DESIGN AND CONSTRUCTION OF ROCK CAP ROADWAYS – A CASE STUDY IN NORTHEAST WASHINGTON

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ABSTRACT
In recent years the Washington State Department of Transportation (WSDOT) has deviated from its normal policy of correcting frost heaving and thawing problems on state highways. WSDOT’s traditional approach for frost design rehabilitation has been to place crushed stone base at least half the depth of the frost penetration. This approach has served WSDOT well on the majority of its highway system, however, other measures were sought to mitigate extensive frost related problems in northeast Washington. To isolate the flow of water from the pavement structure, a capillary break using a free draining aggregate or “rock cap” layer was used on projects during the last five years. The rock cap material used was a 75 mm (3 in.) maximum sized material with 0 to 15 percent passing a 12.5 mm (1/2 in.) sieve. The open graded nature of the rock cap provides a positive drainage blanket so that excess water can be eliminated from the roadway structure thus eliminating frost heaving so that excess water can be eliminated from the roadway structure thus eliminating frost heaving and thaw weakening problems. However, constructing with a large stone material presents special construction considerations particularly pertaining to the stability of the material. WSDOT’s construction experience is detailed in this study as well as a summary of rock cap performance in Washington State.
INTRODUCTION
Frost action in pavements refers to two separate but related processes. The first process is frost heaving resulting mainly from the accumulation of moisture (ice lenses) in the frost susceptible soil during the freezing period. The second is thaw weakening of soil when thawing temperatures occur. These damaging processes are satisfied when three conditions necessary for frost heaving or thaw weakening are met: subfreezing temperatures, the presence of water, and a frost susceptible soil. If any of these three conditions are removed during the design process for new roadways, frost effects will be minimized or even eliminated.

The Washington State Department of Transportation’s (WSDOT) traditional approach to mitigate frost heaving or thaw weakening has been to have that portion of the pavement structure that is composed of non-frost susceptible material equal to at least half the design depth of the frost penetration. This approach has served WSDOT well on the majority of its highway system, however, other measures were sought to mitigate extensive frost related problems in northeast Washington. In recent years, WSDOT has transitioned to correcting frost heaving and thawing problems on state highways by isolating the flow of water from the pavement structure by using a capillary break of free draining aggregate or “rock cap” layer.

The rock cap material used as a capillary break is an open graded aggregate with 100 percent passing the 75 mm (3 in.) sieve and 0 to 15 percent passing the 12.5 mm (1/2 in.) sieve. This free draining material placed immediately on top of embankment or existing pavement intercepts the flow of water before entering the newly constructed upper pavement layers from lower water sources.

Many roadways in northeast Washington were built in the 1930’s and 40’s where a minimal thickness 150 to 200 mm (6 to 9 in.) of crushed stone base was placed on frost susceptible soils prior to a bituminous surface treatment (BST). Over subsequent BST rehabilitation cycles, additional thickness of BST was placed and resulted in a 50 to 75 mm (2 to 3 in.) bituminous layer that is adequate for low volume roadways during dry seasons. However, during the spring thaw, which typically occurs in the last two weeks of February or early March in northeast Washington, this marginal structure does not adequately support truck traffic.

During the spring the roadway can thaw from the top downward, the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take. The net affect of the thaw is that the load carrying capacity of the roadway is drastically reduced and road restrictions or closures must be enforced.

In order to reduce the amount of thawing moisture in the base layers, the concept of a capillary break to remove this moisture was investigated. The open graded nature of the rock cap provides a positive drainage blanket so that excess water can be eliminated from the roadway structure. Additionally, during the freezing process, the migration of water into the untreated base course beneath the asphalt surface is eliminated. The result is a roadway that is not subjected to road restrictions or road closures and is defined as an “all season roadway” by WSDOT.

