretation. As explained in Section 3.3, the wetting and drying effects weaken the rock which would significantly affect the average erosion. The width could be any width, but the wider width gives a better representation of the stream, since the stream power is calculated over the entire channel cross-section. Some sites had localized conditions that affected the average. For instance, at Mill Creek at Rosenbalm, there is a 1 meter (3 ft.) drop in the middle of the stream over a seven year period. However when the average drop for the saturated width of the stream is calculated, the drop is not as severe.

When comparing cross-sections from 1940 for Mill Creek - HWY 20, for example, the section shows a flat river bed and the footing buried into what was termed "soapstone". This bridge was re-sectioned in 1980 for a project that involved widening the bridge. When the 1980 cross-section was compared to the 1940 cross-section, the "soapstone" (now called shale) was no longer present. Upon observation, there is no evidence to substantiate or deny that the footing was buried into the shale, therefore the section from 1980 was used. The Nestucca - Powder Creek site is the only site that was not re-measured with soundings. In December 1995 this site was sounded during high flow, and the water level was compared to a staff gage located at this bridge. In June 1997, the cross-section was obtained by measuring the depth from the water surface

(assumed to be level) to the bedrock using a fiberglass elevation rod. The water surface was then measured in relation to the staff gage. After normalizing the cross-sections to a constant water surface, the depths could be compared.

5.4 DEVELOPMENT OF AN EMPIRICAL MODEL

In order to create a relationship between the variables and average erosion, a review of the scour process is required. This research is focusing on the abrasive characteristics of sediment transport. The first model was developed to calculate the total amount of kinetic energy over the duration of the study from different grain sizes abrading the bedrock. This method does not involve Integrated Stream Power (Ω) or the Continuous Slake Number (β), however, it involves the energy of different size particles as they abrade the bedrock. The final model relates the kinetic energy to the average erosion.

This was achieved by assuming spherical grain sizes from 0.25 mm (fine sand) to 96 mm (cobbles). Mass was then estimated using an assumed specific gravity of 2.65. The critical velocity for incipient motion was calculated for each grain size. Now, with mass and velocity, kinetic energy is calculated for each grain size and multiplied by the number of days in the study. Table 5.2 shows an example of this method for Alsea-Thissel. From this information it is evident that the coarse gravel provides the most abrasive energy, and that the smaller grain sizes provide lower energy, even though they are abrading over a longer duration of time.

Table 5.2: Energy Model for Alsea-Thissel

Classification		Average Size (mm)	Critical Shear (psf)	Velocity* (ft/sec)	Max. Velocity** (ft/sec)	Time (Days)	Mass (kg)	Velocity (m/s)	Energy (Joules)	Energy-Days (Joule-day)
Sand	Fine	0.25	0.0035	0.143	0.385	0	0.0000	0.043442	0.0000	0.000
	Medium	1.21	0.0155	0.385	0.918	0	0.0000	0.117299	0.0000	0.000
	Coarse	3.38	0.057	0.918	1.432	0	0.0001	0.279765	0.0000	0.000
Gravel	Fine	5.5	0.111	1.432	2.396	1156	0.0002	0.436515	0.0000	0.025
	Medium	12	0.24	2.396	3.838	1055	0.0024	0.730361	0.0006	0.675
	Coarse	24	0.486	3.838	6.095	942	0.0192	1.169704	0.0131	12.361
	V. Coarse	48	0.972	6.095	9.668	185	0.1535	1.857863	0.2648	48.993
Cobbles	Small	96	1.94	9.668	19.336	0	1.2276	2.946823	5.3301	0.000
									Total :	62.05

* Velocity from shear velocity relationship specific to site

** Maximum Velocity before particle goes into suspension

Figure 5.1 shows how the energy-days plots with average erosion. The dashed line represents a tentative best-fit estimate. This plot shows a reasonable fit, however the concept does not involve any information regarding the bedrock. Figure 5.1 represents the energy involved in sediment transport which is assumed to be transferred entirely to the bedrock upon impact during saltation. This is half of the problem because a weaker rock would wear down faster on lower energy impact than a rock such as basalt which would take high energy. This leads to the next step, combining geologic and hydraulic data.

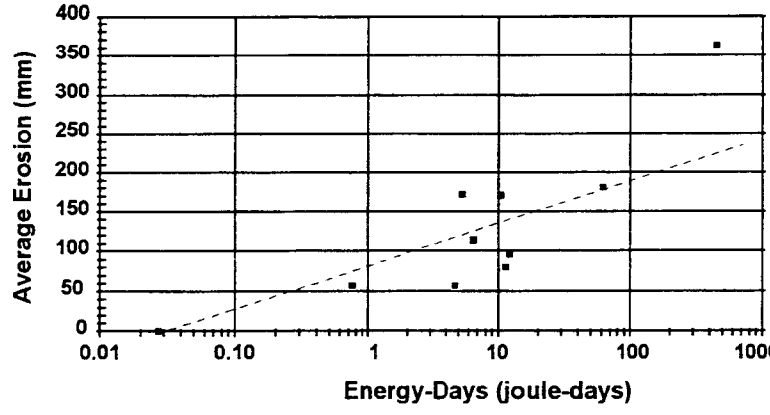

Figure 5.1: Plot of Average Erosion Compared with Energy-Days

Figure 5.1 shows how the energy-days plots with average erosion. The dashed line represents the author's best-fit estimate. This plot shows a reasonable fit, however the concept does not involve any information regarding the bedrock. Figure 5.1 represents the energy involved in sediment transport which is assumed to be transferred entirely to the bedrock upon impact. This is half of the problem because a weaker rock would wear down faster on lower energy impact than a rock such as basalt which would take high energy. This leads to the next step, combining geologic and hydraulic data.

In setting up a relationship, the variables with the highest significance needed to be found. A statistical package (SAS) was used to discover the significant variables through stepwise linear regression analysis. When all the variables were entered with average erosion as the dependent variable, only two of the independent variables remained, the continuous slake number (β), and the integrated stream power (Ω). The average erosion that made the best fit was the method of averaging depths over the width of the stream. Statistically, eleven sites is a small population, so the multivariate linear regression output of a linear model was disregarded, however the significant variables were used to create the final model. The variables β and Ω were individually plotted against average erosion in Figure 5.2.

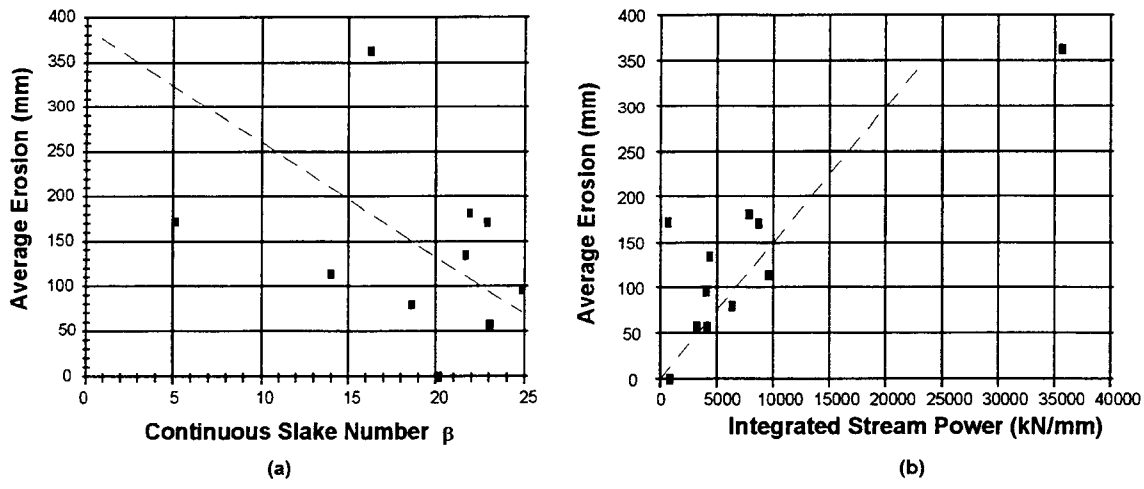

Figure 5.2: Plots of (a) Average Erosion versus β and (b) Average Erosion versus Integrated Stream Power

From Figure 5.2 it is evident that there are possible trends of average erosion with each independent variable. However, as with the kinetic energy model, only one variable is compared with average erosion, and the goal is to combine the hydraulic information with geotechnical information. When the values of β are placed on plot (b) three sets of contours emerge to produce Figure 5.3.

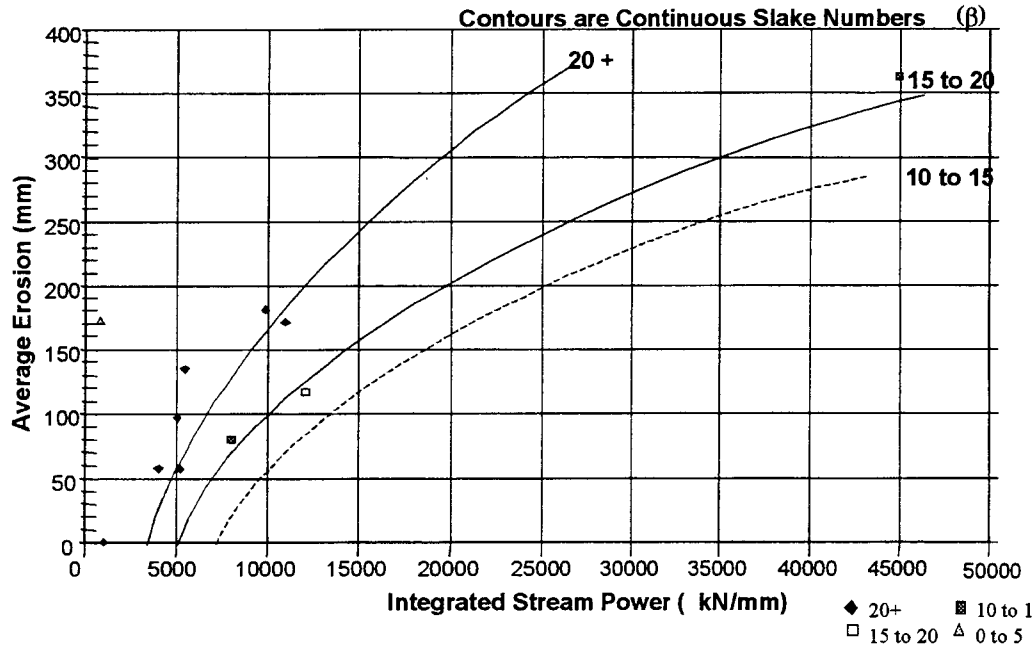

Figure 5.3: Average Erosion versus Integrated Stream Power (Ω) and Continuous Slake Number (β)

Figure 5.3 shows the combined geotechnical and hydraulic information. The dashed line represents an estimated contour based on judgement. It is noted that the contours do not originate at the origin, or zero stream power. As the rock gets weaker (β gets larger), less stream power is required to abrade or scour the rock. However, the rock still has enough strength to resist the abrasion and not scour. Take, for example, two different streams with different bedrock and the Integrated Stream Power equal to 5000 kN/mm. The first site has weak sandstone (β~20) and the other has a harder sandstone (β~15). From the chart, at the first site average erosion is about 55 mm (2 in.) while at site two there is no average erosion. It is only at some threshold that the rock will scour.

