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Investigation of Design and Construction Issues for Long Life Concrete Pavement Strategies

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16. Abstract

This report addresses design and construction issues as they pertain to long-life rigid pavement strategies. The design and construction issues are discussed with the goal of determining the boundaries of existing technology and approaches to rigid pavement design and construction. Design issues addressed include limitations of existing design procedures and the load equivalency concept. Construction topics covered in this report are paving train productivity, concrete fast tracking, and concrete opening strength. In addition, this report includes a brief study on the formation of longitudinal cracks in existing concrete pavements.

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1.0 BACKGROUND OF LLPRS

The California Department of Transportation (Caltrans) Long-Life Pavement Rehabilitation Strategies (LLPRS) Task Force was commissioned in April 1997. The product that Caltrans has identified for the LLPRS Task Force to develop is Draft Long Life Pavement Rehabilitation guidelines and specifications for implementation on projects in the 1998/99 fiscal year. The focus of the LLPRS Task Force has been rigid pavement strategies. A separate task force has more recently been established for flexible pavement strategies, called the Asphalt Concrete Long-Life (AC Long-Life) Task Force.

The University of California at Berkeley (UCB) and its subcontractors, Dynatest, Inc., the Roads and Transport Technology Division of the Council for Scientific and Industrial Research (CSIR), and Symplectic Engineering Corporation, Inc. are investigating for Caltrans the viability of various LLPRS optional strategies that have been proposed.

2.0 OBJECTIVES

2.1 LLPRS Objectives

In recent years, Caltrans engineers and policy makers have felt that existing methods of rigid pavement maintenance and rehabilitation may not be optimum for a benefit/cost or lifecycle cost standpoint. Caltrans is also becoming more concerned about increasingly sever traffic management problems. The agency costs of applying lane closures in urban areas is very large compared to the actual costs of materials and placement, and increased need for maintenance forces to be in the roadway is increasing costs and safety risks. In addition, the costs to Caltrans' clients, the pavement users, are increasing due to the increasing frequency of lane closures, which cause delays, and the additional vehicle operating costs from deteriorating ride quality.

A need was identified to develop lane replacement strategies that will not require the long-term closures associated with the use of ordinary Portland Cement Concrete (PCC) and will provide longer lives than the current assumed design life of 20 years for PCC pavements. Caltrans has developed strategies for rehabilitation of concrete pavements intended to meet the following objectives:

1. Provide 30+ years of service life,
2. Require minimal maintenance, although zero maintenance is not a stated objective,
3. Have sufficient production to rehabilitate or reconstruct about 6 lane-kilometers within a construction window of 67 hours (10 a.m. Friday to 5 a.m. Monday).

2.2 Contract Team Research Objectives

The objective of the contract work is to develop as much information as possible to estimate whether the Long Life Pavement Rehabilitation Strategies for Rigid Pavements (LLPRS-Rigid) will meet the stated LLPRS-Rigid objectives. This Contract Team Research objective has been determined by the Caltrans LLPRS task force.

The research test plan is designed to provide Caltrans with information regarding the following aspects of the LLPRS-Rigid design options being considered by Caltrans as ways to increase the performance and reliability of the pavements being placed in the field. (1) The objectives of the research test plan are the following:

- To evaluate the adequacy of structural design options (tied concrete shoulders, doweled joints, and widened truck lanes) being considered by Caltrans at this time, primarily with respect to joint distress, fatigue cracking and corner cracking,
- To assess the durability of concrete slabs made with cements meeting the requirements for early ability to place traffic upon them and develop methods to screen new materials for durability, and
- To measure the effects of construction and mix design variables on the durability and structural performance of the pavements.

To achieve these objectives, three types of investigation are being performed:

- Computer modeling and design analysis, including use of existing mechanistic-empirical design methods, and estimation of critical stresses and strains within the pavement structure under environmental and traffic loading for comparison with failure criteria;

- Laboratory testing of the strength, fatigue properties, and durability of concrete materials that will be considered for use in the LLPRS pavements; and
- Verification of failure mechanisms and design criteria, and validation of stress and strain calculations under traffic and environmental loading by means of accelerated pavement testing using the Heavy Vehicle Simulator (HVS) on test sections constructed in the field.

The first milestone in the research project is the preparation of a set of reports identifying essential issues that will affect the potential for success of the proposed rehabilitation strategies. This report and three other reports are part of the first milestone. (32, 33, 34)

2.3 Report Objectives

The objectives of this report are to address design and construction issues as they pertain to long-life rigid pavement strategies. The design and construction issues are discussed with the goal of determining the boundaries of existing technology and approaches to rigid pavement design and construction. Several design issues addressed in this report are limitations of existing design procedures and the load equivalency concept. Construction topics covered in this report are paving train productivity, concrete fast tracking, and concrete opening strength. In addition, this report includes a brief study on the formation of longitudinal cracks in existing concrete pavements.

3.0 LIMITATIONS OF EXISTING PAVEMENT DESIGN METHODOLOGIES

3.1 American Association of State Highway and Transportation Officials (AASHTO)

Many existing design procedures are empirically based. The AASHTO Pavement Design Guide was based on the field testing of flexible and rigid pavement structures in Ottawa, Illinois in the late 1950s and early 1960s (2, 3). This empirically based pavement design procedure is used by many practicing engineers worldwide. The AASHTO guide is based on the performance of the test sections under truck traffic and environmental conditions.

One major output of the AASHO Road Test was the load equivalency factor (LEF) concept. LEFs were used to quantify the damage different axle loads and configurations caused to the pavement relative to an 80 kN single axle load (dual wheels). The equivalent single axle load (ESAL) was developed to be the total number of passes of an 80 kN standard axle. ESALs are calculated by multiplying and summing each individual axle load and configuration by its corresponding LEF for a particular pavement structure. One shortcoming of rigid pavement LEFs is that they are based on the performance of the AASHO Road Test concrete pavements, most of which failed due to pumping and erosion. This type of failure is not the predominant failure mode in many rigid pavement structures – many rigid pavements fail because of faulting and fatigue cracking. Some further limitations of the AASHTO Design guide are that the effects of widened lanes (4.3 m) and tied concrete shoulders cannot be analyzed. The AASHTO Design guide also does not directly consider joint spacing and curling stresses in rigid pavements.

3.2 Portland Cement Association (PCA)

The latest versions of the Portland Cement Association (PCA) thickness design for concrete highway and street pavements have more mechanistic features than the empirically based AASHTO guide. (4, 5) The PCA uses the load spectra analysis to calculate the bending stress in the concrete due to various axle loads and configurations. Load spectra analysis is more theoretically sound than ESAL analysis because fundamental stresses and strains are calculated and related to the performance of laboratory concrete fatigue beam tests. Load spectra analysis also allows for calculation of pavement stresses due to an axle load and configuration not originally considered in the AASHO Road Test.

The PCA guide also has many limitations, such as not taking into account temperature stresses in the slab, no ability to analyze widen lanes or different joint spacings, top of the base k-value concept, no consideration of load transfer across the shoulder-lane joint. The top of the base k-value concept refers to increasing the apparent strength of the subgrade based on the thickness and type of base material.

3.3 Overview of Mechanistic-Empirical Pavement Design Procedure

The development of mechanistic-empirical (M-E) design procedures was needed to account for situations where existing empirical studies could not be extrapolated to find a reasonable thickness design solution. Mechanistic-based design guides address the theoretical stresses, strains, and deflections in the pavement structure due to the environment, pavement materials, and traffic. These stresses, strains, and deflections are then related to the field performance of in-service rigid pavements through transfer functions. A common transfer function for concrete pavements is that of fatigue damage to cracking.

Figure 1 shows a flow chart for an M-E design procedure. An M-E design is an iterative procedure with many variables that can be changed to make the design satisfactory.

In an M-E design procedure, new, old, and current pavement features may be analyzed to determine their effect on pavement performance. Examples of pavement design features are slab thickness, shoulder type, joint spacing, load transfer devices, and base type. The pavement engineer can make changes to these design features to accommodate the specific location and constraints of the proposed pavement structure. For example, the behavior of a pavement in a high desert environment should not be expected to be the same as a pavement in a coastal environment, and some pavement design features may need to be adjusted to account for the different environments.

In contrast, with an empirical design guide such as the AASHTO Design guide, changes can be made only to the pavement features that are included in the original field testing. Extrapolation of designs not included in the original field tests could result in unrealistic designs.

In empirical design procedures, analysis is completed on the observed results of the testing. In the future, designs are based on the performance of the pavements from field testing and extrapolations are made to structures not field tested. In M-E procedures, analysis can be used to describe the failure of field tests in terms of stresses, strains, and deflections. Future designs can be outside the scope of any field testing because the mechanisms of pavement failure are quantified with theoretical analysis. Types of analysis include closed-form solutions based on plate theory such as Westergaard's solutions and the finite element method, which allows for more realistic modeling of in-situ pavement structures with complex geometries. (6, 7, 8)

A mechanistic based model is verified through calibration with field test results. A purely mechanistic model would not have to be calibrated with field data, but an M-E model still needs calibration to account for unknown slab behaviors. These unknown behaviors are also addressed in applying a reliability to the design, as shown in Figure 1. Applying a design reliability gives a factor of safety against premature failure.

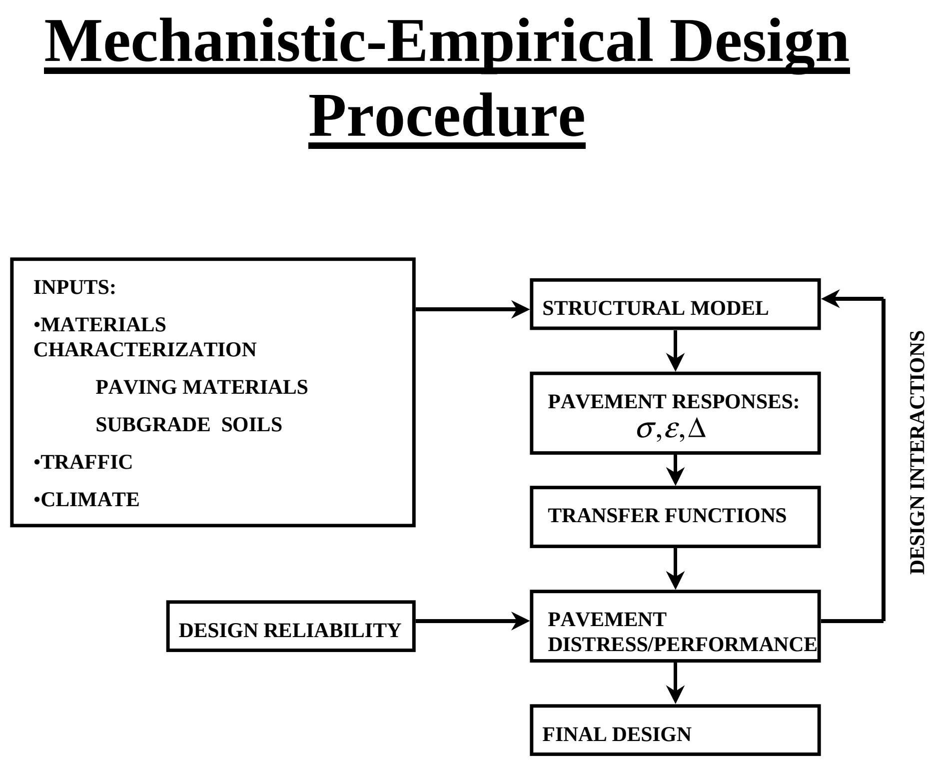

Figure 1. Flow Chart for a Mechanistic Empirical Design Procedure. [from Reference (15)]