In the early 1990’s, WSDOT recognized the need to mitigate spring thaw problems on State Route 20 located in northeast Washington between Colville and Tiger Junction (see Figure 1). This 58 km (36 mi) roadway serves as the primary route for freight, particularly the timber industry, which is the primary economic generator for the area. Of the 58 km (36 mi), 43 km (27
mi) were in poor condition due to spring thaw damage. Yearly load restrictions were imposed causing additional economic hardships for an area that is already economically distressed.

WSDOT, to alleviate this problem, constructed a rock cap roadway on the initial 10 km (6 mi) section in 1997. An additional 18 km (11 mi) was constructed in 1999, 2 km (1.3 mi) in 2000, and the final 13 km (8 mi) of roadway will be constructed in 2002.

The following case study examines rock cap construction on SR 20 in three ways. First, previous research on the use of capillary breaks for frost design will be explored. Capillary breaks of various types have been used both nationally and internationally with good success. Second, the design considerations for rock cap construction will be discussed. WSDOT entered into the design of rock cap with numerous unknowns and valuable lessons have been learned. Third, the technique and experience used by WSDOT for rock cap construction will be examined. This study concludes with a summary of rock cap performance based on falling weight deflectometer (FWD) deflection data.

LITERATURE REVIEW
Several papers were presented at the 42\textsuperscript{nd} Annual Meeting of the Highway Research Board (January 1963) that summarized a substantial amount of information on pavement design for frost conditions. Practices of state highway agencies as well as Canadian provinces, Sweden, and Finland are contained in Highway Research Board Record No. 33. One of the areas reported by a few agencies is the use of capillary breaks to reduce the upward movement of water and/or enhance subsurface drainage.

Studies conducted in Sweden reported that the use of sand layers to cut off capillary flow has been used on a “wide scale.” One example shown by Rengmark (1) contained a sand layer 200 mm (8 in.) thick on top of a silty subgrade. It was noted that the sand layer should be placed above the bottoms of adjacent ditches to increase drainage. An example in Finland by Taivaninen (2) noted that use was made of sand (or gravel) layers to stop the capillary flow of water and were placed on the top of the subgrade. Such layers were generally 150 to 200 mm (6 to 8 in.) thick but could be up to 400 mm (16 in.). Current standards used in Estonia call for 300 mm (12 in.) of free draining sand or a capillary break (based on discussions and observations with Estonian Road Administration personnel).

In Canada, Armstrong (3) reported that British Columbia has used a 300 mm (12 in.) thick layer of clean sand placed on subgrades as part of corrective measures to address frost-heave problems. It was unclear as to whether the sand layer was to enhance subsurface drainage in general, and/or as a capillary break. In Quebec, it was noted that a “cutoff blanket” of sand or gravel was used in areas with high water tables.

In 1973, Johnson (4) reported on a survey of North American DOT roadway design practices in seasonal frost areas. He noted that the State of Maryland used a 305 mm (12 in.) “granular cap” over frost susceptible subgrade soils. A CBR of 7 was assigned to such layers.

This general concept has been applied by the Idaho Department of Transportation (Idaho DOT) as reported by Mathis (5). Idaho DOT has used rock cap construction since the early 1980’s to control frost action and water problems occurring primarily in northern Idaho. Idaho DOT’s initial use of rock cap was an experiment on US-95 and Interstate 90 to provide a positive drainage blanket beneath the bound pavement. This method worked so well that rock cap construction has become standard practice in Idaho. The Idaho DOT estimates that 805 to 970 km (500 to 600 mi) of rock cap roadway have been built to date with the majority in the Coeur D’Alene and Lewiston Districts. Rock cap thicknesses have ranged from 175 to 815 mm (0.60 to 2.67 ft) thick and have been used on both interstate and low volume roadways.
Rock cap roadways in Idaho are designed using two methods. One method places the rock cap over the existing pavement or subgrade, followed by a crushed stone base layer on which hot mix asphalt is placed. The second method replaces the crushed stone base with a “sacrificial” layer of hot mix asphalt placed directly over the rock cap. A leveling and wearing course of hot mix asphalt is then placed over the “sacrificial layer.” Both techniques are illustrated in Figure 2. Each construction practice has special construction considerations as noted by Mathis (5) and Smith (6).

