by 16 mm (0.6 in.). Thus, the high torque at the tire pavement interface caused a downward flow of the surface mix. The lack of stability of the bituminous mixture was determined to be caused primarily by a soft grade of asphaltic cement. An overlay with a stiffer grade of asphaltic cement was placed.

SUMMARY

Pavements can be designed for heavily loaded trucks, but the rate of accumulating fatigue is greatly accelerated. The accumulation of fatigue for heavy trucks is highly disproportionate to the amount of payload transported. For the same fatigue and assumed proportions of trucks, the number of trucks loaded to the legal maximum axle loads is approximately 10 times the number of heavily loaded trucks. For the same fatigue, legally loaded trucks can transport approximately 8.2 times more payload than can heavily loaded trucks.

Pavements designed for normally loaded trucks may deteriorate rapidly and severely when subjected to heavily loaded trucks. Observed deterioration varies from accelerated rutting, both in depth and time, to severe breakup of the paved surface.

ACKNOWLEDGMENT

This study was conducted at the University of Kentucky Transportation Research Program and in part was sponsored cooperatively by the Kentucky Transportation Cabinet and the Federal Highway Administration.

REFERENCES

1. J. Michelow. Analysis of Stresses and Displacements in an N-Layered Elastic System under a Load Uniformly Distributed on a Circular Area. Chevron Research Corporation, Richmond, Calif., 1963.
2. I.S. Sokolnikoff. Mathematical Theory of Elasticity, Second Edition. McGraw-Hill, New York, 1956.
3. R.C. Deen, H.F. Southgate, and J.G. Mayes. The Effect of Truck Design on Pavement Performance. Proc., Association of Asphalt Paving Technologists, Minneapolis, Minn., Feb. 1980.
4. H. Warren and W.L. Dieckmann. Numerical Computation of Stresses and Strains in a Multiple-Layered Asphalt Pavement System. Chevron Research Corporation, Richmond, Calif., 1963.
5. H.F. Southgate, R.C. Deen, J.H. Havens, and W.B. Drake. Kentucky Research: A Flexible Pavement Design and Management System. Proc., 4th International Conference, Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, 1977.
6. J.H. Havens, R.C. Deen, and H.F. Southgate. Design Guide for Bituminous Concrete Pavement Structures. Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Rept. KTRP-81-17, Lexington, Aug. 1981.
7. H.F. Southgate, R.C. Deen, and J.H. Havens. Development of a Thickness Design System for Bituminous Concrete Pavements. Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Rept. KTRP-81-20, Lexington, Nov. 1981.
8. Traffic Factors Used in Flexible Pavement Design. TRB, Research Circular No. 240, April 1982.
9. Interim Guide for Design of Pavement Structures. AASHTO, Washington, D.C., 1972.

Pavement Analysis for Heavy Hauls in Washington State

RONALD L. TERREL AND JOE P. MAHONEY

An evaluation of the haul routes associated with moving heavy nuclear power plant components over existing state and county roads is described. These routes were associated with the planned construction of the power plants at Satsop (southwestern Washington state) and Sedro Wooley (northwestern Washington state). The procedures for evaluating the proposed moves are provided and include descriptions of the field and laboratory tests and analytical techniques. Ultimately, the haul routes analyzed were not used.

In recent years construction of nuclear power plants instigated analyses of the feasibility of using existing highways for hauling very large and heavy machinery. These hauls require special tractors and trailers to accommodate the machinery as well as to spread the load to minimize damage to pavements and bridges.

A special permit is required by those who perform the heavy hauling. For modestly oversize or overweight vehicles, obtaining a permit is somewhat routine as long as local requirements of wheel load and spacing are met, and the fee is usually nominal. Very heavy loads, however, require considerably more analysis to assess potential damage to the existing facilities, and the fee is commensurate with the expected cost to repair the damage.

Plans for constructing two nuclear power plants in Washington State instigated pavement analysis for the purpose of obtaining permits. Each plant had a different owner, and each owner was required to back up the application with an analysis of the significant damage that would be incurred, if any. For one project, Skagit, the supplier of the reactor equipment was required by the plant owner to deliver the equipment to the job site, so the supplier arranged for a consultant to evaluate the proposed route. The consultant's report was, in turn, presented to the Washington State Department of Transportation (WSDOT). For the other project, Satsop, the plant owner requested that the pavement evaluation be made by WSDOT, but be paid for by the plant.

The proposed routes were evaluated to determine expected immediate damage (during hauling) as well as reduced service life. In addition the alternatives of improving the roadway before hauling or making repairs after hauling were considered.

Each project is treated as a separate case history. Figure 1 shows the general location of the projects. In each case the projected route covered local (county) and state highways. The primary analysis was for the main highway. Unsurfaced local roads or new access roads were considered separately and were treated as expected reconstructions to meet load and geometry requirements.

Figure 1. Site map for nuclear power plant component haul locations.

Ultimately, the proposed haul routes were not used. An alternate route was constructed for the Satsop plant because of right-of-way acquisition problems and the other project was cancelled.

CASE 1, SATSOP NUCLEAR POWER PLANT

The Satsop plant is located near Elma, Washington, west of Olympia on US-12. The route to be investigated was about 12 miles along US-12. The data and other information used in evaluating US-12 for the planned heavy hauls were from two primary sources.

The first source was field studies conducted by WSDOT and included the following:

1. Benkelman beam deflection survey;
2. Soil borings, samples of granular materials, and cores of asphalt concrete pavement (ACP) and cement treated base (CTB) pavement; and
3. Plate bearing tests.

Test samples including soil borings, granular base and fill samples, and ACP and CTB cores were obtained during January 1979. The Benkelman beam deflection survey data were used in selecting test sampling locations. Plate bearing tests performed during April 1979 completed the WSDOT field studies. Additionally, during May 1979 a falling weight deflectometer (FWD) was used at selected stations to obtain deflection information and estimate modulus relationships for the pavement layers. Possible pavement distress caused by a slope stability failure along the proposed haul route was also analyzed by WSDOT personnel.

The second source of data was developed at the University of Washington (UW) Department of Civil Engineering pavement materials laboratory in Seattle. The overall goal of the laboratory program was to develop strength parameters for the primary structural materials contained in the US-12 cross sections and specifically to develop the required elastic parameters to enable modeling of the pavement structure as a layered-elastic system.

Field Testing

First, a Benkelman beam deflection survey was performed early in the study so that locations could be selected for soil boring, disturbed granular samples, and ACP and CTB coring. The primary reason for selecting these test sample locations was high pavement surface deflections. It is significant that almost all the Benkelman beam recorded surface deflections were low (preferable condition). A pavement condition survey, which indicated that little surface distress was present, also preceded the selection of the final boring and coring locations.