3.3.1 Tools for Mechanistic-Empirical Pavement Design

The primary tool for a mechanistic-based pavement design is an adequate structural model. With the advent of fast computers, many analyses are completed using finite element analysis. ILLI-SLAB is one type of finite element analysis tool used to evaluate the stresses, strains, and deflections in concrete pavements. (9, 10) Other finite element programs, such as EverFE, KENSLABS, J-SLAB, and FEACON, can be used to analyze rigid pavements. (11, 12, 13, 14)

A pavement program (ILLICON) using the results of finite element analyses was developed as a pavement analysis supplement for the Illinois Department of Transportation

(IDOT) mechanistic-based rigid pavement design procedure. (15) The ILLICON program calculates the total edge stresses (load plus curl stresses) for a given set of pavement features (slab dimensions, joint spacing, temperature differentials, etc.). (16) ILLICON uses algorithms derived from a factorial of ILLI-SLAB runs for various pavement parameters. ILLICON allows the user to answer a variety of “what if?” questions regarding changes in the material properties, environmental conditions, and pavement conditions.

Performance models that can relate the number of traffic repetitions to failure in the field are needed. Performance models are empirically derived by matching calculated damage (fatigue) to observed distresses in the field. Figure 2 shows the relationship between fatigue damage and percent slabs cracked. Ideally, performance models that use laboratory data to predict field performance are preferred. In the fatigue design of concrete pavements, laboratory and field tests are used to derive a relationship between concrete stress ratio and the number of cycles to failure. Currently, laboratory fatigue test results alone cannot be used to accurately predict field performance of concrete slabs.

Another critical input to a mechanistic based design is the design traffic volume and the distribution of axle configurations and weights. The traffic volume must be predicted accurately or the design life of the pavement could be comprised. The ESAL concept is one empirical way to quantify the relative damage between axle weights and configurations. However, a more mechanistic approach is to calculate stresses in the pavement from each axle configuration and weight. This procedure is called load spectra analysis. The most important part of traffic analysis is inclusion of the heaviest axle weights in the design because they do the most damage to the pavement.

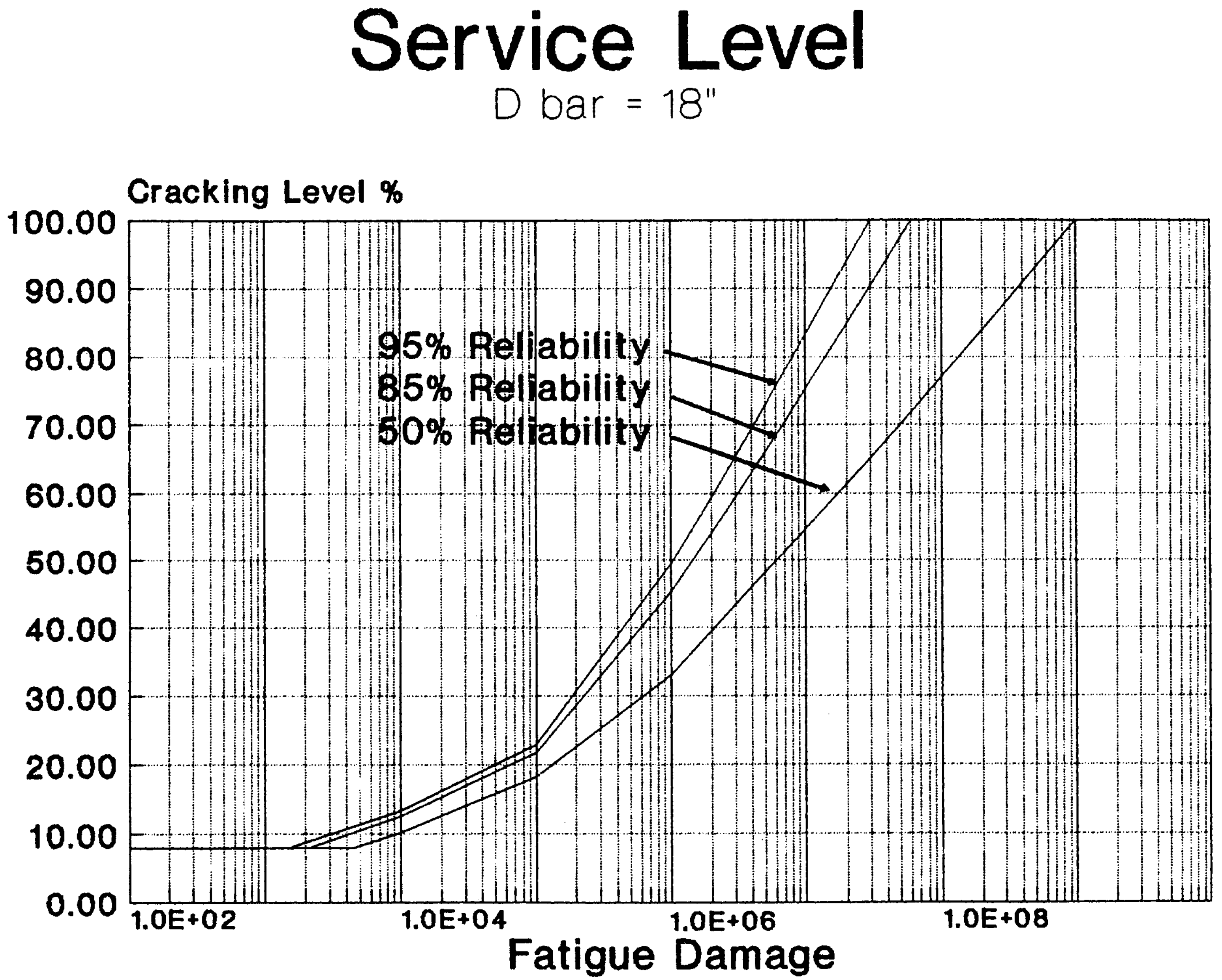

Figure 2. Relationship Between Fatigue Damage and Percent Slabs Cracked. [from Reference (15)]

The climatic region where the pavement is going to be constructed has a large impact on the stresses, strains, and deflections in the slab. Currently, only the temperature differential through the slab is addressed in mechanistic-based design procedures. Heat transfer models are able to predict the temperature gradient in the slab given the climatic conditions (e.g., rainfall, solar radiation, wind speed, air temperature, etc.) for any location. (17) This enables designers to predict maximum temperature differentials without the necessity of field measurements in regions where concrete pavements are going to be built or reconstructed.

The flexural strength or concrete modulus of rupture must be known in order to complete a mechanistic-based design. The flexural strength of a beam is tested in the laboratory to give an idea what the strength of the slab is in the field. Currently, the flexural strength of the beam is assumed to be equal to the in-situ strength of the slab. The flexural strength of the beam is used in the fatigue analysis to calculate the concrete slab stress ratio (slab bending stress divided by concrete modulus of rupture).

3.3.2 Limitations of existing mechanistic designs

The limitations of current mechanistic-based design procedures are mostly due to the inability to accurately measure certain concrete properties. For example, warping stresses in concrete due to differential moisture conditions in the slab are well known analytically, but no methodology to accurately measure them exists. The inclusion of shrinkage, thermal, and creep effects into the rational design and spacing of joints needs extensive work before a true mechanistic model can be implemented. There is also a need to understand the initial stress state of a slab after final setting of the concrete has occurred. Residual stresses may or may not exist due to the initial shape of concrete slab.

Existing concrete fatigue analyses don't predict crack initiation and propagation; they only predict when the first visual crack appears. Evidence exists suggesting that cracks initiate early in concrete slabs and propagate over the course of most of the slab's fatigue life. (18) A better understanding of crack propagation is needed to better predict the remaining lives of concrete pavements.

Many researchers have shown that concrete beams of differing dimensions and loading configuration have different flexural strengths. (19, 20, 21, 22) Furthermore, the static strength of concrete is different if tested in a beam configuration versus a slab configuration. (18) Mechanistic solutions are needed to allow for any slab size, thickness, elastic modulus, and support condition to be related to any representative beam size and loading configuration.

4.0 COMPARISON OF SEVERAL LOAD EQUIVALENCY FACTORS AND AASHTO ESALS IN RIGID PAVEMENT DESIGN

For rigid pavement design, several different methods are currently used for quantifying how various axle loads and configurations affect pavement performance. The most common of these methods involves use of the Equivalent Single Axle Load (ESAL) concept. ESALs compare the damage of any axle load and configuration to the effect of a standard 80 kN axle. Each truck axle in the analysis period is converted to a number of ESALs and the sum of all ESALs throughout the analysis period is used as the measure of total loading during a given pavement's life.

The AASHTO organization has developed the load equivalency factor (LEF) from the AASHO Road test. Use of a LEF is the most common way to convert axle loads and configuration to ESALs for flexible and rigid pavement structures throughout the world. Caltrans currently calculates LEFs based on the following equation:

$$LEF_{single} = (W_{axle}/80\ kN)^{4.2}$$

$$LEF_{tandem} = 2*[(W_{tandem}/2)/80\ kN]^{4.2}$$

$$LEF_{tridem} = 3*[(W_{tridem}/3)/80\ kN]^{4.2}$$

Caltrans LEFs are used to calculate the total number of Caltrans ESALs a pavement may experience during its design life. Tables 1-3 show comparisons between Caltrans and AASHTO LEFs for several single, tandem, and tridem axle weights.

The Caltrans and AASHTO LEFs are similar for single axle weights and thus comparable ESAL results should be expected. However, for tandems and tridems, Caltrans LEFs underestimate AASHTO LEFs, especially at higher axle weights. As mentioned previously, the

Table 1 Comparison of Caltrans and AASHTO Load Equivalency Factors (LEFs) for Single Axle Loads.

Single Axle Weight	**Caltrans LEF**	**AASHTO**	
		8 inch	10 inch
14	0.35	0.347	0.338
16	0.61	0.61	0.601
18	1.00	1	1
20	1.56	1.55	1.58
22	2.32	2.28	2.38
24	3.35	3.22	3.45
26	4.69	4.42	4.85
28	6.40	5.92	6.61
30	8.55	7.79	8.79
32	11.21	10.1	11.4
34	14.46	12.9	14.6
36	18.38	16.4	18.3

Table 2 Comparison of Caltrans and AASHTO Load Equivalency Factors (LEFs) for Tandem Axle Loads.