This threshold is estimated on this figure as the intersection of the contours with the X-axis.

The small population of sites in the statistical analysis did not allow for Figure 5.3 to be defined entirely by points. Between Integrated Stream Power values of 15000 and 45000, there are no points. These areas were fit based on engineering judgment. This estimation leads to some uncertainty. However, while more sites would strengthen the relationship, this figure is still useful as a screening tool. This figure combined with Figure 3.5 (Saturated Density vs β), may help the bridge engineer decide the type of footing or whether an existing bridge is in danger.

5.5 DISCUSSION

The goal of this pilot study was to determine variables which govern the process of erosion and scour of rock. The sample population is too small to establish a robust design method. The plot of average erosion and integrated stream power shows an area where there are no points. This is because the Fisher site is a high power stream of long record, placing a point outside the others. The addition of more points would strengthen the prepared relationship.

Important relationships were discovered in this research. It is apparent that high power streams will dislodge and move particles, depending on the Erodibility Index, and low power streams will wear the stream bed down with abrasion. From this there should be an approximate boundary where scour due to dislodgment and scour due to abrasion share an equivalent range of stream powers. The predominant mechanism for dislodgment is high stream powers combined with low Erodibilty Index material. Conversely, at low stream powers abrasion is the predominant characteristic. When evaluating the Nestucca site it is noted that the stream records are for a time period containing two, one-hundred year floods. This is significant because the characteristic stream power values are much lower. When an Erodibility Index is calculated, Nestucca has the lowest value aside from the sites with siltstone. The uncharacteristically high stream power from the floods between 1995 and 1997 could have caused dislodgment of fragments along with abrasion

(phase 2), increasing the scour depth even though there is a high Continuous Slake Number (β).

The independent variable of integrated stream power is based on a computer model of the cross-sections taken by the researcher. The only way to confirm the stream power that HEC-RAS calculates is to calibrate the stream elevations with known gage information. This would require a stream gauge at every site or daily height and velocity measurements of all the sites every day during the period of record. Since many of the streams have data synthesized from correlations with other streams, this was not possible. The second independent variable is the Continuous Slake Number. As in all testing of materials, representative samples are important. A weathered zone could cause an erroneous value of β which would overestimate scour. These zones of rock were not tested, as the rock in the stream channel was not exposed to the same weathering.

5.6 PROPOSED DESIGN APPLICATIONS

The proposed method could be used as a preliminary screening tool for estimating the depth of scour given easily determined geotechnical parameters (e.g. β or ρ) and the stream flow characteristics anticipated over the design life of the bridge. The following procedure is an example of how one can estimate the erosion of a channel.

1. A thorough geotechnical investigation of the potential bridge site with drilling to identify the rock type, discontinuities, RQD, recovery, etc. Coring with a drill rig is preferred over hand coring because the investigator can determine the RQD, percent recovery, and perform all the laboratory tests. The drilling should extend at least 3 m (10 ft) into the bedrock to insure proper identification of layering or weathered zones. Drilling both sides of the channel is recommended to insure representative characterization of the local bedrock.
2. Based on existing or synthesized flow data, establish the average and integrated stream power per year.

3. A standard hydraulic investigation of the river channel needs to be performed. This would include cross-sections and modeling with a computer modeling system such as HEC-RAS. Calculate the channel stream power for various flows starting with the lowest observed flow up past the highest flow. For example if the flow was between 1.5 cms to 170 cms (50 cfs to 6000 cfs) then the stream power would be calculated for 1.5, 2.0, 4.0, 8.0, 15.0, 30.0, 60.0, 100.0, 150.0, 200.0, 400.0 cms. These power values would then be plotted against the flow and a regression analysis performed to produce the best fit. For most cases, this best fit will be either a power relationship or quadratic relationship. Spreadsheets calculate the R^2 value and provide an equation. This equation can now be used to convert daily flow values into daily power values. Adjustments to the stream power can be applied to account for pier geometry and/or contraction effects.
4. After daily power is calculated, then calculate the area under the daily power by summing the areas. This is the Integrated Stream Power (Ω).
5. Perform the following laboratory tests on representative specimens of the rock. The unconfined compression test is a standard test for establishing the allowable bearing pressure of footings. The Slake Durability test will identify if the footing can be placed on rock that is going to be exposed to wetting and drying. The Continuous Slake Test will determine the long term effect of abrasion nearby and around the footing. If the RQD values and q_u values are low, then scour by dislodgment may be a concern.
6. To estimate the depth of scour over a selected period of time, determine the Integrated Stream Power (Ω) and find the average depth of erosion based on the Continuous Slake Number (β). Once the average depth has been established, apply an appropriate factor of safety. For example, in Figure 5.4 the Integrated Stream Power is 17500 kN/mm and the Continuous Slake Number is 16, then the average erosion would be about 200 mm (8 in).

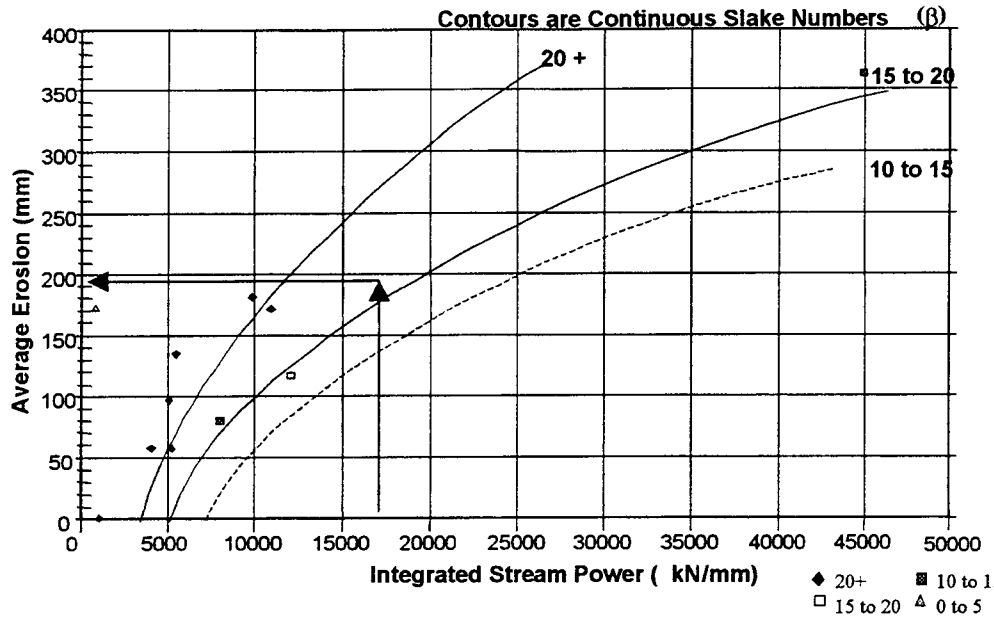

Figure 5.4: Estimating Average Erosion

5.7 EVALUATION OF EXISTING STRUCTURES

This preliminary procedure can be used for evaluating the current safety of bridge foundations on weak rock along with recommendations provided in the FHWA memorandum.

1. Research into early plans to identify the type of bedrock, and the geologic properties that were used in design. If this does not exist, then repeat Step 1 of the proposed design applications.
2. A thorough bridge investigation including visual inspection of all piers and footings, photographs and cross-sections or soundings of the stream channel. Compare this information to previous inspection reports and cross-sections. This information can provide a confirmation to Figure 5.3.
3. Take samples of the bedrock in the saturated condition, either with a truck mounted drill, or with a hand coring drill, and follow Step 5 of the proposed design applications.

4. Locate and/or synthesize appropriate daily flow from nearby stream gages for the remainder of the design life of the bridge.
5. Using the flow data, calculate daily power and integrated stream power for the remaining design life. Step 3 of the proposed design applications.
6. Follow the same procedure as described in the proposed design applications to estimate scour for the remaining design life of the bridge.

5.8 RECOMMENDATIONS FOR FURTHER RESEARCH

Currently no design method exists that includes both hydraulic and geotechnical information. This research is a first step in creating a relationship between the two. Since the research effort contained such a small sample population, further research is required. The following list are recommendations that may aid in future research on this topic.

1. To improve statistical significance of the proposed model, more sites need to be incorporated into the study. This could be facilitated by incorporating data from files of additional agencies (e.g. State DOTs, USGS, US Forest Service Districts, USACE, etc.).
2. Survey error can lead to a faulty erosion value, either to bedload or merely not taking the reading in the same location. Sonic methods now exist that would significantly reduce survey error. This does not necessarily solve the bedload elevation problem, but allows for precise readings that cannot be achieved with surveying.
3. One problem of this research was synthesizing flow data from nearby stream gages. Since this is a long term problem, a stream gage at every site to accurately record flow information over an extended period of time would be valuable. Doing so eliminates errors in estimating flow and allows for calibration of HEC-RAS to give accurate stream power estimations.
4. A more detailed bedload evaluation over various flow conditions could verify the assumptions made in this report, and possibly tighten the relationship. This could involve more sampling from different times of the year, and produce gradations related

to stream flow. Incorporating the gradations into the model could produce a more accurate estimation of average erosion.

5. The most significant variables could be evaluated by the use of flume studies or monitored field study sites with known bedrock, bedload, and controlled flow. This would allow the researcher to have control in evaluating the sensitivity of the different variables. Care should be taken to make sure the bedrock is not chemically reacting with the water, as has been documented with some limestones and dolemites.