Twenty-four soil bore samples were obtained by using a hollow stem auger; the average depth was about 22 ft. These borings were used to identify the soil types underlying this portion of US-12 as well as to obtain standard penetration blow counts and undisturbed and disturbed soil samples. Figure 2 shows the soil types encountered as determined from the soil bore samples and the locations where core samples were obtained.

WSDOT personnel obtained plate bearing test data at selected US-12 locations using both the 12-in. and 24-in. diameter steel plates. Table 1 gives the maximum measured deflection for each of the two plates and the corresponding maximum load. Deflections range from a low of 0.005 in. to a maximum of 0.1 in. The lower deflections occurred at stations 584-00 and 602+00. These stations contain 6-in. thick CTB layers overlaid by 3 in. of ACP. Because of the stiffness of the CTB layer, deflections were low when compared to the other locations (stations 173+25 to 453+60). The principal structural layer for these other stations is 9-in. of ACP. The variability of these data is indicated by some of the significant differences that occurred between the measured deflections at the same station for the outer wheel path (OWP) and the between wheel path (BWP) test locations (a transverse distance of approximately 3 ft).

Laboratory Testing

Samples from the field exploration phase of the study were tested by the WSDOT Materials Division and the UW Department of Civil Engineering pavement materials laboratory. Most of the tests performed can be categorized for three material groups: ACP cores, CTB cores, and granular materials. A few undisturbed subgrade soil bore samples were also provided but these samples were not incorporated into the laboratory testing program because of the complex nature of the soil profile and the stress sensitivity of the resilient modulus for these materials.

The following sections describe the kinds of tests and results for the three material groups studied.

ACP Cores

Figure 3 shows the testing sequence for selected ACP cores. The two primary material characterization tests were resilient modulus and Marshall stability and flow. Typical results are shown in Figure 4 and Table 2.

CTB Cores

Only two CTB samples were obtained and both of these

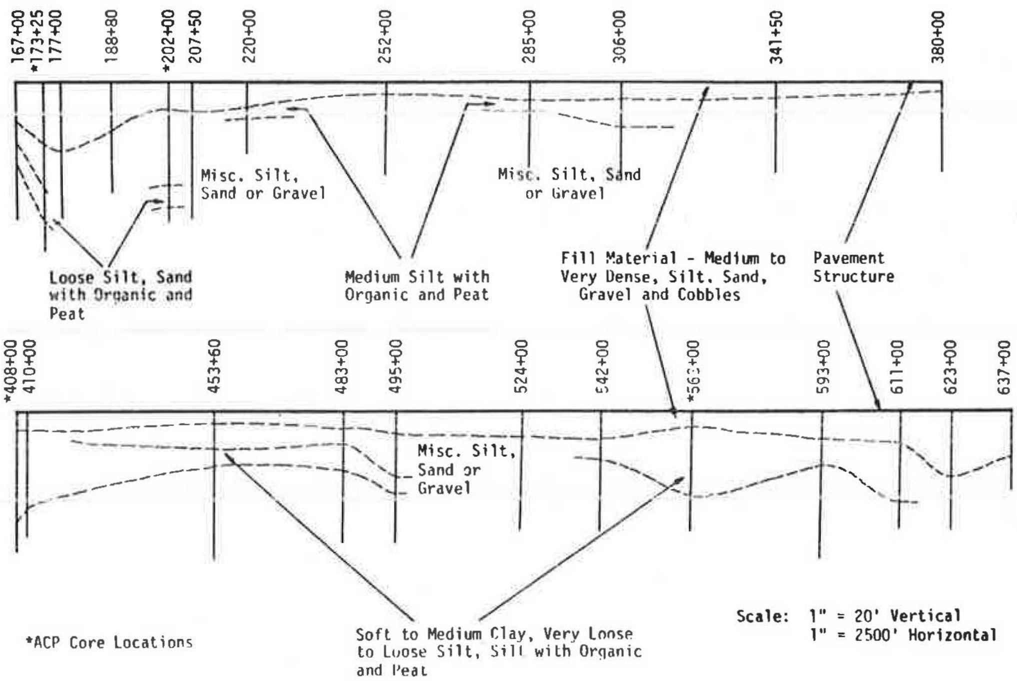

Figure 2. Generalized soil profile of proposed US-12 haul route.

Table 1. Summary of plate bearing tests for US-12.

Station[a]	12-in. Dia. Plate			24-in. Dia. Plate		
	Max. Deflection (in.)	Max. Load (lbs.)	Pavement Temperature (°F)	Max. Deflection (in.)	Max. Load (lbs.)	Pavement Temperature (°F)
173+25 (OWP)[b]	0.033	19,600	62	0.048	27,400	63
173+25 (BWP)[c]	0.077	19,600	69	0.044	27,400	68
202+00 (OWP)	0.032	19,600	64	0.016	27,400	64
202+00 (BWP)	0.044	19,600	72	0.047	27,400	69
341+50 (OWP)	0.080	19,600	80	0.035	27,400	83
341+50 (BWP)	0.054	19,600	85	0.036	27,400	87
408+00 (OWP)	-	-	-	0.026	39,000	62
408+00 (BWP)	0.051	19,600	64	-	-	-
453+60 (OWP)	0.084	19,600	74	-	-	-
453+60 (BWP)	-	-	-	0.100	35,200	75
584+00 (OWP)	0.006	19,600	59	0.005	41,000	58
584+00 (BWP)	0.012	41,000	57	0.006	37,200	56
602+50 (OWP)	0.006	19,600	62	0.010	37,200	65
602+50 (BWP)	0.012	19,600	63	0.006	37,200	60

Note: Data provided by WSDOT.
[a]All measurements made in eastbound outside lane.
[b]Outside wheel path.
[c]Between wheel paths.

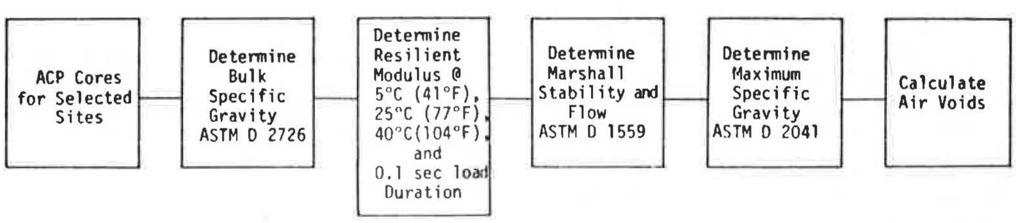

Figure 3. Test sequence for ACP cores from US-12.