Tandem Axle Weight	**Caltrans LEF**	**AASHTO**	
		8 inch	**10 inch**
28	1.22	1.44	1.5
32	2.00	2.27	2.48
36	3.11	3.42	3.87
40	4.65	5.01	5.75
44	6.70	7.16	8.21
48	9.37	10	11.3
52	12.79	13.8	15.2
56	17.09	18.5	20
60	22.41	24.6	25.8
64	28.91	32.1	32.9
68	36.76	41.4	41.5
72	46.13	52.6	51.8

Table 3 Comparison of Caltrans and AASHTO Load Equivalency Factors (LEFs) for Tridem Axle Loads.

Tridem Axle Weight	Caltrans LEF	AASHTO	
		8 inch	10 inch
48	1.83	2.4	2.55
52	2.56	3.27	3.56
56	3.50	4.37	4.84
60	4.67	5.71	6.42
64	6.12	7.37	8.33
68	7.90	9.4	10.6
72	10.04	11.8	13.3
76	12.60	14.8	16.5
80	15.63	18.3	20.2
84	19.19	22.5	24.5
88	23.33	27.5	29.4

ESAL concept is empirically derived and caution should be exercised when applying ESALs to pavement structures not tested at the AASHO Road Test.

4.1 Mechanistic-Based Load Equivalency Factors

A study was undertaken to try to develop LEFs based on mechanistic principles instead of performance based LEFs or weight based LEFs. Because the damage from a vehicle depends in part on the pavement structure, it was desired that new LEFs depend on the stress an axle causes in the pavement. To do this, the stress resulting from different axle weights was evaluated for several pavement structures using the ILLICON rigid pavement design program.

The revised load equivalency factor was calculated according to an equation of the same form as above, where the stress attributable to any axle was related to the stress of an 80 kN dual wheel single axle:

$$LEF_{stress} = (\sigma_{axle})/(\sigma_{80kN})$$

Another type of load equivalency factor was also calculated using the same stresses as above, but by relating them to fatigue. For each vehicle weight, the resulting stress was calculated, which was then converted to the allowable number of repetitions (N) until fatigue failure, according to the equation (23):

$$N = 10\text{^}[17.61-17.61(\sigma/Mr)]$$

The allowable repetitions to fatigue failure were then related to the repetitions to failure from a standard 80 kN axle according to the equation:

$$LEF_{fatigue} = [N_{80\ kN} / N_{axle}]$$

The results of the stress- and fatigue-based LEF analysis are included in Tables 4 through 11. Tables 4 through 10 only include analysis on a pavement structure with bituminous shoulders. While it is apparent that the LEFs calculated from stress correspond better to current values than those calculated from the allowable repetitions to fatigue failure, neither of the new methods yields values that compare well with those currently in use. In particular, the stress LEFs are not very sensitive to axle weight, giving a large weight to lighter axles and not significantly larger weight to heavier axles. For example, a 160 kN single axle should do only 1.8 times as much damage as a 80 kN single axle for a 203 mm slab thickness. On the other hand, using allowable repetitions to failure yields very sensitive results. Light axles do hardly any damage, while a heavier single axle (160 kN) does 2.6 million times more damage than an 80 kN single axle. Using a fatigue-based LEF for conversion of axle spectra to ESALs would yield traffic projections that are much more sensitive to extreme vehicle weights. Note, ESALs were developed for a specific pavement type, loading, and environment irrespective of the distress type. Mechanistic LEFs are specific to each distress type and for each pavement, loading, and environment condition.

Table 4 Stress-Based LEF Analysis, 8-inch (203 mm) Slabs, Single Axle.

Axle Load (kips)	8-inch (203 mm) Slab		
	AASHTO LEF	**Super Singles LEF**	**Dual Singles LEF**
10	0.084	0.734426	0.609836
12	0.181	0.84918	0.711475
14	0.347	0.954098	0.809836
16	0.61	1.059016	0.908197
18	1	1.160656	1
20	1.55	1.255738	1.095082
22	2.28	1.35082	1.183607
24	3.22	1.439344	1.272131
26	4.42	1.527869	1.360656
28	5.92	1.616393	1.445902
30	7.79	1.701639	1.531148
32	10.1	1.783607	1.613115
34	12.9	1.862295	1.695082
36	16.4	1.940984	1.777049
38	20.6	2.019672	1.855738
40	25.4	2.095082	1.934426
42	31.7	2.170492	2.009836

Table 5 Stress-Based LEF Analysis, 10-inch (254 mm) Slabs, Single Axle.

	10-inch (254-mm) Slab		
Axle Load (kips)	**AASHTO LEF**	**Super Singles LEF**	**Dual Singles LEF**
10	0.081	0.722222	0.606481
12	0.175	0.837963	0.708333
14	0.338	0.944444	0.810185
16	0.601	1.050926	0.907407
18	1	1.148148	1
20	1.58	1.25	1.097222
22	2.38	1.342593	1.189815
24	3.45	1.435185	1.277778
26	4.85	1.527778	1.361111
28	6.61	1.615741	1.453704
30	8.79	1.703704	1.541667
32	11.4	1.787037	1.625
34	14.6	1.87037	1.712963
36	18.3	1.949074	1.796296
38	22.7	2.032407	1.875
40	27.9	2.111111	1.958333
42	34	2.185185	2.041667

Table 6 Stress-Based LEF Analysis, 8- and 10-inch (203- and 254-mm) Slabs, Tandem Axle.

Axle Load (kips)	8-inch (203-mm) Slab		10-inch (24-mm) Slab	
	AASHTO Tandem	Tandem LEF	AASHTO Tandem	Tandem LEF
20	0.22	0.560656	0.204	0.58
24	0.462	0.606557	0.441	0.68
28	0.854	0.737705	0.85	0.77
32	1.44	0.819672	1.5	0.87
36	2.27	0.901639	2.48	0.95
40	3.42	0.980328	3.87	1.04
44	5.01	1.052459	5.75	1.13
48	7.16	1.127869	8.21	1.21
52	10	1.196721	11.3	1.29
56	13.8	1.265574	15.2	1.37
60	18.5	1.331148	20	1.44
64	24.6	1.393443	25.8	1.52
68	32.1	1.455738	32.9	1.59
72	41.4	1.518033	41.5	1.66
76	52.6	1.577049	51.8	1.73
80	66.2	1.632787	64.2	1.81

Table 7 Stress-Based LEF Analysis, 8- and 10-inch (203 and 254 mm) Slabs, Tridem Axle.

Axle Load (kips)	8 inch (203 mm) Slab		10 inch (254 mm) Slab	
	AASHTO Tridem LEF	Tridem LEF	AASHTO Tridem	Tridem LEF
40	1.16	0.35	1.18	0.416667
44	1.7	0.41	1.77	0.481481
48	2.4	0.47	2.55	0.546296
52	3.27	0.53	3.56	0.611111
56	4.37	0.60	4.84	0.675926
60	5.71	0.66	6.42	0.74537
64	7.37	0.72	8.33	0.810185
68	9.4	0.79	10.6	0.87963
72	11.8	0.86	13.3	0.949074
76	14.8	0.92	16.5	1.018519
80	18.3	0.99	20.2	1.087963
84	22.5	1.09	24.5	1.162037
88	27.5	1.12	29.4	1.231481
92	-	1.19	-	1.300926

Table 8 Fatigue-Based LEF Analysis, 8-inch (203 mm) Slabs, Single Axle.

Axle Load (kips)	8-inch (203 mm) Slab		
	AASHTO LEF	**Super Singles LEF**	**Dual Singles LEF**
10	0.084	0.00639	0.000597
12	0.181	0.056722	0.004129
14	0.347	0.417549	0.026832
16	0.61	3.0737	0.174347
18	1	21.25803	1
20	1.55	129.7777	6.104877
22	2.28	792.2768	32.89798
24	3.22	4269.423	177.2807
26	4.42	23007.08	955.3309
28	5.92	123980.6	4836.752
30	7.79	627702.5	24488.03
32	10.1	2985806	116482.7
34	12.9	13343720	554075.9
36	16.4	59633776	2635584
38	20.6	2.67E+08	11778563
40	25.4	1.12E+09	52639008
42	31.7	4.7E+09	2.21E+08

Table 9 Fatigue-Based LEF Analysis, 10-inch (254 mm) Slabs, Single Axle.

Axle Load (kips)	10-inch (254 mm) Slab		
	AASHTO LEF	**Super Singles LEF**	**Dual Singles LEF**
10	0.081	2.37E-02	0.004979
12	0.175	1.13E-01	0.019642
14	0.338	4.73E-01	0.077485
16	0.601	1.99E+00	0.28718
18	1	7.36E+00	1
20	1.58	2.90E+01	3.706282
22	2.38	1.01E+02	12.90579
24	3.45	3.52E+02	42.22197
26	4.85	1.23E+03	129.7777
28	6.61	4.01E+03	451.904
30	8.79	1.31E+04	1478.427
32	11.4	4.03E+04	4544.243
34	14.6	1.24E+05	14866.73
36	18.3	3.58E+05	45695.87
38	22.7	1.10E+06	131961.2
40	27.9	3.18E+06	405609.1
42	34	8.62E+06	1246721

Table 10 Fatigue-Based LEF Analysis, 8- and 10-inch (203 and 254 mm) Slabs, Tandem Axle.

Axle Load (kips)	8 inch		10 inch	
	AASHTO Tandem LEF	Tandem LEF	AASHTO Tandem LEF	Tandem LEF
20	0.22	0.000234	0.204	0.003425
24	0.462	0.00056	0.441	0.013512
28	0.854	0.006794	0.85	0.047051
32	1.44	0.032317	1.5	0.163839
36	2.27	0.153723	2.48	0.536006
40	3.42	0.686994	3.87	1.753573
44	5.01	2.71009	5.75	5.389958
48	7.16	11.37907	8.21	16.56712
52	10	42.17405	11.3	47.84275
56	13.8	156.3089	15.2	138.161
60	18.5	544.2894	20	398.9832
64	24.6	1780.671	25.8	1082.509
68	32.1	5825.559	32.9	2937.031
72	41.4	19058.62	41.5	7486.746
76	52.6	58580.49	51.8	19084.36
80	66.2	169169.6	64.2	51779.11

Table 11 Fatigue-Based LEF Analysis, 8- and 10-inch (203 and 254 mm) Slabs, Tridem Axle.