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APPENDICES

APPENDIX A

SITE LOCATION

SITE CROSS SECTIONS
AND DATES OF STUDY PERIOD

Location Information

Site Name

Site Location	*Legal Site Location Description*
Quadrangle Sheet Name (USGS)	*Oregon 7 1/2 Minute Quadrangle Maps*

County

Geotechnical Information

Material Designation
Coring Method
Recovery/RQD
Density
Unconfined Compressive Strength
ASTM Slake Durability
Continuous Slake Number

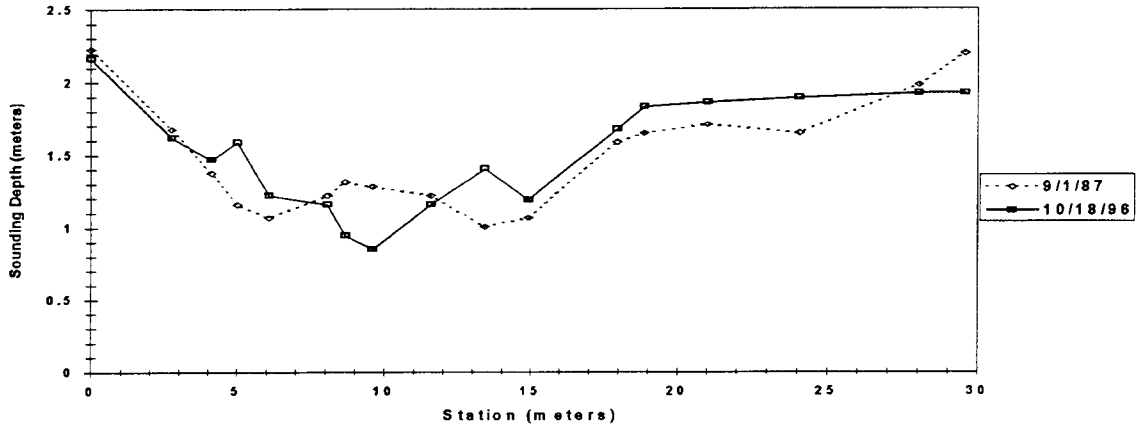
Hydraulic Information

USGS Stream Gage Name
USGS Stream Gage Number

Synthesized Data	*Time period that data was synthesized.*
Average Flow	
Average Velocity	*Averaged over*
Average Power	*the duration of the study*

Integrated Stream Power
Cross-Section Method
Previous Cross-Section

ALSEA AT THISSEL RD.



Location Information

Site Name	Alsea at Thissel Rd.
Site Location	NE1/4,NW1/4,Sec. 6,T14S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Hellion Rapids Quadrangle
County	Lincoln

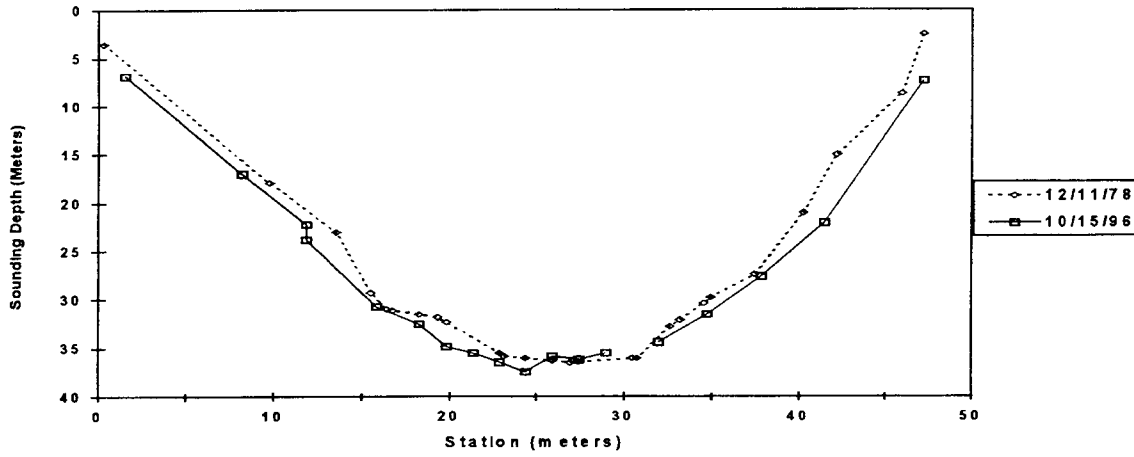
Geotechnical Information

Material Designation	Sandstone (Tyee)
Coring Method	Wire-Line
Recovery/RQD	96% / 77%
Density	2.45 g/cm (152.9 pcf)
Unconfined Compressive Strength	43.6 MPa (6322 psi)
ASTM Slake Durability	73.6%
Continuous Slake Number	21.9

Hydraulic Information

USGS Stream Gage Name	Alsea River near Tidewater
USGS Stream Gage Number	14306500
Synthesized Data	No
Average Flow	40.1 cms (1415.6 cfs)
Average Velocity	1.10 m/s (3.62 ft/s)
Average Power	0.032 kW/m^2 (2.16 lb/ft-sec)
Integrated Stream Power	9856 kN/mm (7817.8 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Siuslaw National Forest

ALSEA AT MISSOURI BEND



Location Information

Site Name	Alsea at Missouri Bend
Site Location	NE1/4,SW1/4,Sec. 13,T14S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Digger Mountain Quadrangle
County	Benton

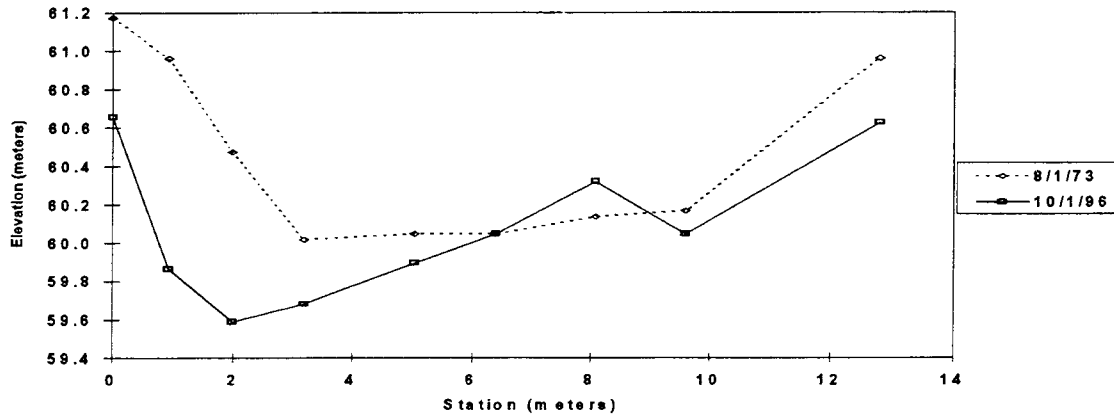
Geotechnical Information

Material Designation	Sandstone (Tyee)
Coring Method	Wire-Line
Recovery/RQD	96% / 68%
Density	2.44 g/cm (152.4 pcf)
Unconfined Compressive Strength	39.9 MPa (5783.7 psi)
ASTM Slake Durability	95.0%
Continuous Slake Number	22.9

Hydraulic Information

USGS Stream Gage Name	Alsea River near Tidewater
USGS Stream Gage Number	14306500
Synthesized Data	No
Average Flow	241.2 cms (851.8) cfs
Average Velocity	0.84 m/s (2.77 ft/s)
Average Power	0.016 kN/mm (2.16 lb/ft-sec)
Integrated Stream Power	10915.7 kN/mm (8658.3 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Siuslaw National Forest

FIVE RIVERS NEAR FISHER



Location Information

Site Name	Five Rivers near Fisher, Oregon
Site Location	NW1/4,SE1/4,Sec. 1,T15S,R10W,W.M.
Quadrangle Sheet Name (USGS)	Five Rivers Quadrangle
County	Lincoln

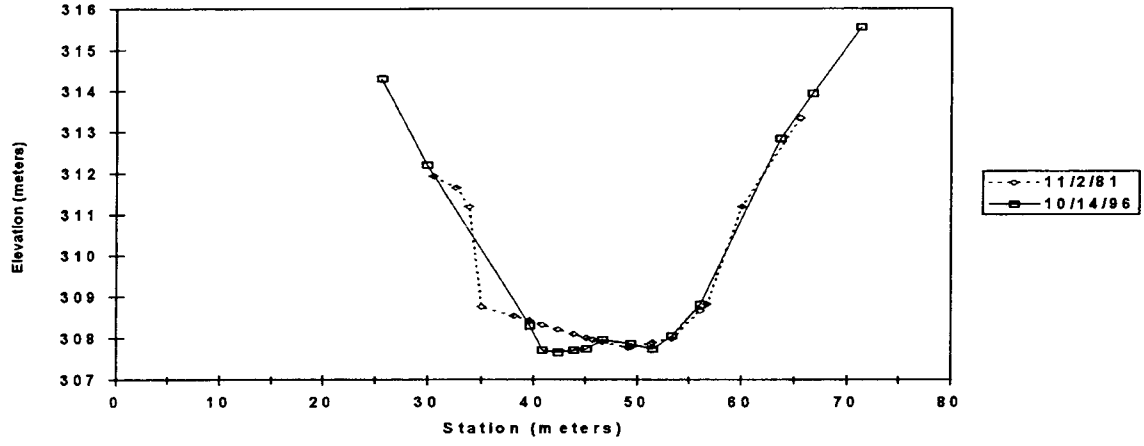
Geotechnical Information

Material Designation	Sandstone (Tyee)
Coring Method	Wire-Line
Recovery/RQD	95% / 82%
Density	2.45 g/cm (152.7 pcf)
Unconfined Compressive Strength	35.6 MPa (5159.7 psi)
ASTM Slake Durability	96.5%
Continuous Slake Number	16.3

Hydraulic Information

USGS Stream Gage Name	Fiver Rivers near Fisher, Oregon
USGS Stream Gage Number	14306400
Synthesized Data	1990 - 1996
Average Flow	146.6 cms (517.6 fps)
Average Velocity	1.39 m/s (4.56 ft/s)
Average Power	0.061 kW/m^2 (4.20 lb/ft-sec)
Integrated Stream Power	44921.7 kN/mm (35631.8 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

MIDDLE FORK COQUILLE @ M.P. 51



Location Information

Site Name	M.F.C. 51
Site Location	SW1/4,SW1/4,Sec. 36,T29S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Camas Valley Quadrangle
County	Douglas

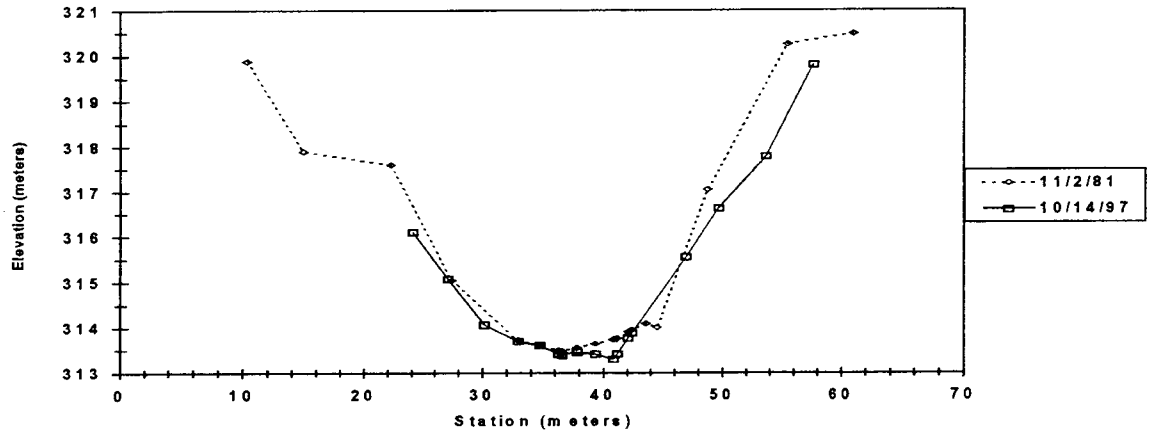
Geotechnical Information

Material Designation	Coarse Sandstone
Coring Method	Wire-Line
Recovery/RQD	100% / 98%
Density	2.59 g/cm (161.9 pcf)
Unconfined Compressive Strength	40.7 MPa (5895.9 psi)
ASTM Slake Durability	72.2%
Continuous Slake Number	18.6