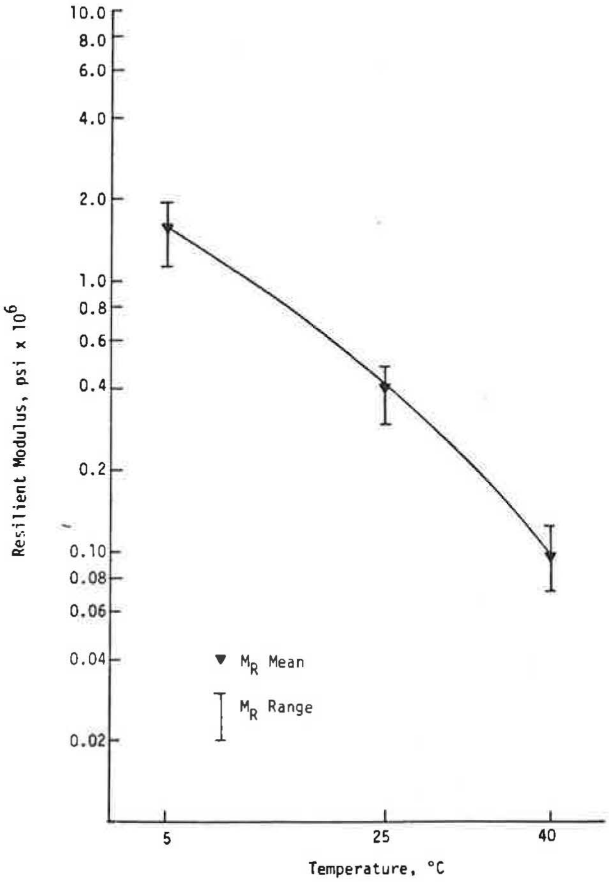

Figure 4. Typical resilient modulus (M_R) of US-12 ACP cores as a function of temperature—station 173+25.

were from station 563+00. The tests conducted on these specimens were resilient modulus and indirect tensile strength. The tensile strength ranged from 320 to 410 psi.

Granular Materials

The granular, disturbed materials sampled by WSDOT were obtained from the shoulder area of US-12 that underlies the relatively thin ACP shoulder surfacing. The samples were placed in canvas bags for delivery to the UW laboratory. Two kinds of granular samples were obtained: crushed surfacing, top-course material and gravel fill material that underlies much of the US-12 haul route. This gravel fill material contained some cobbles with sizes in excess of 2 in. Figure 5 shows the laboratory sequence used to evaluate the granular materials and a partial list of CBR results is given in Table 3.

Evaluation

The overall goal of the field and laboratory investigations of the materials was to obtain by testing, or to be able to otherwise estimate, the elastic parameters required to model US-12 pavement structures as a layered-elastic system. This evaluation uses the calculated stresses, strains, and deflections obtained from the layered-elastic modeling and applies appropriate limiting values or failure criteria to them. Thus, the potential pavement failure or reduction in pavement life attributable to the planned hauls can be estimated.

Modeling the response of the pavement structures consists of the following steps:

1. Select appropriate layered-elastic computer program (Chevron N-Layer and BISAR).
2. Select pavement structures (cross sections) to be evaluated and required material inputs (Figure 6 shows a typical example).
3. Determine load configurations (dimensions and weights, see Figures 7 and 8).
4. Select appropriate limiting values or failure criteria for the predicted pavement stresses, strains, or deflections.
5. Predict stresses, strains, and deflections by using the layered-elastic computer program (step 1) and applying to these results the appropriate limiting value or failure criteria (step 4).

Load Configurations

Two types of heavy haul loads are considered in this analysis: (a) the two trailers used to carry steam generators and the nuclear reactor vessel and (b) the prime mover vehicle. Both the trailer and the prime mover have unique numbers of wheels and wheel loads. To input these loads into a layered-elastic computer program, the analysts had to first find the critical location and then determine the number of wheels needed to distribute the load (see Figures 7 and 8).

The loading conditions used in the analysis included the following:

1. Trailer
   a. 5,675-lb wheel load, ACP temperature of 77°F,
   b. 5,675-lb wheel load, ACP temperature of 90°F,

Table 2. Summary of Marshall test results for US-12 ACP cores.

Sample No.	Bulk Specific Gravity	Air Void Content (%)	Marshall Stability (lbs)	Marshall Flow (0.01 in)
173-1-1	2.47	4.9	1250	18
173-1-3	2.42	7.0	1025	20
173-3-2	2.45	5.7	1273	22
173-3-4	2.43	6.6	1102	16
202-1-1	2.43	8.6	1763	19
202-1-3	2.41	9.5	855	22
202-3-2	2.45	7.8	631	20
202-3-4	2.45	7.8	1144	21
408-1-1	2.51	3.6	1440	25
408-1-3	2.50	4.0	1675	16
408-3-2	2.51	3.6	1904	22
403-3-4	2.53	2.8	1716	18
563-2-1	2.34	3.8	1945	20

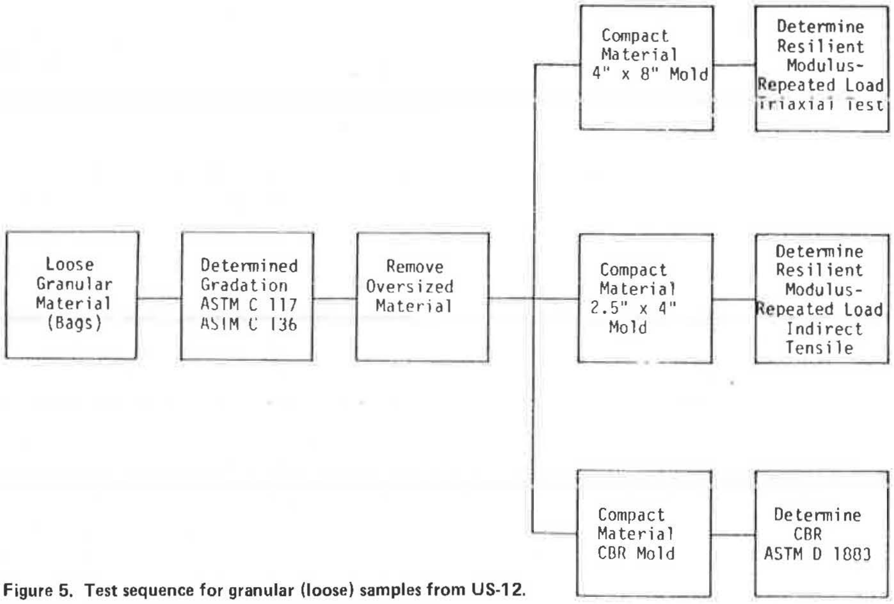

Figure 5. Test sequence for granular (loose) samples from US-12.

Table 3. Summary of CBR results for granular materials—US-12.