Axle Load (kips)	8 inch		10 inch	
	AASHTO Tridem LEF	Tridem LEF	AASHTO Tridem LEF	Tridem LEF
40	1.16	4.6E-06	1.18	3.86E-04
44	1.7	1.41E-05	1.77	9.24E-04
48	2.4	4.34E-05	2.55	2.21E-03
52	3.27	0.000142	3.56	5.30E-03
56	4.37	0.000465	4.84	1.27E-02
60	5.71	0.00152	6.42	3.24E-02
64	7.37	0.005294	8.33	7.75E-02
68	9.4	0.018433	10.6	1.98E-01
72	11.8	0.064187	13.3	5.04E-01
76	14.8	0.237894	16.5	1.28E+00
80	18.3	0.828381	20.2	3.27E+00
84	22.5	5.729166	24.5	8.88E+00
88	27.5	10.69091	29.4	2.26E+01
92		39.62351		5.77E+01

Ioannides, *et al.* tried developing mechanistic-based LEFs similarly to the above method using the above fatigue equation and the PCA fatigue equation. (24) They were unable to find a consistent relationship between mechanistic-based LEFs and AASHTO LEFs.

A third type of LEF was developed based on the fatigue performance of the concrete pavement. This third type was also developed using ILLICON analysis. The number of repetitions of a given axle weight required to fail the pavement was determined, where failure was defined as 20 percent slab cracking. LEFs were then computed in the same form as the equation above and were found to be as sensitive as the LEF based on repetitions to fatigue failure (See Table 12). Huang presented a similar method to calculate equivalent axle load factor (EALF) based on fatigue cracking, but this method requires that the EALF be calculated for each pavement structure and loading condition. (12) It would be impossible to compile an EALF table of all possible structures and loading configurations.

The following analyses on LEFs show the sensitivity of these calculations to mechanistic parameters. Ioannides, *et al.* found it impossible to develop a mechanistic-based LEF. Due to the extrapolation of Caltrans and AASHTO LEFs from the results of the AASHO Road Test and their sensitivity, elimination of the ESAL concept may be an appropriate strategy for the development of a mechanistic-based design guide. (24) This could be accomplished by abandoning the use of aggregated traffic measures, such as ESALs, in favor of a more generalized measure, such as axle load spectra. Axle spectra analysis specifies the number of axle repetitions at a given weight and configuration for the design life of the pavement. The PCA is the only design procedure in the United States that uses axle load spectra analyses in their determination of concrete thickness for highways and streets.

Table 12 Performance Based LEF, 8- and 10-inch (203 mm 254 mm) Slab, Single Axle.

Axle Load	**8 inch (203 mm) pavement**	**10 inch (254 mm) pavement**
(kips)	**Dual Singles LEF**	**Dual Singles LEF**
10		
12	0.0048	
14	0.03	
16	0.171429	
18	1	1
20	6	37.5
22	40	107.1429
24	150	1250
26	800	3750
28		12500
30		37500
32		125000
34		375000
36		1250000

To assess the differences in pavement designs using ESALs and axle spectra, ILLICON was run for several combinations of pavement structure and traffic. Each case was run using traffic specified in terms of both ESALs and axle spectra; the results are shown in Table 13. The ILLICON results showed there was little difference in pavement thickness whether axle spectra or ESALs were used. Note, the load transfer between the tied concrete shoulder and the PCC pavement was assumed to 50 percent. This is why the bituminous shoulder and tied shoulder gave similar pavement thicknesses.

This analysis has shown Caltrans and AASHTO ESALs are within 20 percent of each other. At this time, mechanistic LEFs based on stress and fatigue do not give better results than AASHTO LEFs. Load spectra and ESALs gave approximately the same pavement design using fatigue analysis. Load spectra analysis should be used in future design since it is more theoretically sound and should be able to account for future axle loads and configurations.

Table 13 Pavement Thickness Designs, ESALs versus Load Spectra.

	Pavement Thickness in. (mm)					
LA Climate, 15' (4.57 m) Joint Spacing, k=250pci	**Bituminous Shoulder**		**Tied Shoulder**		**14 ft. (4.3 m) Widened Lane**	
	ESALs	**Spectra**	**ESALs**	**Spectra**	**ESALs**	**Spectra**
San Diego LTPP	10.5 (267)	10.5 (267)	10.5 (267)	10.5 (267)	8 (203)	8 (203)
San Joaquin LTPP	10.5 (267)	10.5 (267)	10.5 (267)	10.5 (267)	8 (203)	8 (203)
LA Climate, 19' (5.79 m) Joint Spacing, k=250pci	**Bituminous Shoulder**		**Tied Shoulder**		**14 ft. (4.3 m) Widened Lane**	
	ESALs	**Spectra**	**ESALs**	**Spectra**	**ESALs**	**Spectra**
San Diego LTPP	12 (305)	12 (305)	12 (305)	11.5 (292)	9 (229)	9 (229)
San Joaquin LTPP	12.5 (305)	12.5 (318)	12.5 (318)	12 (305)	9.5 (241)	9.5 (241)

5.0 LONGITUDINAL CRACKING ANALYSIS

The design of concrete pavements has to be targeted for certain distress types. Transverse cracking is typically associated with fatigue damage, while faulting is a result of poor load transfer at the joint and erosion of the base. A large number of pavements in California exhibit longitudinal cracking. Several studies have found the major distresses on Caltrans highways are transverse and longitudinal cracking and faulting. (23, 25) Macleod and Monismith found 97 percent of pavements designed before 1967 had transverse cracking as the major distress type. (26) The major distress type found on concrete pavements designed after 1967 was longitudinal cracking (60 percent).

Although these studies reference longitudinal cracking as a major distress on California PCC pavements, no published information on the mechanism behind this crack formation in California has been found. One plausible answer for some of the longitudinal cracking is the use of plastic joint inserts for the longitudinal joint instead of saw cutting. There is some evidence that the plastic joint inserts were briefly utilized new PCC construction in California, however inserts were not used in the majority of newly constructed concrete pavements and therefore cannot be the main cause for longitudinal cracking occurrence. Longitudinal crack analysis has to be addressed to avoid having this type of crack reappear on the future concrete pavements especially long-life sections.

Mahoney, *et al.* reviewed urban freeways in the state of Washington during the late 1980s and found longitudinal cracks were predominant on Washington concrete pavements. (27) Much of the early concrete pavement designs in Washington were based on experience and information taken from California. A rigorous surveying, coring, and analysis study by Mahoney, *et al.* found the longitudinal cracks were probably a result of some type of fatigue cracking at the transverse joint. The main evidence for this was cores taken from the in-situ pavement showed the longitudinal cracks started from the bottom of the concrete slabs.

FWD measurements by Mahoney, *et al.* found 30 percent of the joint deflections on one highway had lower deflections than in the middle of the slab. (27) One explanation for this was a pre-compression in the concrete pavement. A mechanistic analysis completed at the transverse joint found for 92 percent load transfer efficiency (LTE) at the joint, a 0.86 MPa in-plane compressive force resulted in equivalent fatigue damage at the transverse joint and longitudinal edge of the slab. Several other conclusions from this study were the following:

1. The critical fatigue location at the transverse joint was in the inner wheel path 2.6 m from the pavement edge for a lateral traffic distribution centered at 457 mm.
2. A secondary fatigue location could occur at 762 to 914 mm from the pavement edge.
3. The pass to coverage ratio or percentage of traffic to be considered in fatigue analysis at the transverse joint is 26 percent.

This analysis completed by Mahoney gives a potential answer to the question of how longitudinal cracks occur. However, based on existing fatigue damage to percent slabs cracked models, the calculated fatigue damage was several orders of magnitude lower than the fatigue damage needed to cause cracking. At this time, this explanation appears to be the most promising.

Several University of California at Berkeley personnel drove kilometers of California pavements (I-80, I-5, I-405, I-10, I-710, I-215, SR60, SR14), and observed the following characteristics of longitudinal cracking:

1. Whether cracks initiate at, approach, or leave slab can not be determined.

2. Most longitudinal cracks run the entire length of the slab (parallel to the direction of traffic).
3. Longitudinal cracks occur on both cut-and-fill and at grade pavement sections.
4. Corner breaks are sometimes seen on sections with longitudinal cracking.
5. Longitudinal cracks occur on both skewed and right angle joints.
6. Longitudinal cracks occur on high and low faulted pavements, but are more prevalent on highly faulted pavements.
7. Transverse contraction joints did not include load transfer devices (dowels).
8. New and old PCC pavements exhibit longitudinal cracks with older slabs having more severe spalling from the cracks.
9. A cement treated base (CTB) layer is most likely under all PCC slabs that exhibit longitudinal cracking.
10. CTB is 100 to 150 mm thick with a low compressive strength (< 10 MPa).
11. There are no longitudinal tie bars present on any lanes.
12. Longitudinal cracks occur in areas of high and low rainfall.
13. Longitudinal cracks occur in high traffic lanes (i.e., truck lanes). No significant cracking is visible in the passing lanes.
14. Longitudinal cracks occur on pavements without joint sealant.
15. Longitudinal cracks appear to be in the wheel paths.
16. In a given section, longitudinal cracks can occur in either or both wheelpaths.
17. Longitudinal cracks can occur on consecutive slabs.

Reasons 7 through 11 are based on the Caltrans Rigid Pavement Design Guide as it applied to the approximate time the pavements were constructed.

Another hypothesis for longitudinal crack formation is the combination of a large negative temperature gradient (nighttime) in the slab, stiff base, and heavy truck traffic near the slab corners. This combination of loading conditions could cause the maximum tensile stress in the slab to occur at the top of the slab at the transverse joint. If this tensile stress exceeds the strength of the concrete, or is high enough to result in fatigue damage at that point, then cracking may occur.

Analytical analyses and field measurements by Yu, *et al.* found residual negative temperature gradients in concrete slabs could cause top-down fatigue cracking. (28) For this stress at the transverse joint to occur, it must be greater than the bending stress induced by a wheel load located at the edge of the slab. Some initial runs on a finite element program found that if no voids existed under the slab at the joint, then the maximum slab stress could not be at the top of the slab near the corner. Furthermore, most of the cement treated bases or soils used in California were constructed of low strength materials and probably would not affect the curling stresses in the slab appreciably.

5.1 Longitudinal Crack Finite Element Analysis

A final hypothesis for longitudinal crack formation is the joints fill up with debris that causes restraint cracking due to shear of the slab at the joint. Figures 3a through 3c show several hypothetical loading conditions caused by incompressible debris in the joints. A simple finite element analysis was performed to analyze the effects of joint incompressibles on the stress state in a slab. The purpose of the analysis was to see if the stress in the concrete could reach or exceed the flexural strength of the concrete. Identification of where and how the incompressibles entered the joint and restrained the joint in a critical manner was not addressed in this analysis. The finite element analysis was performed using the FEAP program developed at the University of California, Berkeley.