Hydraulic Information

USGS Stream Gage Name	Mid. Fork Coquille River near Myrtle Point
USGS Stream Gage Number	14326500
Synthesized Data	1981-1996
Average Flow	40.2 cms (142.0 cfs)
Average Velocity	0.73 m/s (2.4 ft/s)
Average Power	0.017 kW/m^2 (1.13 lb/ft-sec)
Integrated Stream Power	8032.9 kN/mm (6371.7 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

MIDDLE FORK COQUILLE @ M.P. 53



Location Information

Site Name	M.F.C. 53
Site Location	SE1/4,NE1/4,Sec. 36,T29S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Camas Valley Quadrangle
County	Douglas

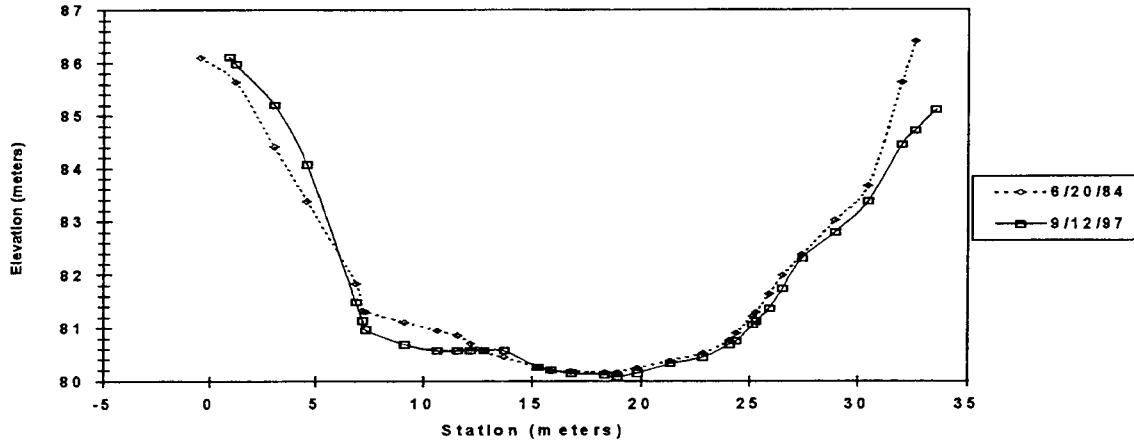
Geotechnical Information

Material Designation	Sandstone
Coring Method	Wire-Line
Recovery/RQD	81% / 72%
Density	2.62 g/cm (163.6 pcf)
Unconfined Compressive Strength	38.3 MPa (5558 psi)
ASTM Slake Durability	97.9%
Continuous Slake Number	14.0

Hydraulic Information

USGS Stream Gage Name	Mid. Fork Coquille River near Myrtle Point
USGS Stream Gage Number	14326500
Synthesized Data	1981-1996
Average Flow	26.1 cms (92.1 cfs)
Average Velocity	0.68 m/s (2.2 ft/s)
Average Power	0.022 kW/m^2 (1.53 lb/ft-sec)
Integrated Stream Power	12027.3 kN/mm (9540 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

LUCKIAMUTE RIVER @ GRANT RD.



Location Information

Site Name	Luckiamute River @ Grant Rd.
Site Location	NW1/4,NW1/4,Sec. 9,T10S,R6W,W.M.
Quadrangle Sheet Name (USGS)	Kings Valley Quadrangle
County	Polk

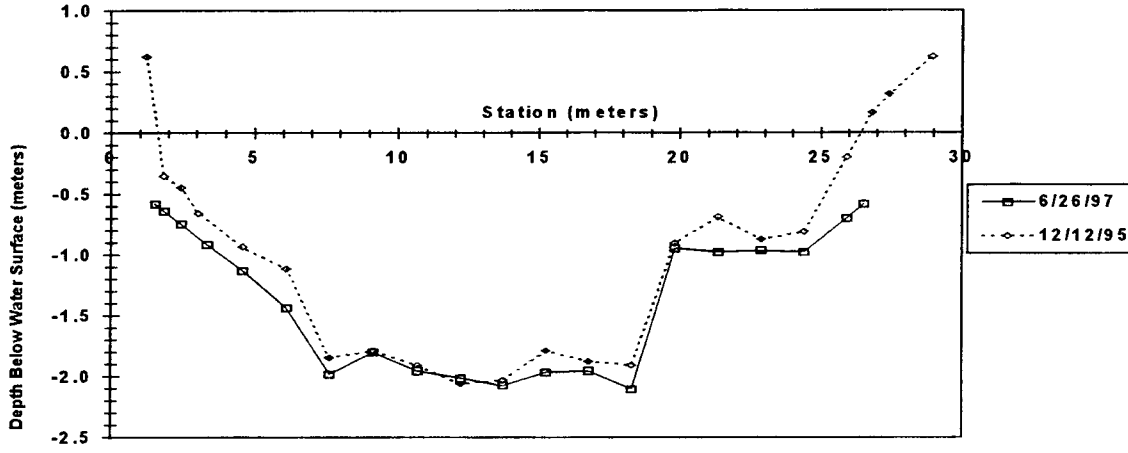
Geotechnical Information

Material Designation	Sandstone (Tyee)
Coring Method	Hand Core
Recovery/RQD	N/A
Density	2.37 g/cm (147.9 pcf)
Unconfined Compressive Strength	N/A
ASTM Slake Durability	N/A
Continuous Slake Number	21.6

Hydraulic Information

USGS Stream Gage Name	Luckiamute River near Pedee, Oregon
USGS Stream Gage Number	14190000
Synthesized Data	1984-1997
Average Flow	84.2 cms (297.3 cfs)
Average Velocity	0.32 m/s (1.07 ft/s)
Average Power	0.013 kW/m^2 (.89 lb/ft-sec)
Integrated Stream Power	5440.3 kN/mm (4315 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

NESTUCCA RIVER @ POWDER CREEK



Location Information

Site Name	Nestucca River at Powder Creek
Site Location	NE1/4,SW1/4,Sec. 3,T4S,R8W,W.M.
Quadrangle Sheet Name (USGS)	Blaine Quadrangle
County	Tillamook

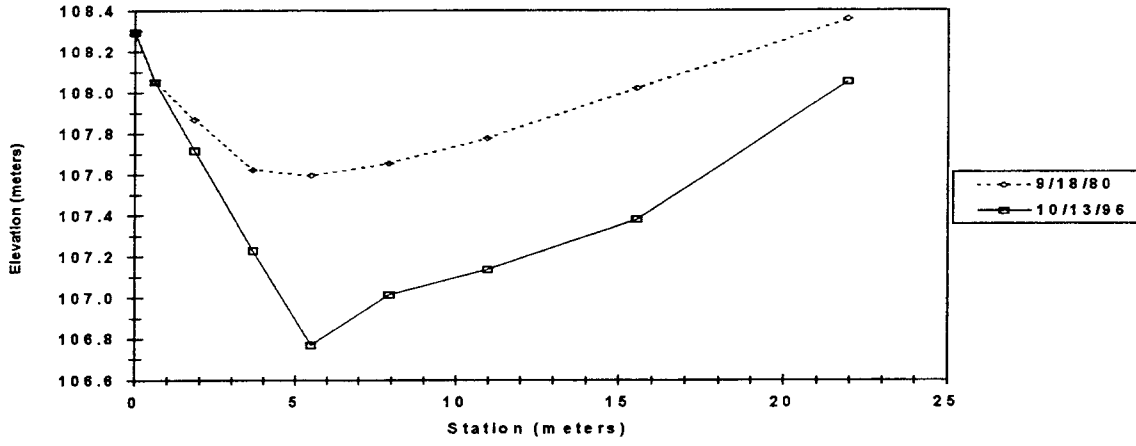
Geotechnical Information

Material Designation	Tuff
Coring Method	Hand Core
RQD	70
Density	2.81 g/cm (175.4 pcf)
Unconfined Compressive Strength	12.7 MPa (1837 psi)
ASTM Slake Durability	99.8%
Continuous Slake Number	5.1

Hydraulic Information

USGS Stream Gage Name	Nestucca River near Beaver, Oregon
USGS Stream Gage Number	14303600
Synthesized Data	1995-1997
Average Flow	345.9 cms (1221.4 cfs)
Average Velocity	0.83 m/s (2.72 ft/s)
Average Power	0.016 kW/m^2 (1.10 lb/ft-sec)
Integrated Stream Power	5440.3 kN/mm (4315 lb/ft)
Cross-Section Method	Depth Measurement with Survey Rod
Previous Cross-Section	Oregon Department of Transportation

MILL CREEK @ HIGHWAY 22



Location Information

Site Name	Mill Creek @ HWY 22
Site Location	NE1/4,SE1/4,Sec. 28,T6S,R6W,W.M.
Quadrangle Sheet Name (USGS)	Sheridan Quadrangle
County	Polk

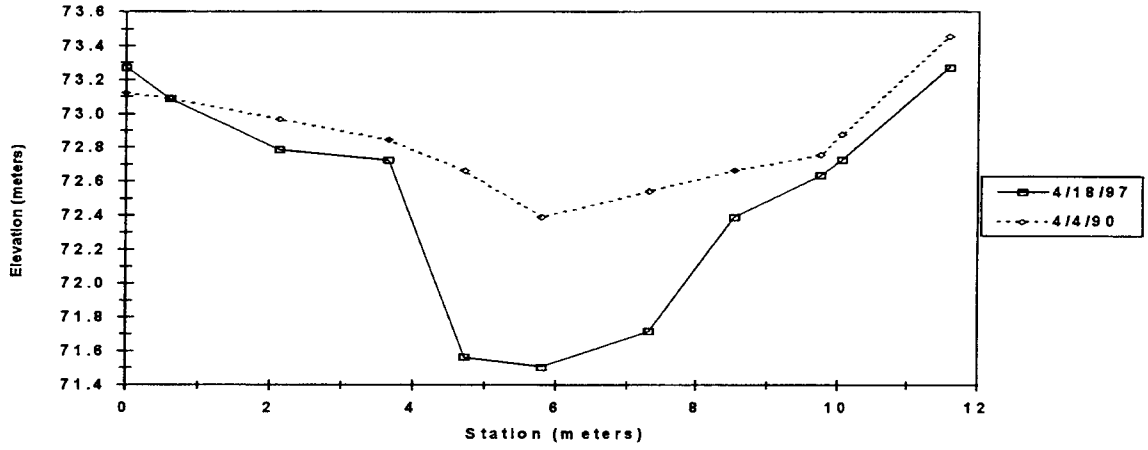
Geotechnical Information

Material Designation	Siltstone
Coring Method	Wire-Line
Recovery/RQD	98% /93 %
Density	2.10 g/cm (131.0 pcf)
Unconfined Compressive Strength	0.9 MPa (126.2 psi)
ASTM Slake Durability	0.0%
Continuous Slake Number	23.1