Sample No.	Density (lb/ft^3)		Moisture Content (%)		Swell (%)	CBR
	In situ	As Molded	In situ	As Molded		
4 (Sta. 202+00)	146.3	151.2	7.9	7.8	0	24
6 (Sta. 341+50)	146.3	152.6	7.3	7.1	0	100+
7 (Sta. 408+00)	139.0	146.0	5.4	5.6	0	100+
9 (Sta. 408+00)	151.3	153.7	6.8	7.2	0	100+
11 (Sta. 453+60)	145.2	148.8	7.0	7.3	0	100+
12 (Sta. 593+00)	143.5	149.1	6.4	5.7	0	100+

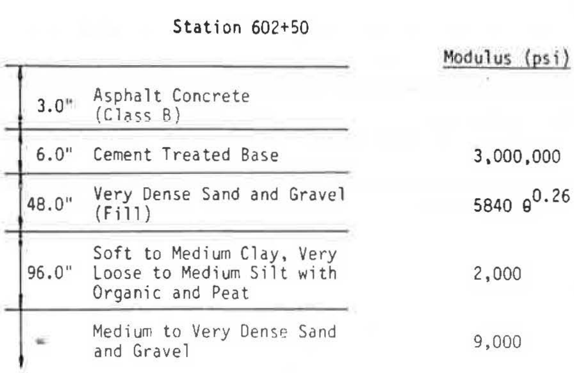

Figure 6. Cross section for station 602+50.

   c. 7,000-lb wheel load, ACP temperatures of 77°F, and
   d. 8,000-lb wheel load, ACP temperature of 77°F.
2. Prime Mover
   a. 84 tons total and
   b. 42 tons total.
3. Standard Axle
   a. 18,000-lb dual-tired single axle, ACP temperature of 77°F.

Limiting Values and Failure Criteria

The results obtained from modeling the US-12 pavement structures to determine the stresses, strains, and deflections for various loading configurations is of little value unless some limiting criteria is applied. Such criteria include limiting tensile strains for the ACP, vertical compressive strains in the subgrade layers, and tensile stresses and strains in the CTB. (Various, and often significantly different, values or relationships have been developed and reported by others.)

The limiting values and failure criteria used in this analysis fall into the following groups:

1. Fatigue (Figure 9): tensile strain at the bottom of the ACP layer,
2. Rutting (Figure 10): vertical compressive strain at the top of the subgrade layers, and
3. Strength: tensile strength of CTB layer.

A limiting value of strength was applied to the CTB layers evaluated. Because CTB is a brittle material, excessive stress induced by heavy hauls could result in cracking and ultimately accelerate deterioration of the overall pavement structure. When calculated stress exceeds the CTB indirect tensile strength, modifications to the applied load or pavement structure may be appropriate.

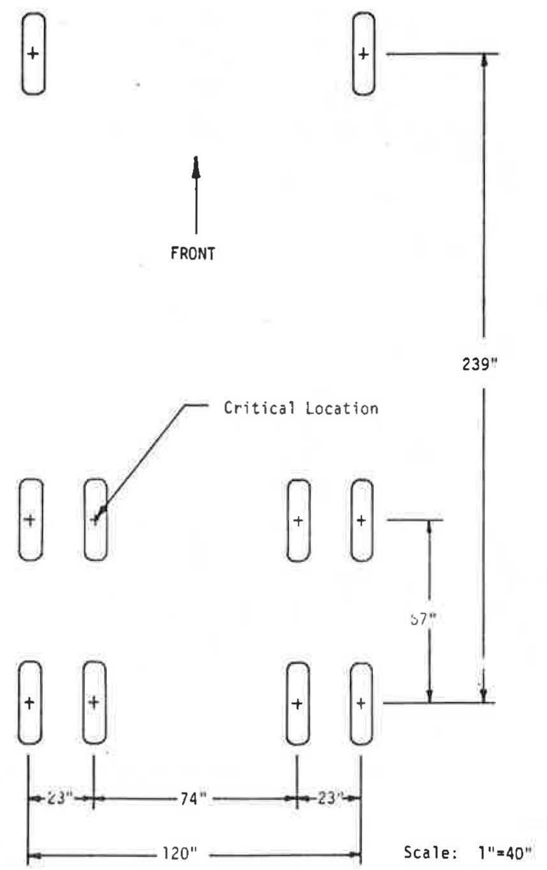

Figure 7. Plan view of prime mover wheel configuration.

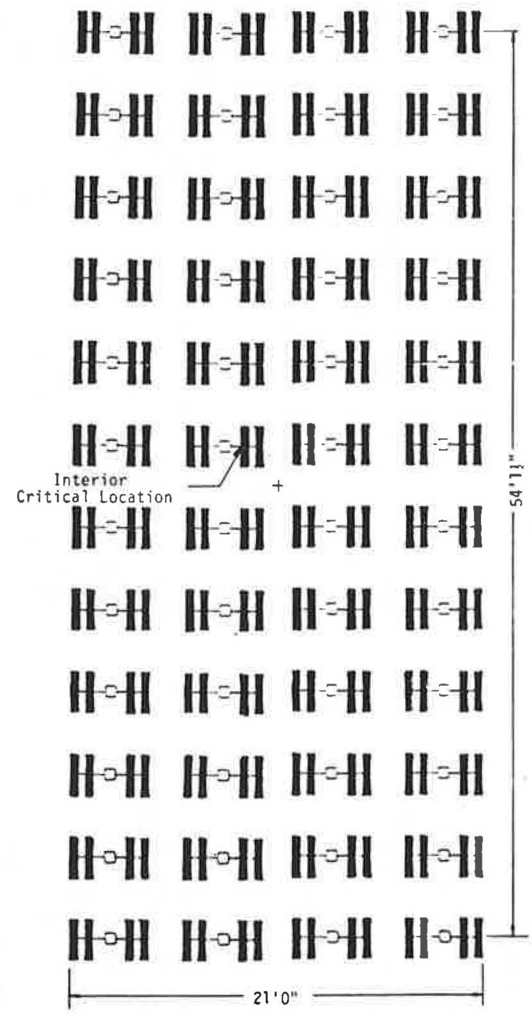

Figure 8. Plan view of one trailer unit (192 wheels).