5.1.1 Finite Element Analysis Mesh

A mesh was constructed to represent the concrete slab in two dimensions. The slab width was 3.6 meters and the length was 5.5 meters. A half-slab model of the geometry and boundary conditions is given in Figure 4. Three different combinations of incompressible debris-filled joints were analyzed: half the length of the joint was filled, a quarter of the joint length was filled, and the wheel paths were filled (see Figure 3). The incompressible materials in the joint were modeled by applying one-dimensional rods at each node. The stiffness of the rod is almost double the stiffness of the concrete. In a test simulation the rod stiffness was tripled and the concrete response only changed by a small amount. The radius of the rods is approximately one third of the slab element size and the length of the rods is 3 mm (represents half the joint opening). A smaller joint opening was not possible in the finite element analysis software used.

The slab was analyzed in plane strain, which assumes infinite depth in the z direction. This presents a "worst case" scenario as there is no support resisting horizontal movement of the slab where the joint is free.

5.1.2 Finite Element Analysis Loading

Loading is applied by a horizontal displacement from the fixed boundary edge in the positive x direction. The amount of the displacement applied to the slab was equal to the thermal expansion of the slab from several temperature changes. The following formula was used to calculate the appropriate slab displacement:

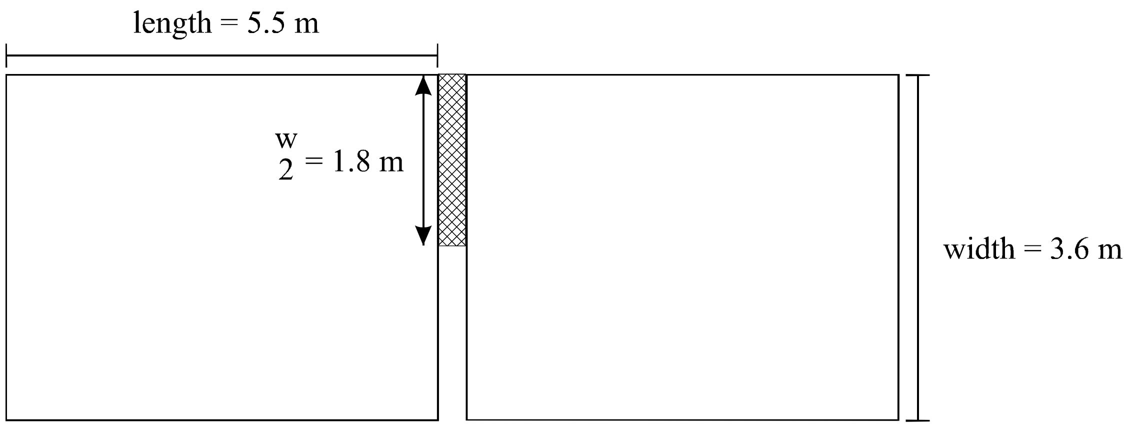

Figure 3a. Incompressible Debris Filling Half the Joint.

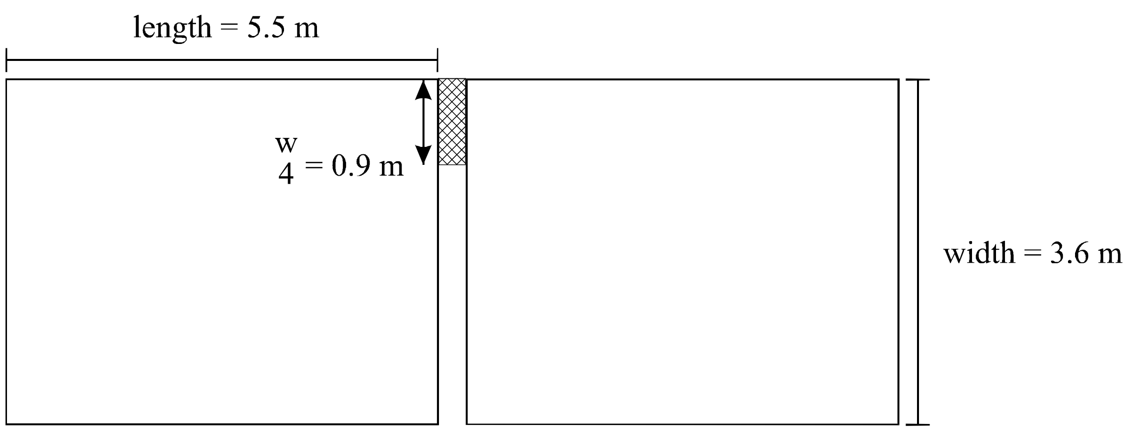

Figure 3b. Incompressible Debris Filling One Quarter of the Joint.

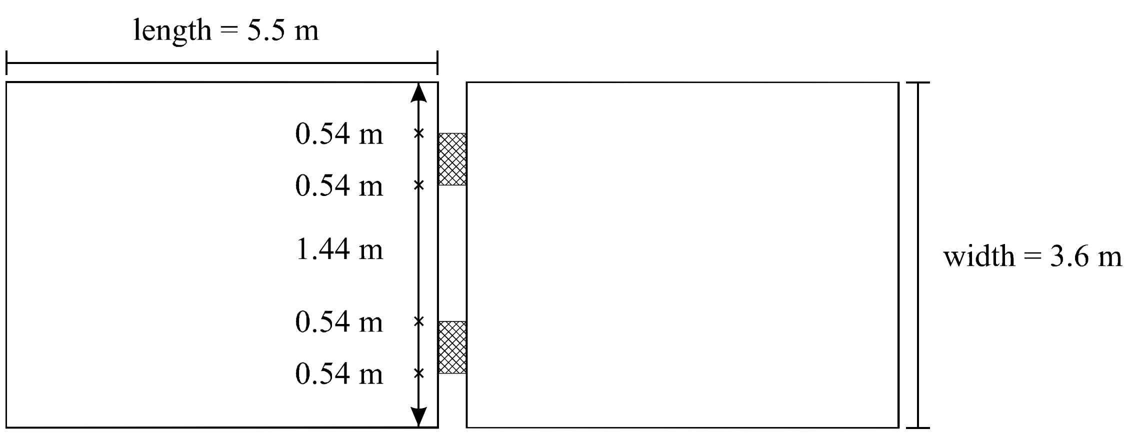

Figure 3c. Incompressible Debris in the Joint in the Wheelpaths.

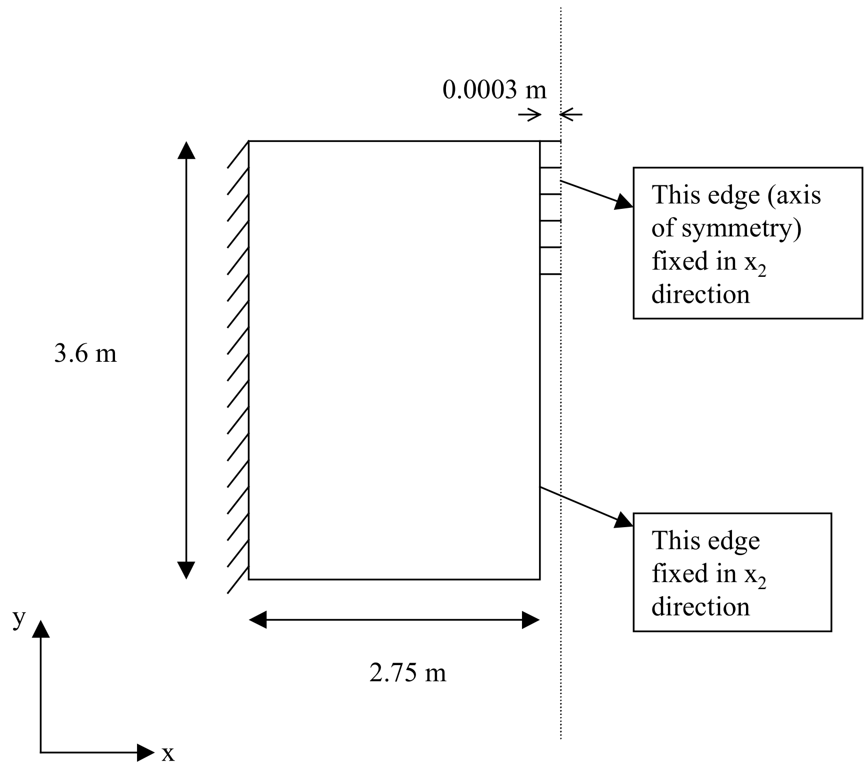

Figure 4. Finite Element Analysis Half-Slab Model Showing the Geometry and Boundary Conditions.

$$\text{Displacement} = \alpha\Delta T$$

where α is the thermal coefficient of expansion and ΔT is the change in temperature. Three values for ΔT were used, 6, 11, and 17 C and α was 1.0×10^{-5}mm/mm/°C.

5.1.3 FEA Results

Principal stress results in MPa from these analyses are presented in Figures 5-7. The plots are all from the ΔT = 17°C case because this temperature change would cause the most critical case. In Figures 5-7, the sign convention is tension positive and compression negative.

The highest stress occurred at the first unrestrained node for all cases. There was a large stress gradient surrounding the node of maximum tensile stress. A large reduction in stresses was experienced further away from the restrained joint opening. The absolute stress values may not be precise due to the simplification of the model. Table 14 lists the maximum principle stress (tension) for each loading configuration at 17°C.

Table 14 Maximum Principal Stress for Each Finite Element Analysis Loading Configuration at 17 C.

Loading Configuration	Principal Stress (Tension)
Half Filled Joint	4.6 MPa
Quarter Filled Joint	4.9 MPa
Wheel Path Filled Joint	4.2 MPa

The most critical joint configuration occurred when the joint was one quarter filled. In this case, the mesh was refined or made finer and the result was the maximum stress increased from about 4.9 MPa to about 10 MPa. This large increase in stress due to mesh refinement indicates there is a large stress concentration occurring from this loading configuration. Typical concrete flexural strength ranges from 4 to 5 MPa. At this concrete strength, it appears from this preliminary analysis that restraint at the joints from incompressibles may result in longitudinal cracking.

If this hypothesis turns out to be true, then it may be appropriate for all joints to be sealed to prevent ingress of incompressibles. There are some unaddressed concerns regarding this hypothesis, such as how the incompressibles orient themselves in the joint to cause cracking and why aren't more longitudinal cracks seen in other lanes, especially the number one lane. These points have to be researched further to confirm or rule out this hypothesis.