Hydraulic Information

USGS Stream Gage Name	Mill Creek near Willamina, Oregon
USGS Stream Gage Number	14193300
Synthesized Data	1980-1996
Average Flow	50.7 cms (178.9 cfs)
Average Velocity	0.70 m/s (2.3 ft/s)
Average Power	0.010 kW/m^2 (0.70 lb/ft-sec)
Integrated Stream Power	5170.2 kN/mm (4101 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

MILL CREEK @ ROSENBALM



Location Information

Site Name	Mill Creek @ Rosenbalm Rd.
Site Location	NW1/4,NW1/4,Sec. 9,T6S,R6W,W.M.
Quadrangle Sheet Name (USGS)	Sheridan Quadrangle
County	Polk

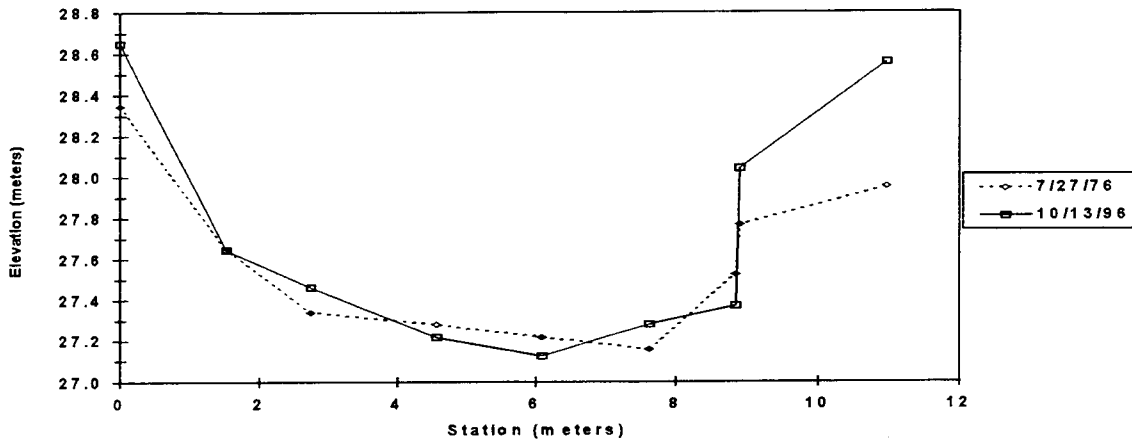
Geotechnical Information

Material Designation	Siltstone
Coring Method	Wire-Line and Hand Core
Recovery/RQD	97% / 81%
Density	2.17 g/cm (135.1 pcf)
Unconfined Compressive Strength	0.9 MPa (126.2 psi)
ASTM Slake Durability	0.3%
Continuous Slake Number	24.8

Hydraulic Information

USGS Stream Gage Name	Mill Creek near Willamina, Oregon
USGS Stream Gage Number	14193300
Synthesized Data	1990-1996
Average Flow	83.3 cms (294.3 cfs)
Average Velocity	1.0 m/s (3.3 ft/s)
Average Power	0.023 kW/m^2 (1.55 lb/ft-sec)
Integrated Stream Power	5038.0 kN/mm (3996 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

YAQUINA RIVER @ M.P. 2.4



Location Information

Site Name	Yaquina M.P. 2.4
Site Location	SE1/4,SW1/4,Sec. 35,T10S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Eddyville Quadrangle
County	Lincoln

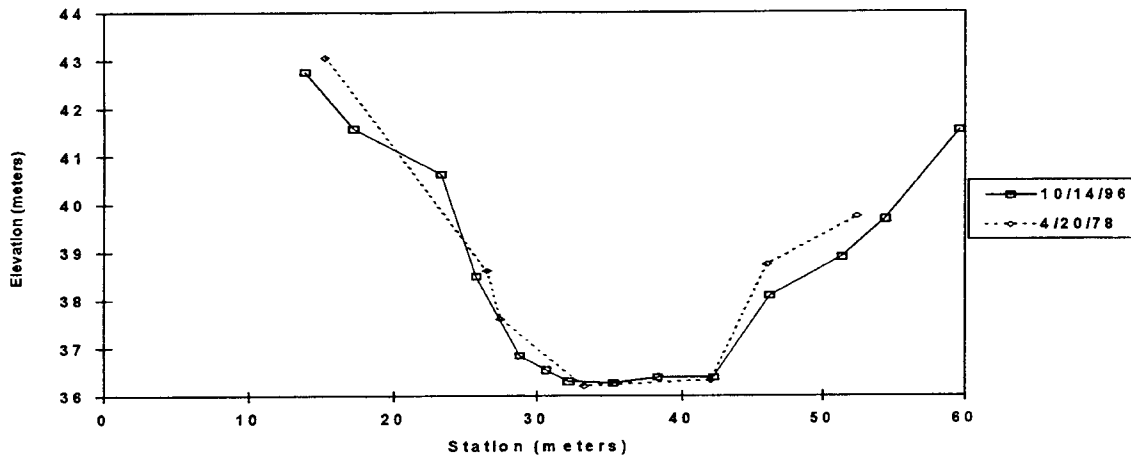
Geotechnical Information

Material Designation	Siltstone
Coring Method	Wire-Line
Recovery/RQD	96% / 80%
Density	2.31 g/cm (144.0 pcf)
Unconfined Compressive Strength	1.8 MPa (256.7 psi)
ASTM Slake Durability	3.2%
Continuous Slake Number	23.0

Hydraulic Information

USGS Stream Gage Name	Yaquina River near Chitwood, Oregon
USGS Stream Gage Number	14306030
Synthesized Data	1991-1996
Average Flow	40.3 cms (142.2 cfs)
Average Velocity	0.46 m/s (1.5 ft/s)
Average Power	0.005 kW/m^2 (0.36 lb/ft-sec)
Integrated Stream Power	4021.4 kN/mm (3190 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

YAQUINA RIVER @ M.P. 4.9



Location Information

Site Name	Yaquina M.P. 4.9
Site Location	SE1/4,SW1/4,Sec. 36,T10S,R9W,W.M.
Quadrangle Sheet Name (USGS)	Eddyville Quadrangle
County	Lincoln

Geotechnical Information

Material Designation	Fine Sandstone
Coring Method	Wire-Line
Recovery/RQD	87% / 75%
Density	2.32 g/cm (144.6 pcf)
Unconfined Compressive Strength	43.0 MPa (6234 psi)
ASTM Slake Durability	73.6%
Continuous Slake Number	20.2

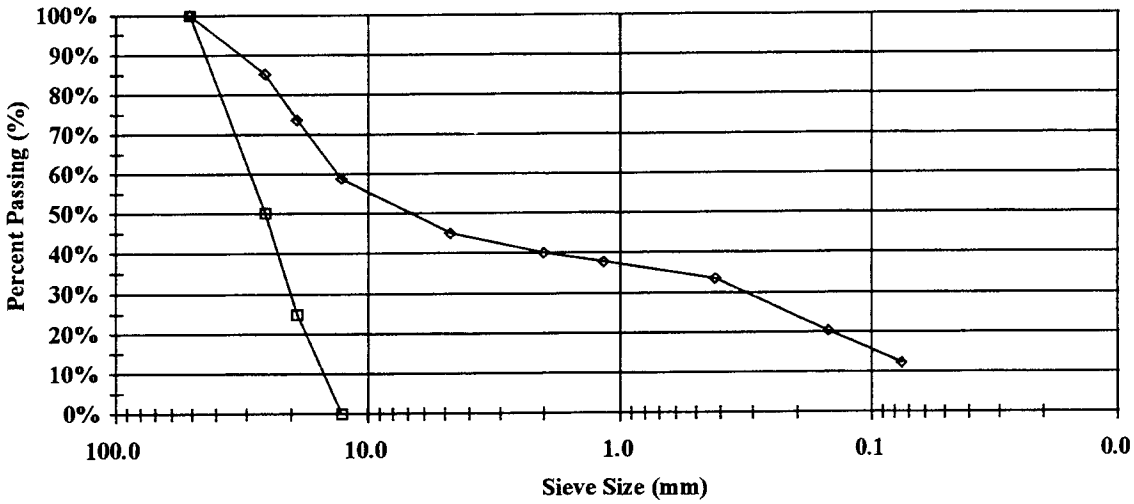
Hydraulic Information

USGS Stream Gage Name	Yaquina River near Chitwood, Oregon
USGS Stream Gage Number	14306030
Synthesized Data	1991-1996
Average Flow	34.7 cms (122.4 cfs)
Average Velocity	0.36 m/s (1.17 ft/s)
Average Power	0.001 kW/m^2 (0.10 lb/ft-sec)
Integrated Stream Power	1070.2 kN/mm (848.9 lb/ft)
Cross-Section Method	Soundings
Previous Cross-Section	Oregon Department of Transportation