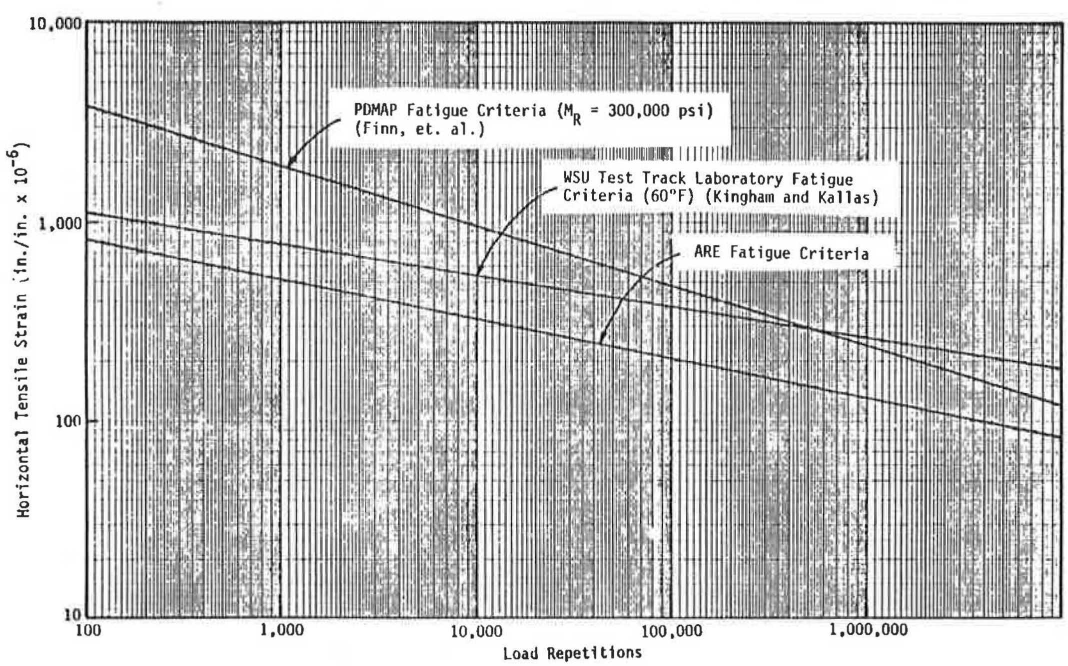

Figure 9. Fatigue criteria for horizontal tensile strain at the bottom of the ACP layer.

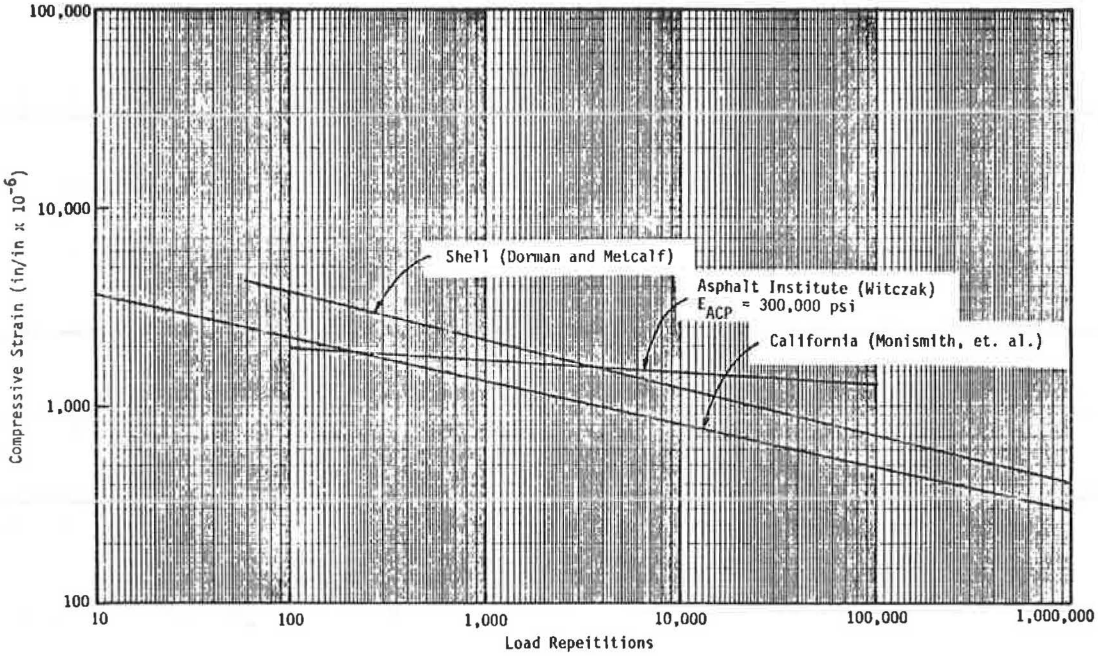

Figure 10. Criteria for vertical compressive subgrade strain to minimize rutting.

Evaluation Results

The preceding data were used to estimate potential damage to the US-12 pavement structures by the originally planned heavy hauls. The modeled pavement cross sections and haul loads were input to the BISAR computer program; then the resulting response data were compared with appropriate failure criteria to estimate pavement damage. Results obtained from the BISAR program were summarized for all pavement sections, and an example is given in Table 4.

An examination of estimated repetitions to failure in Table 5 shows the large differences between the fatigue criteria listed. The fatigue criteria developed by Kingham and Kallas from the Washington State University (WSU) test track are felt to represent best the US-12 ACP (1). The rutting criteria developed for California conditions (Table 6) were felt to represent best the US-12 subgrade soil conditions.

The maximum expected number of wheel load applications at any point along the proposed US-12 haul route would be 144 repetitions for the two trailer units (six separate moves and 24 axles for each move) and 24 repetitions for the prime movers. By dividing the maximum expected repetitions by the allowable repetitions for a specific loading configuration, an approximate estimate was made of the pavement life consumed.

Because a wide range in the fatigue criteria was observed, pavement life reductions were calculated for all three. The most probable category was assigned to the Kingham and Kallas criteria (1), the optimistic category to the Finn et al. criteria (2), and pessimistic to the ARE, Inc. criteria (3). Small percentage reductions in the pavement life are estimated for the load configurations considered for the most probable and optimistic categories (Table 7). Somewhat larger values are reported for the pessimistic category, however, the largest reduction (for the 8,000-lb wheel load on the trailer) is less than 10 percent.

Conclusions: Case 1

1. Subgrade soils along the proposed haul route were highly variable in composition and strength.

2. The existing US-12 pavement structure was well constructed and was in good structural condition. The asphalt concrete and cement treated base materials were of good-to-high quality and strength.

Table 4. Calculated stresses, strains, and surface deflections for trailer with 8,000-lb wheel load and ACP temperature of 77°F–station 408+00.

			Stress (psi)		Strain (in./in. x 10^{-6})	
Layer No.	Layer Thickness (in.)[a]	Top or Bottom of Layer	Vertical Stress	Maximum Horizontal Stress	Maximum Horizontal Strain	Vertical Strain
1	9.0	Top	-100	-237	-485	113
		Bottom	-12	190	462	-348
2	1.8	Top	-12	-4	462	-1,114
3	3.6	--	--	--	--	--
4	24.0	Top	-10	-3	464	-1,020
5	204.0	Top	-6	-1	647	-1,430
6	∞	--	--	--	--	--

Note: Tension (+); Compression (-).

[a]Pavement Surface Deflection = 0.192 in.

Table 5. Estimated repetitions to failure for various load configurations and fatigue criteria—station 408+00 (1-3).