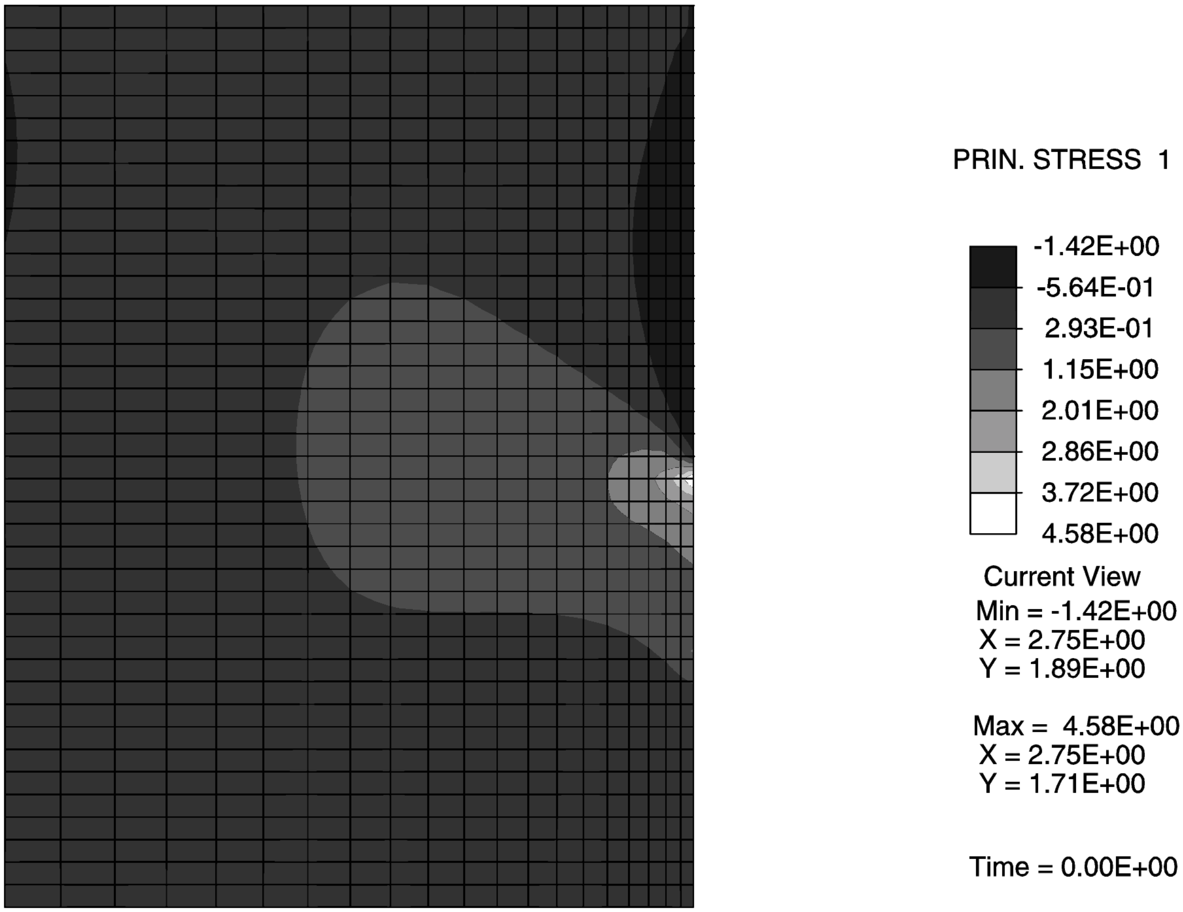

Figure 5. Principal Stress Results from Finite Element Analysis Loading of Half Filled Joint Case.

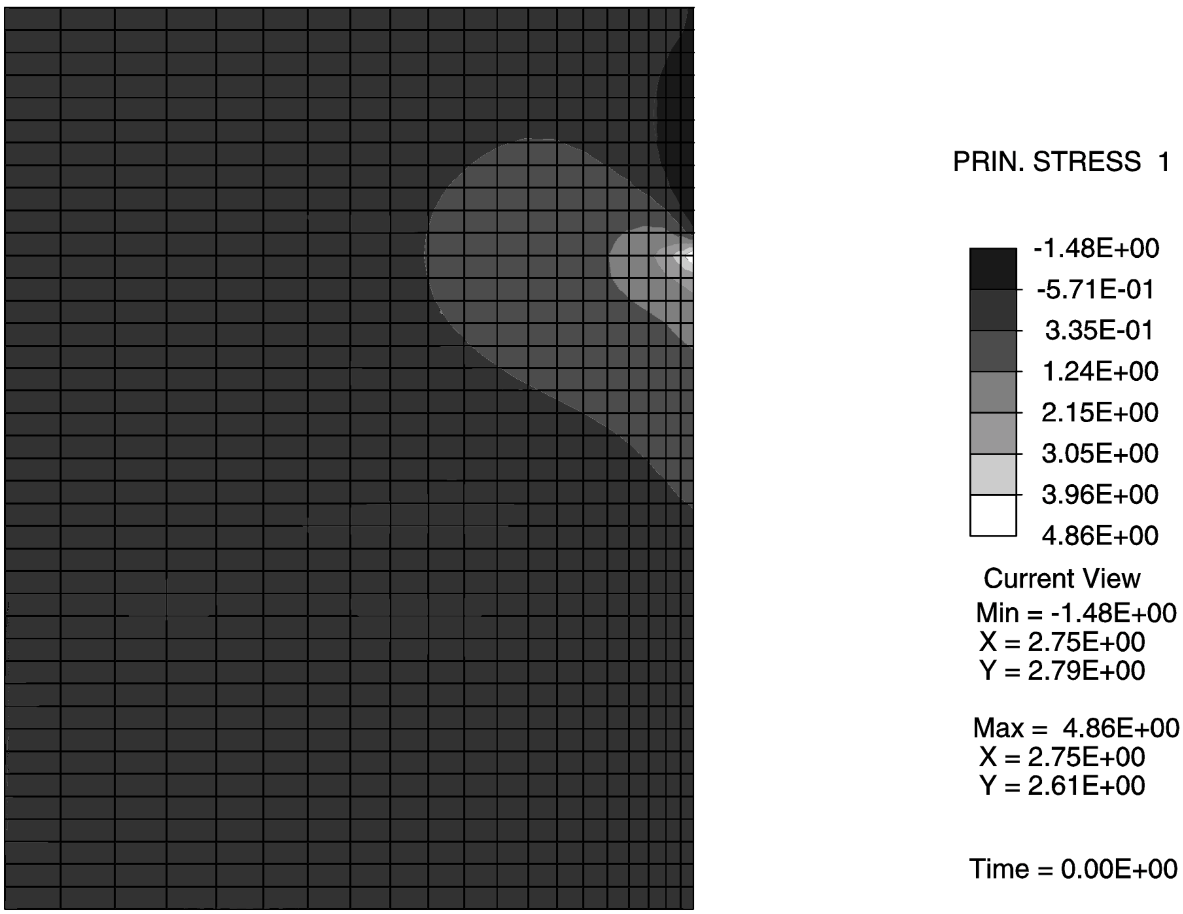

Figure 6. Principal Stress Results from Finite Element Analysis Loading of Quarter Filled Joint Case.

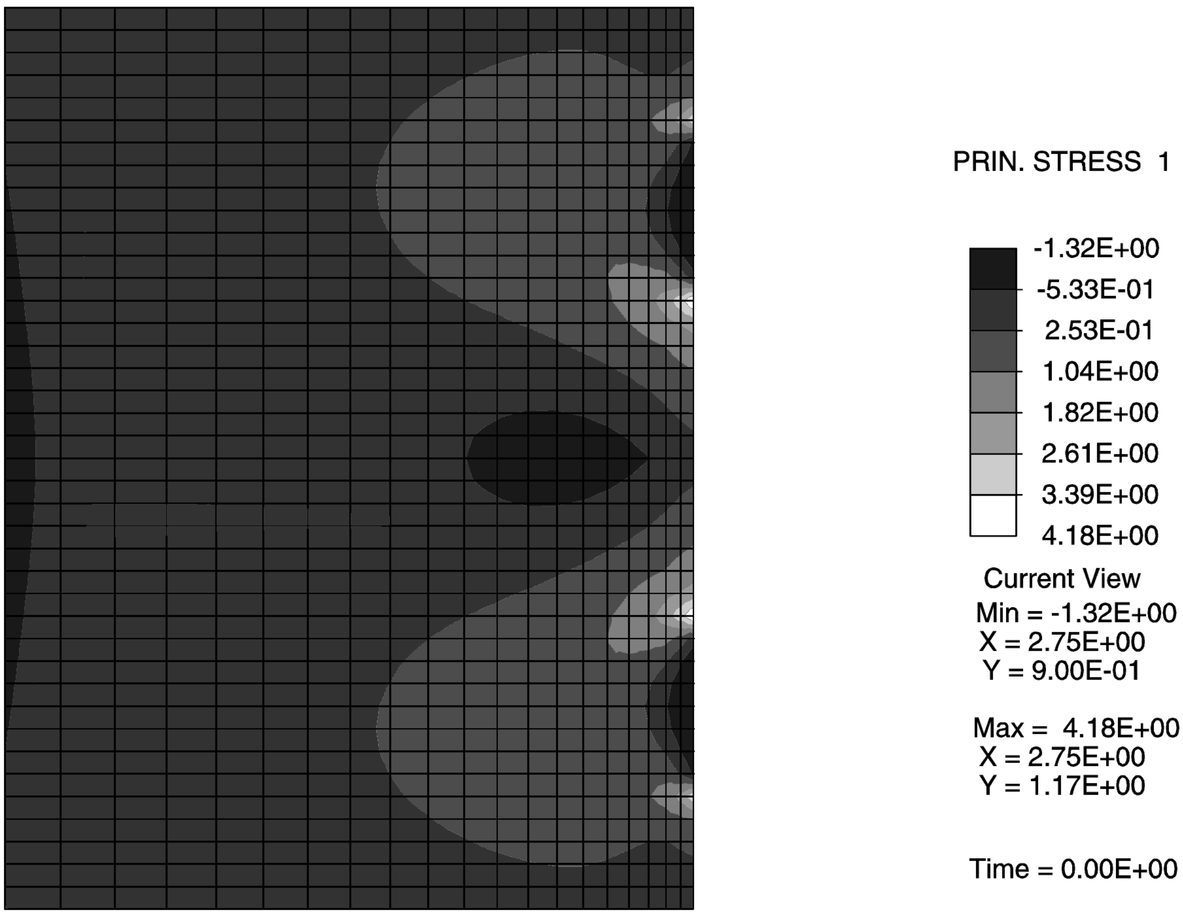

Figure 7. Principal Stress Results from Finite Element Analysis Loading of Joint Filled in the Wheelpath Case.

6.0 CONCRETE PAVEMENT OPENING TIME TO TRAFFIC

The primarily need for fast setting cements is for their early strength gain properties. Pavements in areas where long lane closures or closures at inopportune times require rapid setting concrete to minimize traffic congestion problems. The one concern that has to be addressed with respect to the use of FSHCC is at what strength can traffic (truck and car) be placed on the rehabilitated concrete slabs. This topic has been addressed by other state DOTs and the American Concrete Paving Association (ACPA) for fast-track paving operations. (29) Airport construction is one area that currently requires high early strength concrete because closure of any runways or taxiways result in a loss of capacity and revenue to the airport.