Idaho DOT reports backcalculated material properties (layer moduli) for the rock cap layer range from 170 to 415 MPa (25,000 to 60,000 psi). The Idaho DOT has not observed significant seasonal change in these moduli.

Currently, for structural design purposes, 30 mm (1.2 in.) of rock cap material is substituted for 25 mm (1 in.) of untreated base as reported by Smith. This equivalency is based on the assumption that rock cap has better drainage characteristics.

DESIGN CONSIDERATIONS
The following section discusses the design considerations used for the construction of the rock cap roadway on SR 20. A description of the project, use of falling weight deflectometer data, field sampling of the existing subgrade and crushed surfacing materials followed by the structural design process used to design the rock cap structure are discussed.

Project Description
The topography or SR 20 consists of rolling terrain with small farms of pasture or hay and areas of draws and valleys vegetated with ponderosa pine. The roadway traverses cut slopes and embankments with streams adjacent to the roadway in many locations. Precipitation is normally 450 mm (18 in.) per year with the bulk of the moisture falling as snow from October to March. The average freezing index is about 440°C-days (800°F-days) with design freezing indices of about 600 to 720°C day (1,100 to 1,300°F-day).

The existing roadway consisted of a two-lane roadway with 3.3 m (11 ft) lanes and 0.6 m (2 ft) shoulders. The average daily traffic (ADT) varies from 3,500 through Colville and drops to 750 vehicles for majority of the project. Trucks along this low volume route are nearly 25 percent of the total traffic (or about 200 trucks per day).

The pavement distresses throughout the entire section included substantial amounts of fatigue cracking, rutting, shoving and flushing. Frost heaves occurred throughout the corridor with a typical heave length ranging from 15 to 120 m (50 to 400 ft). As thaw weakening occurred, significant water was noted in the roadside ditches, often filling the entire ditch. The roadway structure became saturated and remained wet until summer. Extensive asphalt maintenance patching had been placed to “band aide” the road together. These patches generally performed three to four years before fatigue cracking occurred requiring additional pavement repair or more patching. Figure 3 shows the typical extent of distress along SR 20.

Falling Weight Deflectometer Data
As a way to compare the existing roadway to the roadway overlaid with rock cap, WSDOT conducted deflection testing prior to and following rock cap construction on SR 20. The deflection data prior to the reconstruction of the roadway was taken over three periods that included April 1993, September 1994 and June 1995.
Over the years, WSDOT has found that the use of the Area value provides a “good picture” of the relative condition of the pavement structure found throughout a project. The Area value represents the normalized area of a slice taken through any deflection basin between the center of the test load and 914 mm (36 in.). By normalized, it is meant that the area of the slice is divided by the deflection measured at the center of the test load, $D_0$, as described in the WSDOT Pavement Guide (7).

WSDOT experience has shown that weak BST structures are represented by Areas values of 300 to 380 mm (12 to 15 in.). Roadways with Area values of 380 to 430 mm (15 to 17 in.) are typical of thin BST pavements. Thin asphalt concrete pavements (around 100 mm (4 in.)) are typically represented by Area values in the 410 to 530 mm (16 to 21 in.) range. Figure 4 shows the range of Area values typically observed in WSDOT pavements (7).

The Area values for the April 1993 FWD testing ranged from 353 to 388 mm (13.9 to 15.3 in.) and reflected project limits where the base and subgrade were weak from excess moisture. The deflections taken during September 1994 and June 1995 reflected the base and subgrade that had drained and recovered to a somewhat stiffer condition. The Area values for the September and June FWD testing ranged from 380 to 453 mm (15.0 to 17.8 in.). Ideally, both spring thaw and summer deflection testing for the entire project would have been preferred, however, this data was not available. Based on the typical Area values observed in Figure 4, the relative condition of the existing roadway was generally weak BST to a thin structure of BST pavement.