Load Configuration	Maximum Horizontal Strain (in/in x 10^{-6})	Estimated Repetitions to Failure		
		ARE Fatigue Criteria[a]	PDMAP Fatigue Criteria[b] (Finn et al.)	WSU Test Track[c] Laboratory Fatigue (Kingham and Kallas)
Trailer(each wheel):				
5675-lb (ACP 77°F)	341	7730	312,610	146,010
5675-lb (ACP 90°F)	443	2000	132,130	31,370
7000-lb (ACP 77°F)	413	2880	166,420	47,370
8000-lb (ACP 77°F)	462	1610	115,070	24,510
Prime Mover(total wt.)				
84-ton (ACP 77°F)	532	800	72,330	10,700
42-ton (ACP 77°F)	309	12,860	432,370	260,550
Standard Axle:				
18,000-lb (ACP 77°F)	225	66,130	1,228,210	1,681,080

[a]Equation: $W_{18} = 9.7255 \times 10^{-5}(\frac{1}{\varepsilon})^{5.1627}$.

[b]Equation: $\log N_f(\leq 10\%) = 15.947 - 3.291 \log(\frac{\varepsilon}{10^{-6}}) - 0.854 \log(\frac{E}{10^{-3}})$.

[c]Equation: $\log N_f = -17.2278 + 5.87687 \log(\frac{1}{\varepsilon}) + 0.033594(\text{Temp})$.

Table 6. Estimated repetitions to failure for various load configurations and rutting criteria—station 408+00 (4-5).

Load Configuration	Maximum Vertical Subgrade Strain (in/in x 10^{-6})	Estimated Repetitions to Failure		
		Shell (Dorman and Metcalf)	California (Monismith et al.)	Asphalt Institute (Witczak)
Trailer(each wheel):				
5675-lb (ACP 77°F)	-1017	100,000	25,000	>10,000,000
5675-lb (ACP 90°F)	-1067	80,000	18,000	>10,000,000
7000-lb (ACP 77°F)	-1258	31,000	7,000	>10,000,000
8000-lb (ACP 77°F)	-1430	16,000	3,000	100,000
Prime Mover(total wt.):				
84-ton (ACP 77°F)	-	-	-	-
42-ton (ACP 77°F)	-592	1,700,000	400,000	>10,000,000
Standard Axle:				
18,000-lb (ACP 77°F)	-406	10,000,000	2,300,000	>10,000,000

Table 7. Pavement life used by heavy hauls—station 408+00.

Load Configuration	Pavement Life Consumed (%)					
	Most Probable[a]		Optimistic[b]		Pessimistic[c]	
	Fatigue	Rutting	Fatigue	Rutting	Fatigue	Rutting
Trailer (each wheel):						
5675-lb (ACP 77°F)	0.099	0.576	0.046	-	1.863	-
5675-lb (ACP 90°F)	0.459	0.800	0.109	-	7.200	-
7000-lb (ACP 77°F)	0.304	2.057	0.087	-	5.000	-
8000-lb (ACP 77°F)	0.588	4.800	0.125	-	8.944	-
Prime Mover (total wt.):						
84-ton (ACP 77°F)	0.224	-	0.033	-	3.000	-
42-ton (ACP 77°F)	0.009	0.006	0.006	-	0.187	-
Standard Axle:						
18,000-lb(ACP 77°F)	-	-	-	-	-	-

[a]Kingham and Kallas (1).
[b]Finn et al (2).
[c]ARE, Inc. (3).

3. The most probable damage to the noncement-treated base pavement sections (fatigue and rutting caused by the 5,675-lb trailer wheel loads and the 42-ton prime mover) is expected to be small--less than 1 to 2 percent of available pavement life. Increasing the trailer wheel loads to 8,000 lb would use approximately 5 to 10 percent of the available pavement life. An increase in both the wheel loads and pavement temperature would produce greater losses in pavement life. To illustrate this point, an increase of pavement temperature of only 13°F (from 77°F to 90°F) for the 5,675-lb trailer wheel load indicates that the loss in pavement life can increase by a factor of one to almost four depending on the failure criterion used.
4. Based on limited data, tensile stresses in the CTB layer due to the 5,675-lb trailer wheel loads may exceed tensile strength. The possibility therefore exists that cracking of the layer may occur. Such cracking would accelerate pavement deterioration and ultimate failure.

CASE 2, SKAGIT NUCLEAR PLANT

This proposed project, which has since been abandoned, is located in the northwestern part of Washington State (Figure 1). Many of the features of this analysis are similar to those for Satsop, although the projects were owned by different agencies and the hauling was to be done by different companies. The plant was to have been near the town of Sedro Woolley, and the nuclear reactor vessel and other large components would be transported by barge up the Skagit River then off-loaded from a barge and transferred to a special vehicle consisting of a transporter-tractor and two trailer units. This heavy hauling unit and the reactor together would weigh approximately 3,000,000 lb.

Specifically, the original objectives of the study were

1. To examine whether heavy hauling on the two pavement structures, Fruitdale Road (gravel surfaced) and US-20 (ACP surfaced), would cause pavement failure,
2. To determine relative damage to the pavements due to heavy hauling, and
3. To recommend procedures and necessary precautionary measures or construction either before hauling or afterward.

The approach used was similar to that for the Satsop project; therefore, much of the basic material data will not be presented. Two different types of roadways were included in the route: a country road (Fruitdale) and a state highway (US-20).

Field Investigation

The field study included the following:

1. Benkelman beam deflection survey,
2. Soil borings,
3. Plate bearing test, and
4. Falling weight deflectometer study.

The Benkelman beam deflection survey was performed as a first step early in the study so that soil borings, disturbed soil samples, and asphalt concrete coring locations could be selected. It was significant that almost all recorded surface deflections were high. Also, preceding the selection of the final boring and coring locations, a pavement condition survey was made which indicated that the two roads, US-20 and especially Fruitdale Road, had severe surface distress.

Soil samples at various depths and asphalt pavement cores were obtained at selected sites and were used to identify the soil type underlying the two roads as well as to obtain standard penetration blow counts and material samples. Examination of these data revealed a soil profile that was relatively uniform with respect to the kind and strength of the various soil layers encountered. Most of the materials in various layers of Fruitdale Road and US-20 are sand with varying amounts of gravel and silt.

All the asphalt core samples obtained were on US-20 because Fruitdale Road had only a surface treatment layer. All the materials below the AC layer obtained were loose, disturbed samples. Water content and in-place density were also determined for materials about 1 to 2 ft below the surface.

Plate bearing tests were conducted at the locations that appeared to be critical. The plate bearing test included the use of 8-, 18-, 24-, and 30-in. diameter steel plates. Most, if not all, deflection values at maximum plate loads were between 0.2 to 0.3 in., which indicated the two roads were relatively weak.

Additionally, FWD tests were made at selected sites along the Fruitdale Road and US-20 haul route to obtain deflection information and estimate moduli for the pavement layers.

Laboratory Testing

The granular base and subbase for both US-20 and Fruitdale Road are similar. In many areas, the subbase material is almost the same as the subgrade, extending to considerable depth. Disturbed or grab samples were received in the laboratory for testing. Both the California Bearing Ratio (CBR) and resilient modulus (M_R) were measured on laboratory compacted specimens.