The ACPA compiled a fast-track paving technical memorandum. (29) Table 15 shows the opening strengths and strength data for various fast-track projects. The materials used by other agencies to achieve high early strength concrete are Type I PCC with accelerators, Type III PCC with mineral admixtures such as fly ash or silica fume, and other proprietary fast setting hydraulic cement concrete products (e.g., Rapid Set from CTS and Five Star Highway Patch from Five Star Products).

The main concern with opening the rehabilitated concrete to traffic is premature cracking of the slabs. If the flexural strength of the concrete is not sufficient to resist the applied truck loads, then flexural fatigue cracking will result. Table 16 lists the recommended opening strength for a variety of pavement features taken from an FHWA report. (30) The required strength for opening to traffic was based on fatigue analysis and the estimated number of ESALs the pavement could resist before fatigue cracking. The required minimum flexural strength for all pavements was 2,068 kPa (300 psi).

Table 15 Opening Strengths and Other Data for Several Fast-Track Paving Projects. (from ACPA [29])

Location and Description	Year	Cement Type	Cement Content kg/m^3 (lb/yd^3)	Water/Cement Ratio	Fly Ash kg/m^3 (lb/yd^3)	Curing/ Insulation	Opening Strength Specified MPa (psi)	Time to Meet Specified Strength, Hours
US-71 Bonded Overlay Storm Lake, IA	1986	III	380 (640)	0.45	42 (70) Type C	Wax-Based Compound/R=0.5 Blankets	Flexural 2.4 (350)	7.5
Runway Keel Reconstruction Barksdale AFB (LA)	1992	Special Blended	418 (705)	0.27	None	Wax Based Compound/None	4 Hr. Flex. 3.1 (450)	4
Highway 100 Intersection Replacements Cedar Rapids, IA [1]	1988	III	440 (742)	0.380	47 (80) Type C	Wax-Based Compound/R=0.5 Blankets	12 Hr. Flex. 2.8 (400)	7.5
SR-81 Arterial Reconstruction Manhattan, KS	1990	III	427 (719)	0.44	None	Wax-Based Compound/R=0.5 Blankets	Flexural 3.1 (40)	24
Lane Addition to I-496 Lansing, MI	1989	III	(418 (705)	0.45	None	Wax-Based Compound/R=0.5 Blankets	24 Hr. Flex. 3.8 (550)	19
I-25 to I-70 Interchange Ramp Reconstruction Denver, CO	1992	I	446 (752)	0.32	None	Wax-Based Compound & Plastic Sheets/None	12 Hr. Comp. 17.2 (2500)	8 [3]
Single-Route Access Road Reconstruction Dallas County, IA	1987	III	380 (640)	0.425	42 (70) Type C	Wax Based Compound/None	Flexural [2] 2.4 (350)	9
Interstate 80 Widening Rawlins, WY	1992	III	(390 (658)	0.47	None	Wax Based Compound/None	24 Hr. Comp. 20.7 (3000)	20 [3]
SR 832 and I-90 Interchange Reconstruction Erie County, PA	1991	I	446 (751)	0.37	None	Monomolecular Compound & Plastic Sheets/R=2.5 Blankets	24 Hr. Comp. 20.7 (3000)	13
I-70 Bonded Overlay Copper County, MO	1991	III	421 (710)	0.40	None	Polyethylene Sheets/None	18 Hr. Comp. 24.1 (3500)	10
Runway 18/36 Extension Reconstruction Dane County, WI	1992	III	392 (660)	0.455	None	Wax Based Compound/None	12 Hr. Comp. 24.1 (3500)	11 [3]
SR 13 Bonded Overlay North Hampton, VA	1990	II	445 (750)	0.420	None	Wax-Based Compound.R-0.5 Blankets	24 Hr. Comp. 20.7 (3000)	18
US-81 Reconstruction Menominee, NE	1992	III	363 (611)	0.423	None	Wax Based Compound/None	24 Hr. Comp. 24.1 (3500)	36
US-70A Inlay of Asphalt Intersection Approaches Smithfield, NC	1990	I	424 (715)	0.35	None	None/R=0.5 Blankets	48 Hr. Flex. 3.1 (450)	18

1) Contractor had two fast track mix choices on the project depending on the desired set speed – details are for faster set mix and intersection work.
2) Centerpoint flexural strength (flexural strength for all other projects in table are third point).
3) Interpreted from available data.

Table 16 Recommended Opening Flexural Strengths (psi) for a Variety of Pavement Structures. (30)

Slab Thickness in. (cm)	Foundation Support psi/in. (kPa/cm)	Modulus of Rupture for Opening (psi), to Support Estimated ESALs Repetitions to Specified Strength				
		100	**500**	**1000**	**2000**	**5000**
8 (20.3)	100 (271)	370	410	430	450	470
	200 (543)	310	340	350	370	390
	500 (1357)	300	300	300	300	310
8.5 (21.6)	100 (271)	340	370	380	400	430
	200 (543)	300	300	320	330	350
	500 (1357)	300	300	300	300	300
9 (22.9)	100 (271)	300	300	320	360	390
	200 (543)	300	300	300	300	320
	500 (1357)	300	300	300	300	300
9.5 (24.1)	100 (271)	300	300	300	330	350
	200 (543)	300	300	300	300	300
	500 (1357)	300	300	300	300	300
10 (25.4)	100 (271)	300	300	300	300	320
	200 (543)	300	300	300	300	300
	500 (1357)	300	300	300	300	300
10.5 (26.7)	100 (271)	300	300	300	300	300
	200 (543)	300	300	300	300	300
	500 (1357)	300	300	300	300	300

A brief fatigue analysis was performed with an existing mechanistic-empirical rigid pavement design/analysis program called ILLICON. The ILLICON program calculates the cumulative damage in the concrete pavement due to truck traffic and temperature curling. In this simplified analyses, temperature curling was assumed to be zero and only pavement thickness, mean distance from the slab edge, and concrete strength were varied. The following are the inputs used in the ILLICON analysis:

Concrete Modulus of Elasticity = 28 GPa

Concrete Thickness = variable

Slab Length = 5.8 m

Concrete Strength (Third Point at 90 days) = variable

Mean distance from Slab Edge = variable

Base Modulus of Elasticity = 3.4 GPa

Base Thickness = 102 mm

Poisson's Ratio = 0.15

Bituminous Shoulder

Modulus of Subgrade Reaction (k-value) = 27 MPa/m

Concrete pavement failure was assumed to occur when 20 percent of the slabs were cracked in the ILLICON analysis. Table 17 shows the results of the fatigue analyses for early opening time for concrete pavements. The results in Table 17 relate the number of ESALs required to have 20 percent slabs cracked. For 203 mm pavements, the minimum concrete strength should be around 3100 kPa (450 psi). However, if the slab thickness is 254 mm, the minimum concrete strength could be as low as 2240 kPa (325 psi). One factor in selecting the correct strength for opening to traffic is the expected number of ESALs per day the concrete pavement may experience. Current long term pavement performance (LTPP) "high truck traffic" data from two locations in California is shown in Table 18.

The high level of traffic per day on these pavements indicates the required opening strength should be as high as 3450 kPa (500 psi) if the pavement is located in the San Joaquin Valley and is 203 mm thick. On the other hand, if a 254 mm concrete pavement is constructed, then 2240 kPa (325 psi) concrete may be sufficient if the traffic is kept away from the slab edge.

Table 17 Results of Fatigue Analyses for Early Opening Time for Concrete Pavements.

	ESALs to 20 Percent Slab Cracking				
Slab Thickness = 8" (203 mm)	**Mean Distance from Edge of Pavement**				
MR psi (kPa)	0" (0 mm)	6" (152 mm)	12" (305 mm)	18" (457 mm)	24" (610 mm)
300 (2068)	<1000	<1000	<1000	<1000	<1000
325 (2241)	<1000	<1000	<1000	<1000	<1000
350 (2413)	<1000	<1000	<1000	<1000	<1000
375 (2585)	<1000	<1000	<1000	<1000	<1000
400 (2558)	<1000	<1000	<1000	<1000	<1000
425 (2903)	<1000	<1000	<1000	1000	5000
450 (3103)	1000	1000	5000	12000	35000
475 (3275)	6000	13000	27000	60000	>100000
500 (3447)	20000	50000	110000	270000	>100000
Slab Thickness = 9" (229 mm)	**Mean Distance from Edge of Pavement**				
MR psi (kPa)	0" (0 mm)	6" (152 mm)	12" (305 mm)	18" (457 mm)	24" (610 mm)
300 (2068)	<1000	<1000	<1000	<1000	<1000
325 (2241)	<1000	<1000	<1000	<1000	<1000
350 (2413)	<1000	<1000	<1000	1000	4000
375 (2585)	1000	2000	4000	11000	30000
400 (2558)	7000	16000	32000	80000	>100000
425 (2903)	40000	90000	>100000	>100000	>100000
450 (3103)	>100000	100000	>100000	>100000	>100000
475 (3275)	>100000	>100000	>100000	>100000	>100000
500 (3447)	>100000	>100000	>100000	>100000	>100000
Slab Thickness = 10" (254 mm)	**Mean Distance from Edge of Pavement**				
MR psi (kPa)	0" (0 mm)	6" (152 mm)	12" (305 mm)	18" (457 mm)	24" (610 mm)
300 (2068)	<1000	<1000	<1000	1000	4000
325 (2241)	1000	4000	8000	18000	58000
350 (2413)	16000	37000	70000	>100,000	>100,000
375 (2585)	>100,000	>100,000	>100,000	>100,000	>100,000
400 (2558)	>100,000	>100,000	>100,000	>100,000	>100,000
425 (2903)	>100,000	>100,000	>100,000	>100,000	>100,000
450 (3103)	>100,000	>100,000	>100,000	>100,000	>100,000
475 (3275)	>100,000	>100,000	>100,000	>100,000	>100,000
500 (3447)	>100,000	>100,000	>100,000	>100,000	>100,000

Table 18 Current LTPP ESALs for Two California Locations.

Location	ESALs/yr.	ESALs/day
San Diego	2.5 million	6,800
San Joaquin	5.4 million	14,700

The vehicle's mean distance from the edge has an effect on the required opening strength. As the mean distance from the edge increases, lower opening concrete strength can be used or a larger number of ESALs can be applied for a given concrete strength.

There are several strategies to facilitate opening the concrete pavement to traffic. A concrete strength of 2070 kPa (300 psi) or less may be used if trucks are restricted from the rehabilitated lane or freeway. Trucks would have to use alternate routes for several days until the concrete gained the minimum strength to limit any fatigue damage. However, the difficulty of enforcing the alternate routes, levying fines if a truck does travel over the newly constructed pavement, and the difficulty of restricting trucks from highly traveled corridors most likely make this strategy unreasonable. Furthermore, it would take only several trucks, especially if they are overloaded, to greatly reduce the service life of the pavement.