FWD testing showed that the subgrade resilient modulus values varied from 60 to 210 MPa (9,000 to 31,000 psi). The higher values are in areas of near surface bedrock.

Following the reconstruction of the roadway with rock cap, yearly deflections were measured from 1999 to 2002. Deflection measurements were taken between mid April and mid June to get a range of deflections and the associated Area values to compare with deflection data taken prior to the reconstruction with rock cap. Specific FWD deflection testing locations were chosen to correspond with the milepost markers. In the eastbound direction, six test locations were taken over a 0.40 km (0.25 mi) section beginning at each milepost marker. In order to test the entire roadway width, each group of tests alternated in the outer and inner wheel paths. The FWD deflections data was normalized to a 40 kN (9000 lb) load and were adjusted for pavement thickness and to a temperature of 25°C (77°F). The results of the post construction FWD testing will be discussed under the Performance section.

Field Sampling
A number of cores were collected in June 1994 to determine the BST thickness and to obtain gradation samples of the existing base and subgrade materials. The BST depths varied from 40 to 145 mm (1.6 and 5.6 in.) thick with the majority of the project being 75 mm (3 in.) thick; the thicker areas generally corresponded to extensively patched areas.

The evaluation of the base material indicated the presence of a crushed stone base and a borrow subbase layer. The total thickness was approximately 245 mm (9.6 in.). The crushed stone base was silty gravel approximately 60 mm (2.4 in.) thick. The borrow subbase was a silty sandy gravel and silty gravel approximately 185 mm (7.2 in.) thick. Sieve analysis performed on four samples showed the percent passing the 0.075 mm (No. 200) sieve for both materials varied from 11.7 to 17.9 percent. The natural moisture content as measured in June 1994 varied from 3.2 to 4.3 percent.
The subgrade is predominately silty sand with gravel and cobbles derived from decomposed granite. The subgrade was extremely frost susceptible as evidenced by three samples with 13 to 43 percent passing the 0.075 mm (No. 200) sieve. The moisture content measured in June 1994 varied from 3.3 to 12.0 percent.

Additional testing showed that numerous locations throughout SR 20 had a high near surface water table. In some locations this was verified by augering 150 mm (6 in.) holes and observing the holes filling to the surface in a matter of minutes.

Structural Design
Based on the success of rock cap construction in Idaho, WSDOT assimilated much of Idaho’s practice during the early 1990’s into the design of SR 20. Use of rock cap would provide a capillary break, drainage, and the insulation needed to help protect the subgrade from freezing.

The pavement structure needed to accommodate a subgrade with a typical 80 MPa (12,000 psi) resilient modulus and 500,000 ESALs over the 20-year design period required a total depth of 300 mm (12 in.). With frost penetration depths of 1,500 mm (60 in.) or more on SR 20, WSDOT would require a 760 mm (30 in.) structure to satisfy the depth of 50 percent of the frost penetration. The economics of constructing these projects came into question since funds were limited. The estimated cost of the rock cap was escalating beyond the available construction dollars.

The proposed roadway for the SR 20 corridor was a 670 mm (26 in.) section that included 100 mm (4 in.) of hot mix asphalt, 100 mm (4 in.) of crushed stone base and 460 mm (18 in.) of rock cap. The pavement design required that these layers be placed on the existing BST surfacing primarily due to unknowns with the existing subgrade conditions. Previous field investigations showed that the existing subgrade was marginal and if the existing surface was completely removed, it would be difficult to place the rock cap material without extensive sub excavation. This decision was confirmed during roadway preparation, as unsuitable material excavation was required at several locations. Within these locations, excavations exposed log rafts and organic materials placed to build a platform for the original SR 20 construction likely dating back 100 years.