These materials were generally uniform throughout and consisted primarily of sand with varying amounts of gravel and silt. In-place density and water content were determined.

Pavement Analysis

The field and laboratory material investigations were used in the pavement analysis performed for both US-20 and Fruitdale Road. The data resulting from these investigations were used to estimate the elastic parameters required to enable modeling of both highways as layered elastic structures.

By applying appropriate limiting values or failure criteria to the calculated stresses, strains, or deflection resulting from the layered elastic modeling, potential pavement failure, or reduction in pavement life was estimated for various loading conditions.

The modeling of the response of the US-20 and Fruitdale Road pavement structures required several steps and was similar to that done for the Satsop project. A typical result of the overall pavement characterization effort is shown in Figure 11.

Load Configurations

Two types of heavy haul loads are considered in this analysis: (a) the two trailers used to carry the steam generators and nuclear reactor vessel and (b) the prime mover vehicle.

Trailer

The trailer system was composed of 12 axle sets of 16 wheels (192 wheels total). Alternatively, one-half of the trailer system could be composed of 14 axle sets of 16 wheels each (for a total of 224

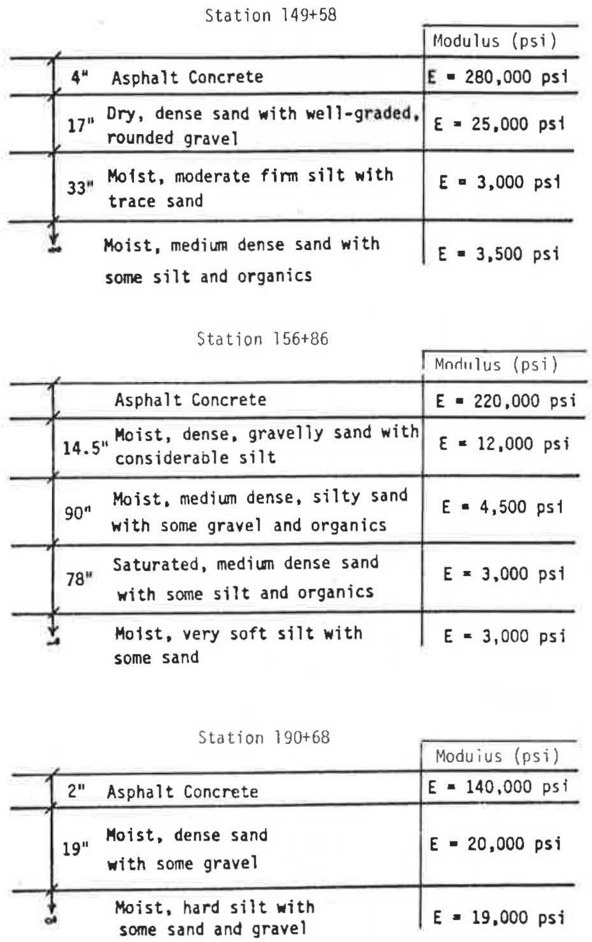

Figure 11. Typical cross sections and material parameters of various stations on SR-20.

wheels). The configuration was similar to that shown in Figure 8.

To enter these loads into the layered-elastic computer program, first the critical location was found. Based on previous work, the number of wheels selected for the trailer system was 15 clustered around the critical point. Other wheel loads further from the critical point than these 15 did not contribute significantly to the cumulative stress or strain conditions. In fact, some of these wheels tend to reduce the net stress and strain at the critical point for some loading conditions.

The expected wheel load for each wheel on the trailers is 7,750 lb for 12 axles and 6,642 lb for 14 axles. The expected total weight of a steam generator move including the trailer weight is approximately 2,976,000 lb.

Prime Mover

It was understood that two prime movers would be used for each haul, one to pull and one to push. These vehicles are heavily ballasted to increase tire contact friction with the pavement. The total prime mover load can range from 84 tons to 42 tons distributed on 10 wheels. Hence, three gross weights were studied for this vehicle: (a) 42 tons, (b) 63 tons, and (c) 84 tons. A plan view of the prime mover wheel configuration was shown in Figure 7.

Based on the above discussion, the following wheel loads were used for further pavement analysis.

1. Trailer
   a. 7,750-lb wheel load
   b. 6,642-lb wheel load
2. Prime Mover
   a. 84 tons total - 16,800-lb wheel load
   b. 63 tons total - 12,600-lb wheel load
   c. 42 tons total - 8,400-lb wheel load

Evaluation of Pavement Analysis Results

The results of the computer analysis of heavy hauls for three pavement structures (stations 49+11, 149+50, and 209+28) are summarized in Table 8. At station 49+11, the pavement structure has no significant bituminous surface layer. Thus, only the vertical strains and stresses were computed.

An examination of Table 8 shows that for station 49+11, the critical vehicle load is the prime mover at 84 tons because the surface deflection is the highest for that wheel load. Maximum vertical strains for all loads appeared to be about the same for this station.

For station 149+58 if the maximum vertical strain is used as the failure criterion, the prime mover load of 84 tons appears to be the critical load. On the other hand the horizontal strain at the bottom of the AC layer is highest for the trailer load of 7,750 lb per wheel even though the radial stress is lower than for the prime mover at 84 tons. The critical load for station 209+28 is similar to station 149+58.

Using the failure criteria previously selected, the estimated allowable repetitions of various vehicle loads for the three stations were determined and summarized in Table 9. For station 49+11, all the vertical strains due to various loads including the standard axle load of 18,000 lb are similar. The allowable number of repetitions for those magnitudes of strain are less than available rutting criteria can be used to predict.

Table 10 is a summary of the reduction in pavement life due to heavy hauls assuming that three hauls are made on each road. Using that as a base, the number of repetitions for the prime mover would be equal to 2 (prime movers) x 3 (number of axles) x 3 (hauls) or 18 repetitions. For the trailer, the number of repetitions for 7,750 lb/wheel would be equal to 2 (trailers) x 14 (axles) x 3 (hauls) or 84. By dividing the expected repetitions by the allowable repetitions for a specific loading configuration, an approximate estimate of the pavement life reduction was made.

Overlay Designs

Station 49+11 on Fruitdale Road and 209+28 on US-20 were selected for overlay designs because they were the critical sections. For trailer loads of 7,750 lb/wheel, to keep the reduction in pavement life within 10 percent, the gravel overlay for Fruitdale Road was estimated to be about 10 in. On US-20, the asphalt concrete should be about 7.5 in. For a trailer load of 6,642 lb/wheel, the gravel overlay on station 49+11 should be about 8 in. and the asphalt concrete on US-20 about 5 in. to keep pavement life within reasonable limits.