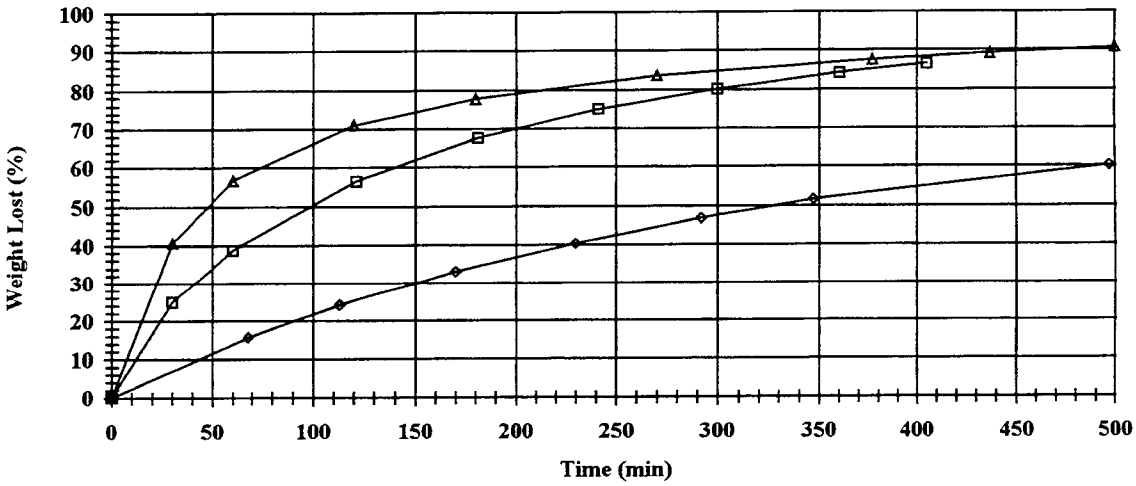
APPENDIX B

ALSEA - THISSEL

LA Abrasion for Alsea - Thissle

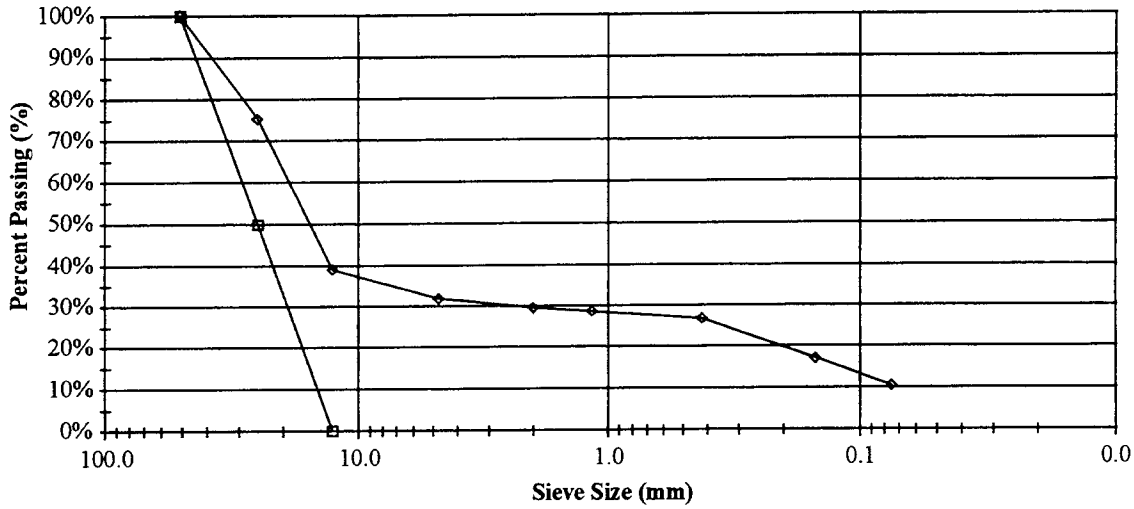


Continuous Slake Test For Alsea - Thissle

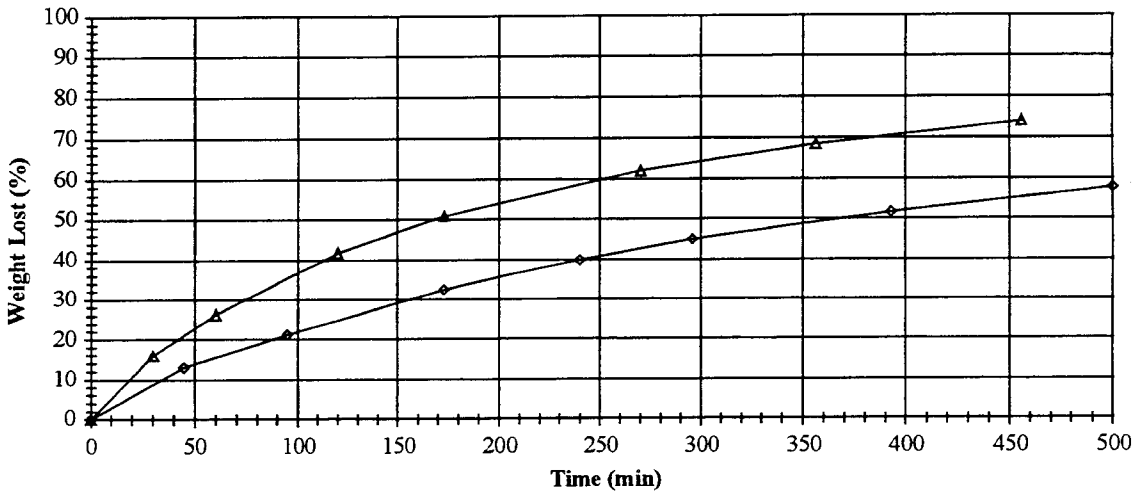


ALSEA - MISSOURI BEND

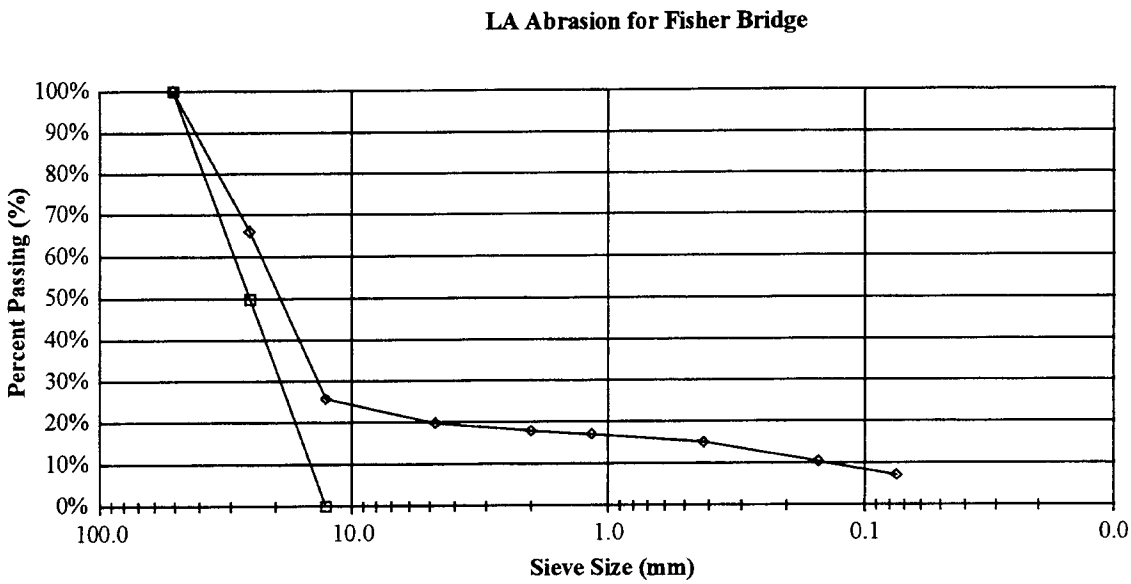
LA Abrasion for Alsea - Missouri Bend

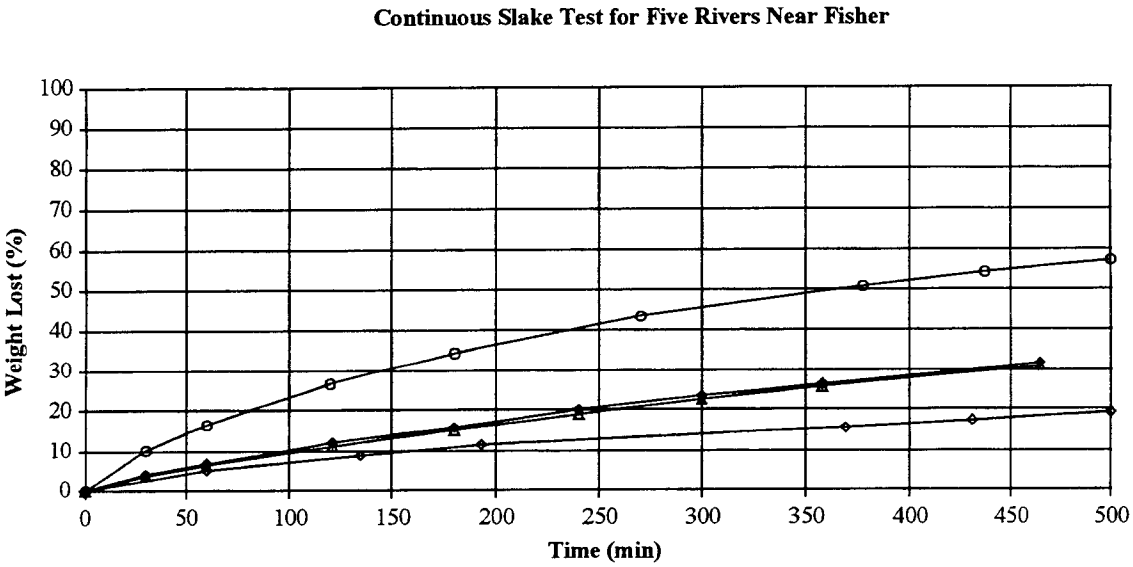


Continuous Slake Test for Alsea - Missouri Bend

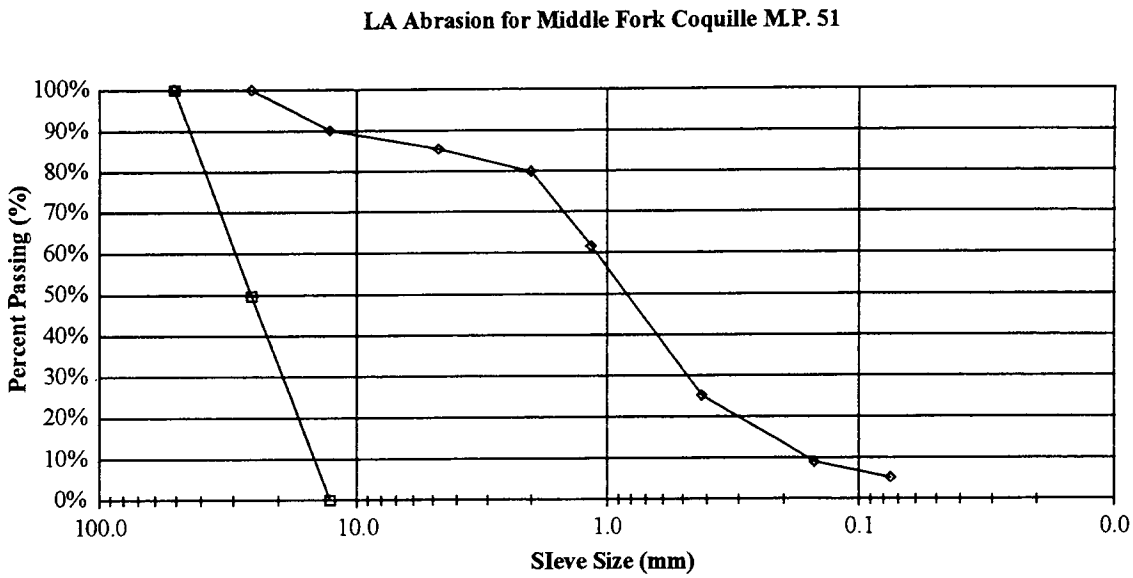