WSDOT’s decision to construct a 670 mm (26 in.) roadway section was partially based on economics and experience. While the 670 mm (26 in.) section was less than the desired 760 mm (30 in.) structure required for traditional frost design, WSDOT considered the benefits of the rock cap section and the almost total elimination of water from the pavement section to justify the lesser thickness.

As WSDOT proceeded with the preparation of contract plans, a reduction in the rock cap depth was requested to aid in fitting a profile to the roadway as the existing profile had high and low spots due to settlement, maintenance patching and repairs. While a 460 mm (18 in) layer of rock cap was desirable, additional pavement coring was performed to define the existing BST or asphalt depths along the project. Based on this coring, the rock cap thickness was reduced from 460 mm (18 in.) to 400 mm (16 in.) to give credit for 60 mm (2.4 in.) of the existing BST surfacing found throughout much of the project limits.

CONSTRUCTION CONSIDERATIONS
The Idaho DOT had previously described rock cap material, when under traffic, as acting much like an arrestor bed on a truck escape route (6). With no previous experience placing and maintaining the stability of a large stone base such as rock cap and the construction difficulties
noted by Idaho DOT, WSDOT envisioned problems with vehicles stranded in the rock cap material. However, as with the experience in Idaho, WSDOT was willing to proceed with placing a rock cap roadway with the belief that contractors could build such a roadway in Washington.

Despite these early fears, the 1998 project exceeded all expectations and the stability of the roadway for automobiles and construction vehicles was excellent. WSDOT found that the top size of 75 mm (3 in.) for the rock cap material was normally very stable and only experienced two problems: trucks hauling rock cap material got stuck while dumping and car tires were occasionally punctured by the sharp rocks. The first problem was solved by providing a grader to pull out the occasional truck that became stuck. The second problem was reduced by compacting the material with at least three passes prior to allowing vehicles on the rock cap surface. Automobile and construction traffic were able to briefly traverse the roadway with little difficulty until a 60 mm (2.4 in.) thick layer of crushed stone aggregate was placed on the roadway. This layer of crushed stone base tightened the rock cap surface by filling voids thus providing an improved albeit temporary driving surface.

**Rock Cap Material**

The rock cap specification used by WSDOT is a slight variation of that used by Idaho DOT. Table 1 shows the gradations used on SR 20 compared to the gradations used by Idaho DOT. Idaho’s rock cap specification has evolved from a 100 mm (4 in.) maximum size material down to a 63 to 75 mm (2.5 to 3 in.) maximum size, which is being used today as reported by Smith (6). WSDOT chose a gradation without intermediate sieve sizes to allow the crushing contractor flexibility with the potential for less cost because of less sieve sizes to crush.

The rock cap material came from a quarry located on US Forest Service property just north of SR 20. This material source was derived from a metamorphic rock and was extremely dense. The Los Angeles Abrasion Resistance was 16 with the bulk specific gravity of 3.03. Since the material came from a quarry, the 100 percent fracture provided excellent aggregate interlock allowing an extremely stable surface.

Initially, the rock cap material specified by WSDOT was a 75 mm (3 in.) top size aggregate and crushing was done to obtain the maximum size. However, as work progressed the contractor crushed a 63 mm (2.5 in.) top size material, as the smaller size seemed to grade, compact and provide a more stable roadway to carry traffic.

**Rock Cap Placement**

The rock cap was placed directly on the existing roadway using belly dump trucks, except where widening occurred. A geotextile fabric used as a separator was placed over the subgrade in the widened areas to prevent intrusion of the fine materials into the rock cap.

The general sequence used on the SR 20 projects was to place the rock cap from several trucks and grade the material directly behind the placement. Immediately after grading, additional trucks would dump material followed by grading. This operation was continued until a 90 to 150 m (300 to 500 ft) area had a lift of 150 to 200 mm (6 to 8 in.). Next, the rock cap was compacted and the placement operation would drop back and add another lift to the previously compacted area.