Conclusions and Recommendations: Case 2

Overall, the most critical situation was for US-20. The heavy haul exceeded criteria normally used to prevent rutting and cracking. However, the pavement was currently at or exceeding (in some areas) these criteria and it was probably unfair to apply the criteria directly as one would for a newly con-

Table 8. Summary of heavy haul responses.

Location		Vehicle Type	Load	Maximum Surface Deflection, δ, in.	Maximum Tensile Stress, σ_R, psi	Maximum Tensile Strain, ϵ_{R1} 1X10^{-6}	Maximum Vertical Strain, ϵ_{V1} 1X10^{-6}
Fruitdale Road	49+11	Trailer	7750 lb./Wh	0.3172	-	-	5,664
			6642 lb./Wh	0.2755	-	-	5,699
		Prime Mover	42 tons	0.1920	-	-	6,060
			63 tons	0.2710	-	-	5,890
			84 tons	0.3460	-	-	5,610
		Standard Axle Load	18,000 lb.	0.0902	-	-	5,680
SR 20	149+58	Trailer	7750 lb./Wh	0.2284	180	582	2,095
			6642 lb./Wh	0.1960	169	521	1,798
		Prime Mover	42 tons	0.1190	169	456	1,300
			63 tons	0.1770	190	525	1,620
			84 tons	0.2330	201	567	2,130
		Standard Axle Load	18,000 lb.	0.0394	-	411	1,000
	209+28	Trailer	7750 lb./Wh	0.3201	330	1,061	3,153
			6642 lb./Wh	0.2756	316	1,002	2,926
		Prime Mover	42 tons	0.1760	326	902	3,190
			63 tons	0.2580	345	975	3,720
			84 tons	0.3390	344	995	4,040
		Standard Axle Load	18,000 lb.	0.0647	-	848	2,460

Table 9. Summary of allowable repetitions of various heavy haul loads (1-3).

Location		Vehicle Type	Load	Tensile Strain, ϵ_R, 1X10^{-6}	Allowable Repetition, N	Vertical Strain, ϵ_V, 1X10^{-6}	Allowable Repetition, N
Fruitdale Road	49+11	Trailer	7750 lb./Wh	-	-	5,664	-[a]
			6642 lb./Wh	-	-	5,699	-[a]
		Prime Mover	42 tons	-	-	6,060	-[a]
			63 tons	-	-	5,870	-[a]
			84 tons	-	-	5,610	-[a]
		Standard Axle Load	18,000 lb.	-	-	5,680	-[a]
SR 20	149+58	Trailer	7750 lb./Wh	582	6,000	2,095	310
			6642 lb./Wh	521	11,000	1,798	720
		Prime Mover	42 tons	456	30,000	1,300	5,000
			63 tons	525	10,500	1,620	1,400
			84 tons	567	7,200	2,130	270
		Standard Axle Load	18,000 lb.	411	54,000	1,000	25,000
	209+28	Trailer	7750 lb./Wh	1,061	140	3,153	-[a]
			6642 lb./Wh	1,002	200	2,926	-[a]
		Prime Mover	42 tons	902	380	3,190	-[a]
			63 tons	975	250	3,720	-[a]
			84 tons	995	210	4,040	-[a]
		Standard Axle Load	18,000 lb.	848	580	2,460	110

[a]Allowable repetitions are low because strains exceed reasonable failure criteria.

structed pavement. In other words, some of US-20 pavement had already failed.

Even though the findings of this analysis were not implemented, the results were used to determine the improvements necessary to upgrade the two roads for heavy loads. The following improvements were recommended.

1. Reduce the loading for both the trailer and prime mover to 42 tons maximum load and place two additional lines of axles on each trailer. This would increase the total wheels to 224 per trailer and reduce each wheel load to 6,642 lb.

2. Fruitdale Road. Widen the shoulders to accommodate the wide load and add an overlay of 8 in. of compacted crushed aggregate over the entire roadway. A bituminous surface treatment before hauling should keep the surface from raveling and provide a smoother haul. A repeat of the surface treatment after hauling may be required.

Table 10. Pavement life reduction due to heavy hauls (3-6).

Location		Vehicle Type	Load	Pavement Life Reduction (%)	
				Based on ε_t	Based on ε_c
Fruitdale	49+11	Trailer	7750 lb./Wh	-	-[a]
			6642 lb./Wh	-	-[a]
		Prime Mover	42 tons	-	-[a]
			63 tons	-	-[a]
			84 tons	-	-[a]
SR 20	149+58	Trailer	7750 lb./Wh	1	23
			6642 lb./Wh	< 1	12
		Prime Mover	42 tons	0	< 1
			63 tons	0	1
			84 tons	0	7
	209+28	Trailer	7750 lb./Wh	51	-[a]
			6642 lb./Wh	42	-[a]
		Prime Mover	42 tons	5	-[a]
			63 tons	7	-[a]
			84 tons	9	-[a]

[a]Strains exceed failure criteria.

3. US-20. If the actual hauling will not take place for at least 2 years, make final surface condition and roughness surveys on US-20 just prior to the hauling operation. Following the hauling, repeat the surveys and assess the actual pavement damage due to the heavy loads. Also, an overlay of asphalt concrete will probably be required on US-20 within 2 or 3 years because of current heavy truck traffic. The heavy hauls will cause additional damage but the extent of the damage is a function of the WSDOT maintenance or rehabilitation performed to US-20 to accommodate traffic conditions.

ACKNOWLEDGMENT

The authors acknowledge the Washington State Department of Transportation (WSDOT) for the funding associated with the proposed Satsop haul route, specifically, the efforts of the personnel of the WSDOT Materials Laboratory who were crucial in the successful completion of that portion of the report work. Also, the authors acknowledge the support for the Skagit project provided by the American-Marks Company.

REFERENCES

1. R.I. Kingham and B.F. Kallas. Laboratory Fatigue and its Relationship to Pavement Performance. Res. Rept. 72-3, The Asphalt Institute, College Park, Md., April 1972.
2. F. Finn et al. The Use of Distress Prediction Subsystems for the Design of Pavement Structures. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Minn., Minneapolis, Minn., 1977, pp. 3-38.
3. Austin Research Engineers. Asphalt Concrete Overlays of Flexible Pavements--Vol. I. Development of New Design Criteria. Rept. FHWA-RD-75-75, Federal Highway Administration, U.S. Department of Transportation, Wash., D.C., June 1975.
4. G.M. Dorman and C.T. Metcalf. Design Curves for Flexible Pavements Based on Layered System Theory. HRB, Highway Research Record 71, 1964, pp. 69-84.
5. N.W. Witczak. Design of Full-Depth Asphalt Airfield Pavements. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, Univ. of Mich., Ann Arbor, 1972, pp. 550-567.
6. C.L. Monismith, N. Ogawa, and C.R. Freeme. Permanent Deformation Characteristics of Subgrade Soils Due to Repeated Loading. TRB, Transportation Research Record 537, 1975, pp. 1-17.