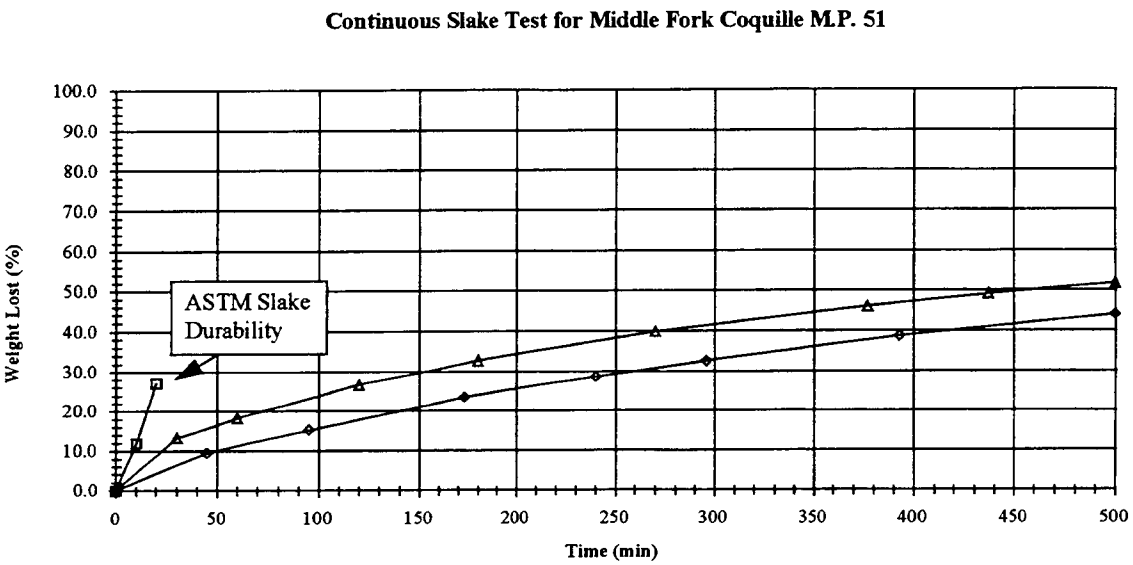
FIVE RIVERS NEAR FISHER





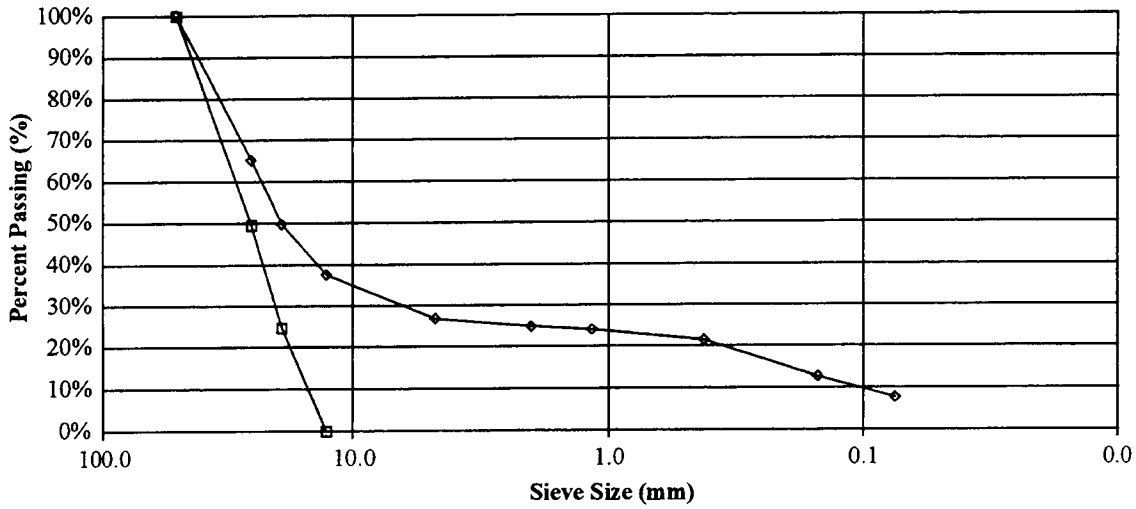
MIDDLE FORK COQUILLE @ M.P. 51



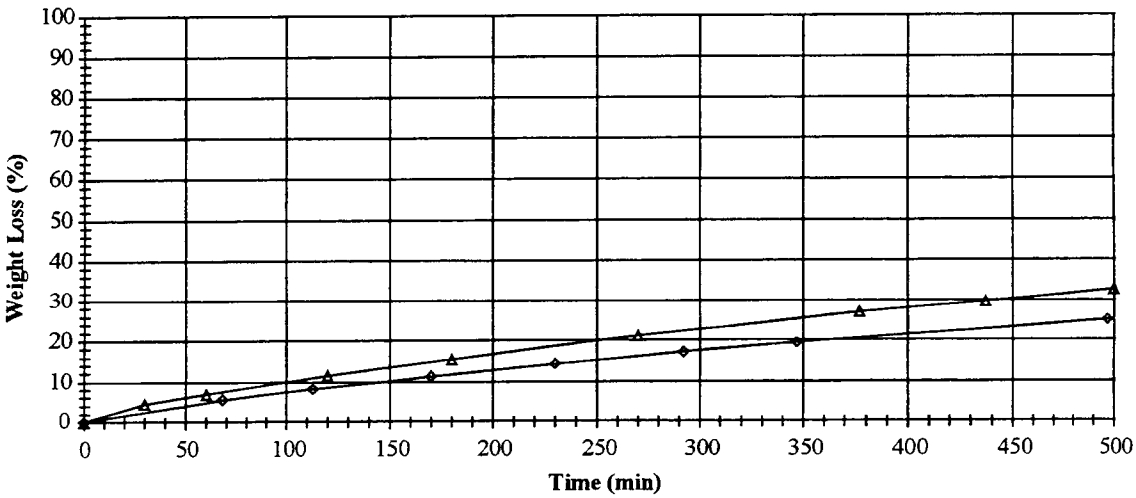


MIDDLE FORK COQUILLE @ M.P. 53

LA Abrasion for Middle Fork Coquille M.P. 53

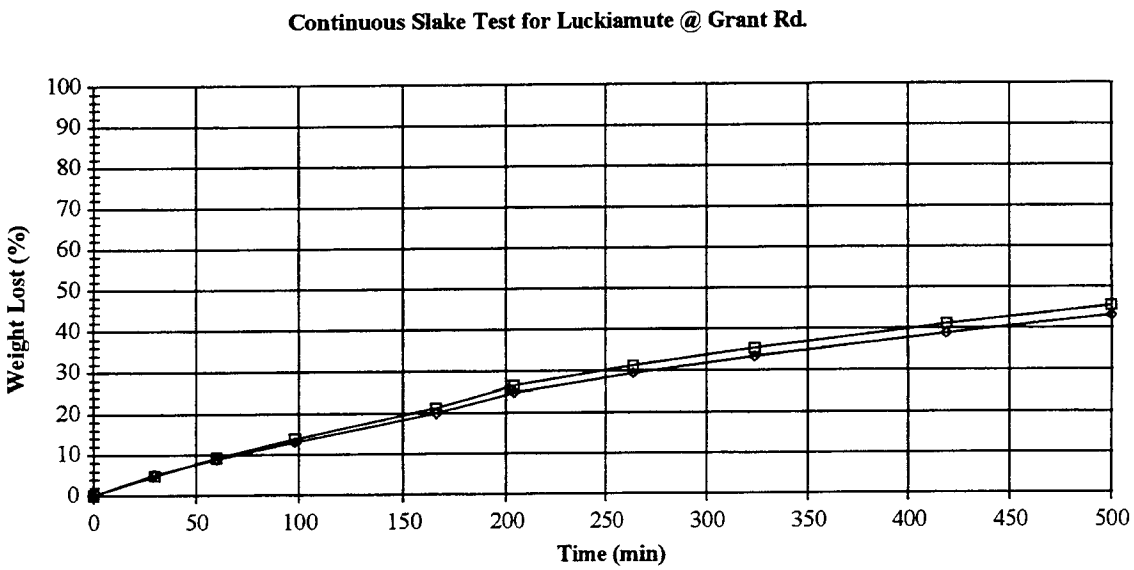


Continuous Slake Test for Middle Fork Coquille M.P. 53



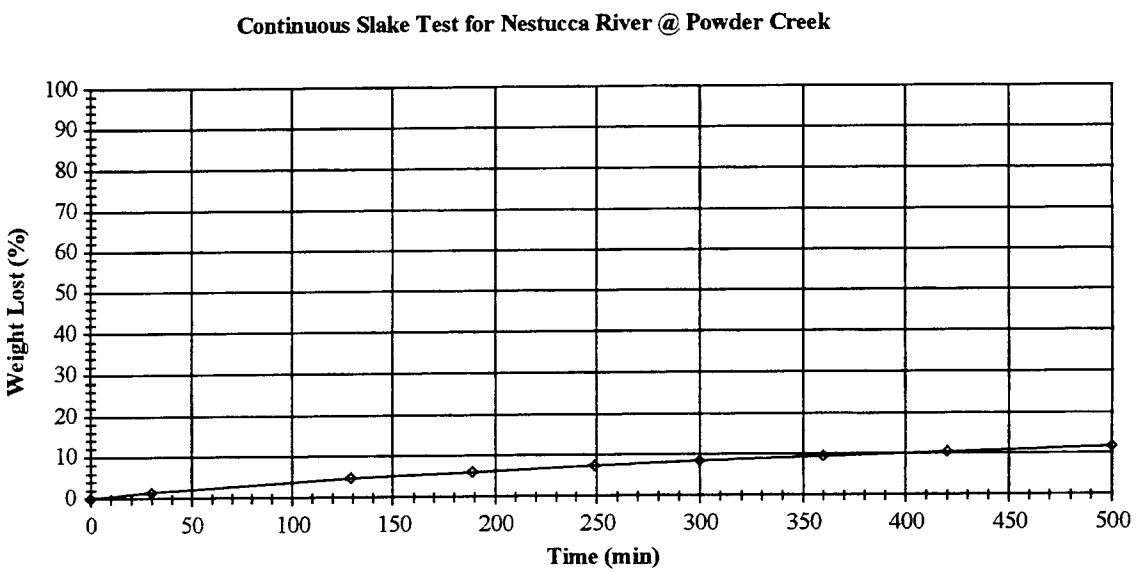
LUCKIAMUTE RIVER @ GRANT ROAD

LA Abrasion Test Not Performed for Luckiamute @ Grant Rd.