On the most recent 2002 rock cap project, the contractor placed nearly 5,500 tonnes (6,000 ton) per day over 10-hour shifts. This particular contractor had 24 trucks operating with 32 tonne (35 ton) and 23 tonne (25-ton) capacities. The typical one-way haul was 20 minutes
with the contractor daily placing 460 m (1,500 ft) of rock cap 400 to 460 mm (16 to 18 in.) thick. On previous projects, the contractor placed 3,600 to 4,100 tonnes (4,000 to 4,500 ton) per day using 18 tonne (20 ton) trucks.

The rock cap was compacted by using a 9,000 kilograms (10-ton) vibratory roller having a dynamic force of at least 13,600 kilograms (30,000 pounds) with at least 1,000 vibrations per minute. A roller pattern using four full coverages for each 150 mm (6 in.) lift was specified. Additionally, the contract required that the contractor compact the material further by routing empty and loaded hauling equipment evenly over the entire width of the roadway.

The use of water during compaction was a necessity to coat the individual rocks thus reducing the inter-particle friction when compaction energy was applied. WSDOT found that the more water the contractor used, the tighter and easier the material was to compact.

WSDOT was typically able to maintain the rock cap grade within 0.00 mm high to 40 mm low (0.00 in. high to 1.2 in. low) by providing grade stakes at 15 m (50 ft) intervals. WSDOT specifications required that all rock cap placed be constructed full width and depth on the same shift.

Figure 5 shows various construction photos of rock cap placement, grading, and compaction as discussed above and the placement of the crushed stone base as discussed in the following section.

**Crushed Stone Base Placement**

To maintain the long-term stability of the rock cap layer, the rock cap must be confined. WSDOT provided this confinement by placing a 100 mm (4 in.) crushed stone base with a 45 mm (1 in.) maximum size aggregate directly over the rock cap. However, to “lock up” the rock cap surface, WSDOT specified that a minimum 60 mm (2.4 in.) of crushed stone base must be placed on the roadway prior to the opening of traffic. WSDOT found that with construction speeds of 48 km per hour (30 mi per hour) the stability of the rock cap was maintained until the full thickness of crushed stone base could be placed. Typically, the contractors placed about 3 km (2 mi) of rock cap and the minimal depth of 60 mm (2.4 in.) crushed stone base before dropping back and placing remaining thickness of crushed base. The 100 mm (4 in.) lift of crushed stone base allowed the contractor to fine grade the material and provide a finished grade prior to placing the temporary BST surfacing and final hot mix asphalt wearing surface.

Since the rock cap material is open graded and crushed stone base is dense graded, there is potential for the fine aggregate in the crushed stone base to filter through the voids of the rock cap. If this happens, the crushed stone base may eventually deform and cause rutting in the asphalt surface. Idaho DOT reports that surface deformation in the asphalt surfacing has been observed on some of the earliest rock cap projects but this is more likely due to the use of a 100 mm (4 in.) maximum size rock cap material and not the 63 to 75 mm (2.5 to 3 in.) maximum size material used today. Idaho DOT noted that when using crushed stone base the infiltration loss could be 25 to 30 percent (6). WSDOT observed an infiltration loss of approximately 15 percent.

Ideally, designing the crushed stone base layer according to filter criteria would eliminate the deformation problem, however, knowing what rock cap gradations a contractor will crush is difficult. For this reason, Idaho DOT routinely uses a sacrificial lift of asphalt concrete placed directly on top of the rock cap layer prior to placement of the hot mix asphalt surface.

In order to maintain the non-frost susceptibility of the crushed stone base, WSDOT specified that the crushed stone base be a very clean material with 6 percent or less passing the
0.075 mm (No. 200) sieve. However, once crushing began, the WSDOT project office expressed concerns of maintaining the raveling and wash boarding potential of the crushed stone base on the roadway under traffic due to such a low amount of fines in the material. Additionally the crushing contractor was running out of stockpile room due to an excess of 0.075 mm (No. 200) material being generated. A change order was issued to allow the contractor 3 to 9 percent passing the 0.075 mm (No. 200) sieve.

The WSDOT Materials Office had concerns with this revision since this project was being constructed to remove the potential for freeze/thaw and moisture damage. The concern was that moisture could infiltrate the asphalt surface from cracks or pass through the rock cap layer from ditches and contribute moisture to the crushed stone base. A review of the project records showed that the crushed stone base had 6.3 percent passing the 0.075 mm (No. 200) sieve. The standard deviation was 1.3 with a range of 3 to 11 percent. Overall, a relatively clean crushed stone material was obtained, however, WSDOT recommends their standard crushed stone base with 7 percent or less passing the 0.075 mm (No. 200) rather than modifying the gradation and potentially allowing a frost susceptible base course material (which was the case for sections on SR 20).

**Temporary Surfacing – Bituminous Surface Treatment Application**

Following the construction of a typical 3 km (2 mi) section of the rock cap and crushed stone base layers, WSDOT applied a BST surface. Placing this BST surface provided a roadway surface very acceptable to the public prior until the final ACP wearing surface was placed. The greatest benefit from using the BST surface was that fine grading of the crushed stone base for the roadway cross-section and profile was done only once. Traffic used the BST surface for one to three months prior to placing the final asphalt wearing surfaces and maintenance concerns were eliminated.

**PERFORMANCE**

Figure 6 displays the Area values for both the pre-construction (1993, 1994, and 1995) and the post-construction Area values from 1999 to 2002. The 2001 data was not plotted due to some problems encountered during the testing. The preconstruction Area value had an average value of 405 (15.9 in.). In comparison, the Area value for the rock cap roadway had an average Area value of 447 mm (17.6 in.) resulting in an average increase of 45 mm (1.7 in.).

Based on this assessment, it is observed that the structural condition of the rock cap overlaid roadway improved from a typical BST flexible pavement to a thin asphalt pavement (Based on Figure 4). Overall, Figure 6 shows that average Area values at each milepost, following reconstruction, increased from 14 to 91 mm (0.5 to 3.6 in.) from the measurements of the pre-existing roadway.

A more realistic view of the impact of the rock cap is to compare deflection measurements (Area values) taken during June 1995 (existing roadway) and April 2002 (rock cap roadway). This is a particularly interesting comparison since one would expect the existing roadway to have higher stiffness during the summer compared to a less stiff condition in the spring when water saturates the structure. An increase in the Area values of 11 to 74 mm (0.4 to 2.9 in.) was observed in the rock cap measurements during a wet period when compared to the dry period with the existing roadway. The 74 mm (2.9 in.) increase occurred at a section of roadway where there was a particularly wet area prior to construction.
Another observation is made when comparing deflection testing occurred during April 1993 and 2002 when it was expected that the base or subgrade materials were in a weakened or saturated condition. When tested during the same months, the post construction Area values were 13 to 123 mm (0.5 to 4.8 in.) greater with the rock cap structure.

Since the construction of the rock cap, there have been no road restrictions due to thawing in the pavement structure. The rock cap sections are performing well and no maintenance has been required due to distress in the roadway (The sections range in age from two to four years).

SUMMARY AND CONCLUSIONS
The focus of this paper was to examine the use of rock cap as a capillary break to reduce or eliminate frost heaving in pavements. Although WSDOT has constructed only 64 km (40 mi) of rock cap roadways statewide, experience has shown that the rock cap sections provides a structural section that resists thawing and frost heaving effects thus providing an all season roadway. Despite fears that construction vehicles and traffic could not travel on the open graded material, WSDOT found that a 75 mm (3 in.) rock cap material was stable and only had a few surface stability problems. Based on the experience gained, WSDOT will continue to use rock cap material as a capillary break on roadways sensitive to frost heaving and thaw weakening. WSDOT will continue to monitor the rock cap roadways that have been placed.