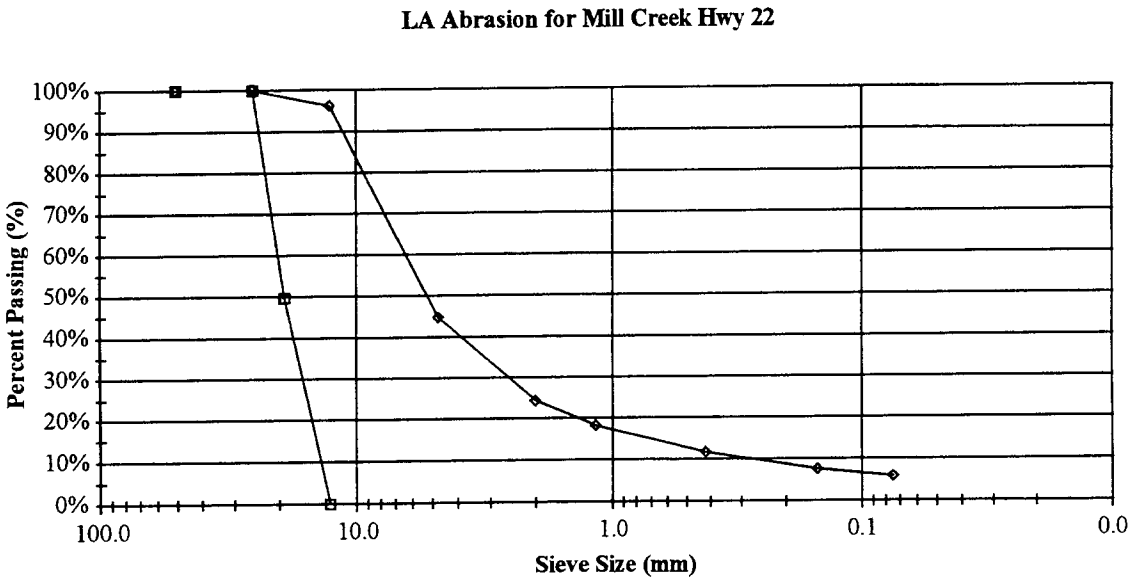


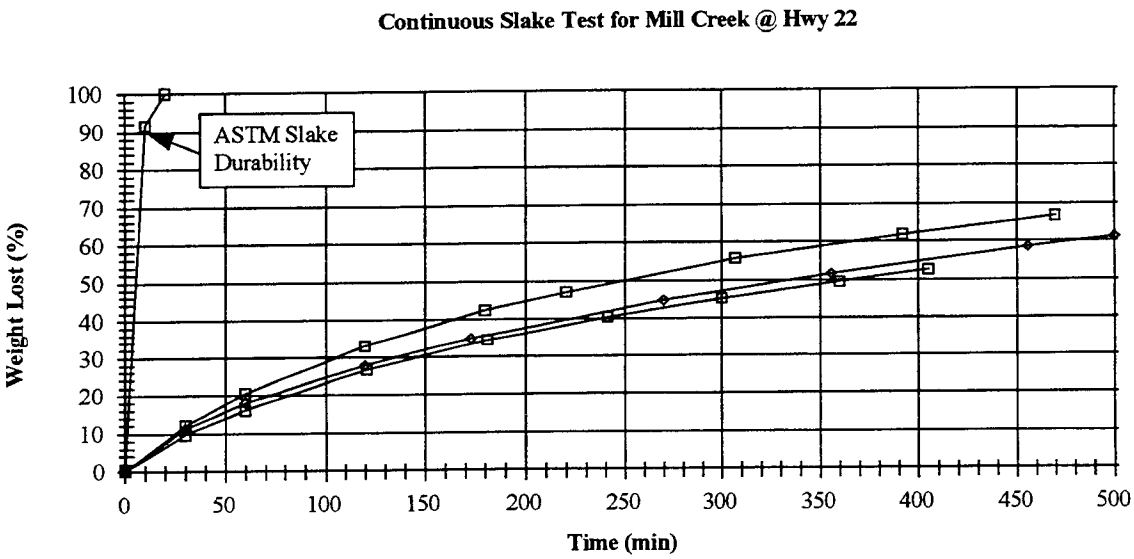
NESTUCCA RIVER @ POWDER CREEK

LA Abrasion Test Not Performed for Nestucca River @ Powder Creek



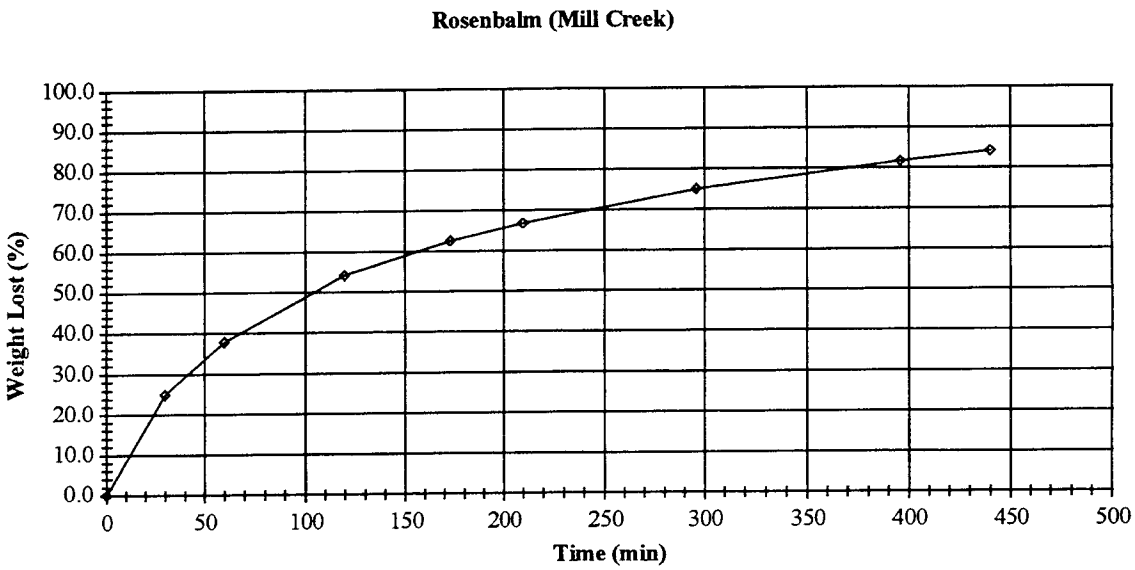
MILL CREEK @ HIGHWAY 22



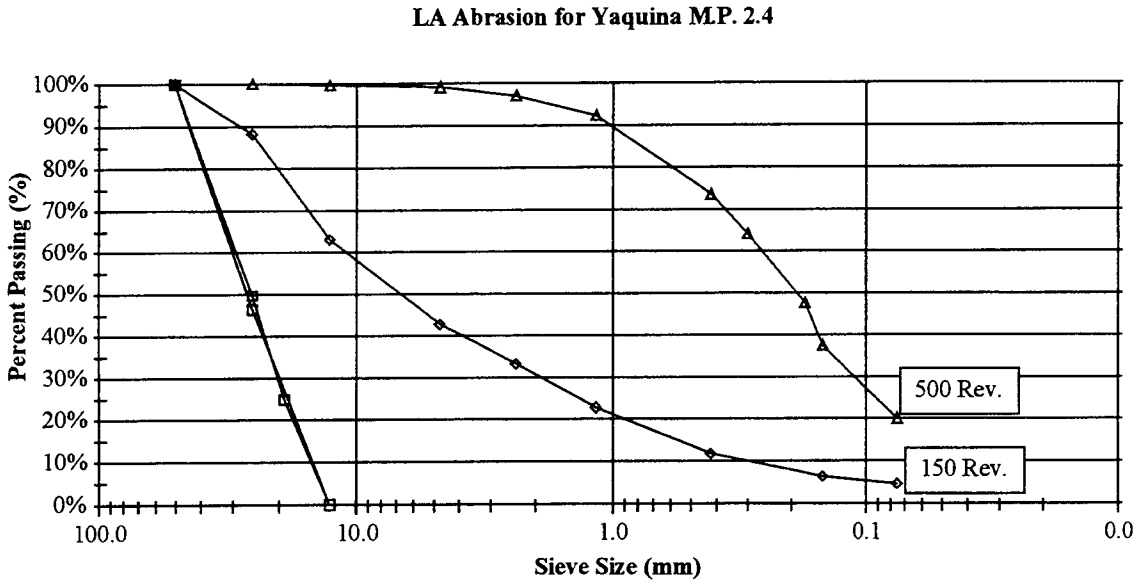


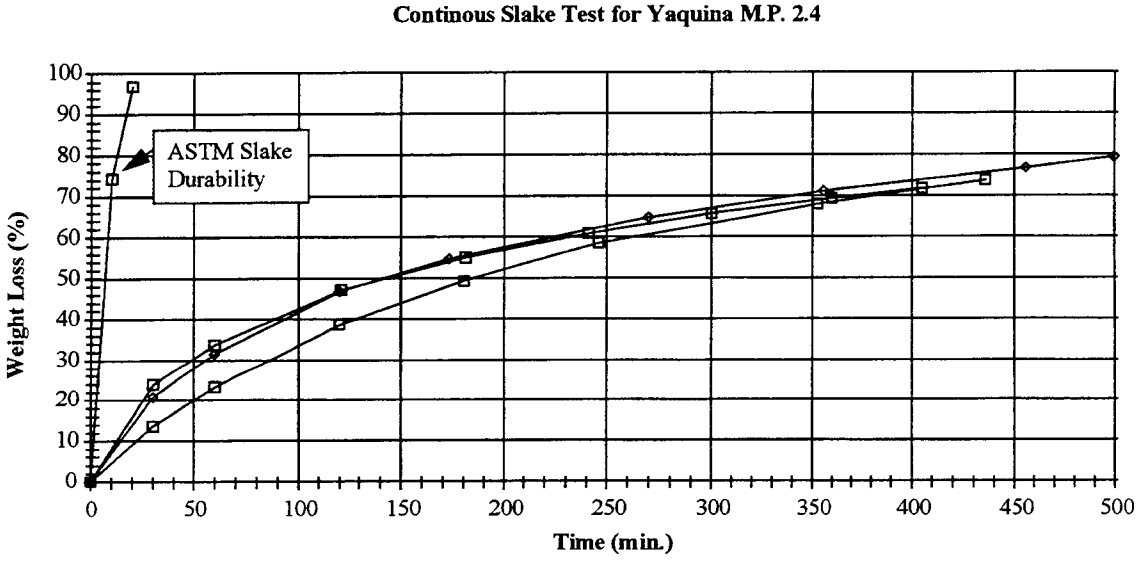
MILL CREEK @ ROSENBALM ROAD

LA Abrasion Test Not Performed for Mill Creek @ Rosenbalm Road



YAQUINA RIVER @ M.P. 2.4





YAQUINA @ M.P. 4.9