Specific conclusions about the construction and performance of rock cap roadways follow:

- The use of a 63 mm (2.5 in.) maximum size rock cap material facilitated construction by providing a more stable roadway for construction and vehicle traffic than the 75 mm (3 in.) material.
- Placement of a 60 mm (2.4 in.) thickness of crushed stone base directly on the final grade of rock cap “locks up” the rock cap surface and allows vehicle traffic until the final crushed stone base and asphalt surfacing can be placed.
- Placement of a BST following construction of the crushed stone base on the rock cap layer eliminated the need for additional grading while providing a drivable wearing surface prior to the asphalt overlay.
- Spring load restriction and extensive maintenance on the existing frost susceptible roadway were eliminated after construction of the rock cap.
REFERENCES


List of Tables and Figures

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FIGURE 2  Comparison of the two rock cap structures used by Idaho DOT. Method (a) uses asphalt surfaces placed on crushed stone base over rock cap. Method (b) uses a sacrificial lift of asphalt placed directly on the rock cap followed by asphalt surfacing.

FIGURE 3  Examples of the extent of distress caused by freezing and thawing on SR 20.

FIGURE 4  Range of Area values typically observed in WSDOT pavements.

FIGURE 5  Placement of rock cap and crushed stone base layers: (a) Existing roadway; (b) Placement of rock cap from belly dump trucks; (c) Grading rock cap aggregate; (d) Watering and compacting rock cap; (e) Placement of crushed stone base on top of rock cap aggregate; (f) Crushed stone base placed on the rock cap aggregate.

FIGURE 6  Comparison of FWD deflection measurements for the existing and reconstructed rock cap roadway.

TABLE 1  Summary WSDOT and Idaho DOT Rock Cap Specifications.
FIGURE 1 SR 20 rock cap project location.
(a) Rock Cap Overlaid with Crushed Stone Base  (b) Rock Cap Overlaid with Asphalt Surfacing

FIGURE 2 Comparison of the two rock cap structures used by Idaho DOT. Method (a) uses asphalt surfaces placed on crushed stone base over rock cap. Method (b) uses a sacrificial lift of asphalt placed directly on the rock cap followed by asphalt surfacing.
FIGURE 3  Examples of the extent of distress caused by freezing and thawing on SR 20.
FIGURE 4  Range of Area values typically observed in WSDOT pavements.
FIGURE 5 Placement of rock cap and crushed stone base layers: (a) Existing roadway; (b) Placement of rock cap from belly dump trucks; (c) Grading rock cap aggregate; (d) Watering and compacting rock cap; (e) Placement of crushed stone base on top of rock cap aggregate; (f) Crushed stone base placed on the rock cap aggregate.
FIGURE 6  Comparison of FWD deflection measurements for the pre constructed (original BST) and reconstructed rock cap roadways.
### TABLE 1  Summary of WSDOT and Idaho DOT Rock Cap Specifications.

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<tbody>
<tr>
<td>Percent Passing</td>
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<tr>
<td>100 mm (4 in.)</td>
<td>100</td>
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<tr>
<td>75 mm (3 in.)</td>
<td>100</td>
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<tr>
<td>63 mm (2 ½ in.)</td>
<td>65-80</td>
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<tr>
<td>37.5 mm (1 ½ in.)</td>
<td>65-80</td>
<td>55-85</td>
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<tr>
<td>19 mm (3/4 in.)</td>
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<td>15-30</td>
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<tr>
<td>12.5 mm (½ in.)</td>
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<td>4.75 mm (No. 4)</td>
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</table>

*Idaho DOT Supplemental Specification Section 307.*