Another strategy would be to place edge barriers such as cones approximately 600 to 900 mm from the slab edge to reduce the maximum stress in the concrete. This strategy may be more feasible because it does not restrict the corridor or newly constructed lane to truck traffic.

The opening strength analyses have shown that there are many combinations of thickness, traffic, distance from edge of the pavement, and concrete strength that may work for a given pavement location. These analyses did not include temperature-induced stresses that may increase or decrease the total bending stresses in the concrete pavement and may cause premature failure under certain conditions.

7.0 CONSTRUCTION PRODUCTIVITY ISSUES

As stated in Section 2.1 of this report, one of the objectives of LLPRS is to have sufficient production to rehabilitate or reconstruct about 6 lane-kilometers within a construction window of 67 hours (10 a.m. Friday to 5 a.m. Monday). Many of the long-life pavement rehabilitation projects will occur on freeways in the Los Angeles area. The paving productivity of 6 lane-kilometers in a 67 hour window will be the major bottleneck to overcome if all LLPRS-Rigid objectives are going to be met. Several contractors from midwestern states have stated this paving productivity has been achieved before. However, it is unlikely any contractor in the state of California has done this type of paving productivity, especially in an urban environment.

To determine the bottlenecks in concrete paving, the following areas of a concrete paving operation will be briefly discussed: batch plant, supply of concrete to job site, transit time, paver type, pavement geometry and material constraints, time of paving, and condition of existing pavement.

7.1 Batch Plant

Improvements in concrete batch plant design have increased their productivity to 800 cubic yards per hour for a twin drum automated plant. An 800 cu yd./hr (612 m^3/hr.) plant can produce enough material to pave 2,160 lane-feet (658 lane-meters) of a 10-inch (25.4 cm) concrete pavement per hour. The LLPRS goal of 6 lane-km per weekend is easily achievable and would take approximately 10 hours to complete. The American Concrete Pavement Association states that the average contractor productivity has doubled over the past 30 years to 300 cu yd./hr (230 m^3/hr.). At 300 cu yd./hr. (230 m^3/hr.), only 810 lane-feet (247 lane-meters) can be constructed per hour. At this productivity level, constructing the 6 lane-kilometers to meet the LLPRS-Rigid objective would take 25 hours. Current concrete pavement construction should therefore not be bottlenecked by batch plant productivity.

7.2 Concrete Paver

The next major piece of equipment to analyze is the concrete paver. The most productive paver is the slip-former because it saves the step of setting up side forms. The average maximum paver speed is about 480 feet per hour (146 m/hr.), as long as sufficient concrete is being supplied. The paver can go at this speed no matter the pavement width, as long as the batch plant productivity is higher than paver productivity. At this rate, the paver is traveling much slower than the 2,160 lane-feet/hr. (658 m./hr.) made possible by the 800 cu yd./hr. (612 m^3/hr.) batch plant output. For a 10-inch concrete pavement requiring one lane rehabilitation, a 180 cu yd./hr. (138 m^3/hr.) batch plant is all that would be required.

In order to increase paver productivity, multiple lanes would have to be reconstructed simultaneously. Table 19 below lists the number of lane-feet that could be completed if more than one lane were reconstructed with a 10-inch (25.4 cm) slab.

Table 19 Construction Times for Multi-Lane Construction Scenarios, 10-inch (25.4 cm) Slab Thickness.

Number of Lanes	Production lane-feet/hr. (lane-meters/hr.)	Required Plant Production cu yd./hr. (m^3/hr.)	Number of Hours to finish 6 lane-km
1	480 (146)	180 (138)	41
2	480 (146)	360 (275)	21
3	480 (146)	540 (413)	14
4	480 (146)	720 (551)	11

Besides adding another paver to the job site, the only way to increase productivity is to increase the number of lanes reconstructed simultaneously. Reconstructing one lane is not very efficient, and employment of two pavers would still take 21 hours to pave 6 lane-km, as shown in Table 19. A recent presentation by a continuous reinforcement concrete pavement (CRCP) industry group stated the record paving day for Texas was 5,200 cubic yards (3976 m^3) placed. This translates into 4.3 lane-km of 25.4 cm concrete slabs. Additionally, this paving was not done in a high traffic volume area in Texas. A former contractor present at the CRCP meeting worked with several California contractors to schedule a weekend CRCP paving job and found they could expect to pave about 2,500 cubic yards (1911 m^3), or 2 lane-kilometers per weekend. The production for continuously reinforced concrete pavements would be expected to be slower than jointed plain concrete due to the high amount of steel placement.

Another contractor stated that the largest paving operations in California occur at airports where twin drum plants and end dumps can be used, and the paving widths are larger. The contractor said that one of the largest airport pours in California was 5,000 cubic yards (3823 m^3) in one day. This volume of concrete translates into 4 lane-km per day for a 25.4 cm concrete slab.

7.3 Concrete Supply Trucks

Another bottleneck in the production can be the supply of the concrete from the batch plant to the paver. Ready mix trucks can legally carry 7 cubic yards (5.4 m^3) per trip. If a 400 cu yd./hr. (306 m^3/hr.) operation is required, then a rate of 57 trucks per hour will be required to supply the job.

One problem with ready mix trucks is that it takes some time to charge them with concrete and it takes a longer time to unload the concrete. This makes them inferior to end dump trucks when high speed is desired. End dump trucks can be efficiently charged and dumped in front of the paver. They also can hold about 12 cubic yards (9.2 m^3) per load. If a 400 cu yd./hr.

(306 m^3/hr.) operation is required, then a rate of 34 end dump trucks per hour will be required to supply the job.

The transit time from the batch plant to the paver may also slow down production. If the batch plant is close to the job site, then production should not be affected. However, if trucks must go some distance to reach the job site, especially if through heavy traffic, then productivity must decrease. As the paving job continues, the batch plant is automatically going to be farther away from the job site unless multiple batch plants are used.

7.4 Construction Materials Limitations

7.4.1 Dowels

Some construction materials, such as dowels, can slow down paving. If dowel baskets are used, then using end dump trucks right in front of the paver becomes difficult. Dowel baskets require a placer in front of the paver to distribute the concrete uniformly. Placers slow down productivity because the concrete end dump trucks cannot unload as quickly. The use of automated dowel bar inserters on the paver is one way to eliminate dowel baskets and maintain a high productivity.

7.4.2 Existing Pavement Structure

The productivity considerations discussed in Sections 7.0-7.4.1 assume that the existing pavement structure, cement treated base, subbase, and subgrade, are in satisfactory condition and will not need to be replaced. If any of these components need to be replaced, then the overall productivity in a weekend in terms of lane-kilometers has to decrease. Non-destructive testing is recommended prior to construction to identify areas that will require replacement.

7.4.3 Type of Paving Material

The type of material to be used in concrete paving has not yet been addressed. The productivity rates discussed in Sections 7.0-7.4.1 assume that the type of paving material would not affect productivity. However, the use of fast setting hydraulic cement concrete may reduce productivity because it is a new product with which contractors do not have much experience. This lack of experience with FSHCC for contractors around California will result in a lower productivity when compared to conventional PCC pavement construction until contractors become more familiar with the material.

Other issues, which have not been fully explored, are the distance and time FSHCC be transported without agitation if end dump trucks are used to increase productivity speeds. In addition, there are still unanswered questions about the buildup of FSHCC in trucks and on the paver, which must be cleaned out frequently, and the ability of the construction crew to finish the pavement behind the paver for an extended work period. All these considerations regarding FSHCC construction will have either no effect or some negative effect on productivity.

7.5 Other Productivity Issues

Another factor, which will slow down overall productivity, is weekend or nighttime-only construction versus continuous construction. Weekend or nighttime-only construction was chosen to minimize delays in traffic during peak times. However, overall construction productivity is reduced if continuous construction is not utilized because of the huge drop in productivity that occurs with mobilization and demobilization.

7.6 Sensitivity of Productivity to Concrete Opening Strength Specification

The proposed four-hour specification of modulus of rupture greater than 2760 kPa (400 psi) for opening the concrete pavement to traffic appears to be reasonable for most pavement structures and locations as shown in Tables 16 and 17. A concern arises as to how much productivity the contractor is losing if the specification were to have an 8- or 12-hour strength requirement. If the pavement construction were a continuous process (7 days per week), then production would not be affected by any strength requirement. However if weekend construction were being performed, then the productivity in terms of lane-kilometers completed in a weekend may be reduced with a more gradual strength gain specification. This means the contractor has fewer hours to pave because he must allow the concrete sufficient time to gain a minimum opening strength.

Table 20 shows the length of 254 mm (10-inch) concrete pavement that can be constructed in various paving times. Table 20 also shows the reduction in productivity in terms of lane-kilometers if paving time is reduced. There are considerable reductions in paved length especially at low paver productivity (100CY/HR). However, these low rates are not acceptable for the LLPRS objectives. A minimum of 400 cu. yd./hr paver productivity must be achieved if 6 to 7 lane-kilometers are going to be paved in a weekend. If the 4 hour specification were relaxed to an 8 hour specification at 400 cu. yd./HR, then there would be a 16 percent reduction in paved length (7.9 to 6.6 lane-km). If the 4 hour specification was relaxed to a 12 hour specification then the paving length would be reduced by 33 percent.

This analysis assumes the contractor will stop paving four hours before opening the entire project back to traffic. However, detailed scheduling of a project needs to be completed in order to determine if four hours is enough time for a contractor to clean up and demobilize from a site. If four hours is not sufficient time for the contractor to clean up and demobilize, then the strength

Table 20 Length of 254 mm Concrete Pavement That Can Be Constructed in Various Paving Times.

Length of Paving Time (hours)	Lane-km constructed at 100 cu yd./hr. (77 m^3/hr.)	Lane-km constructed at 200 cu yd./hr. (153 m^3/hr.)	Lane-km constructed at 400 cu yd./hr. (306 m^3/hr.)	Lane-km constructed at 800 cu yd./hr. (612 m^3/hr.)
12	1.0	2.0	4.0	7.9
16	1.3	2.6	5.3	10.5
20	1.6	3.3	6.6	13.2
24	2.0	4.0	7.9	15.8

specification of four hours is not on the critical path. Table 20 also indicates that a contractor will probably have to pave for at least 20 to 24 hours on a weekend to complete 6 lane-kilometers of pavement. The feasibility of paving continuously for 20 to 24 hours over a weekend has to be explored given that it will take some time to remove the existing pavement structure and prepare the pavement substructure for concrete.

8.0 OTHER STATE DOT USE OF HIGH EARLY STRENGTH CONCRETE

Other states and agencies have addressed the need for high early strength concrete and high concrete pavement productivity. The term associated with these two criteria is “fast-track” concrete pavements. Fast-tracking began in the late 1980s and early 1990s on airport and highway pavements. The majority of fast-track projects have required traffic opening concrete strengths to be met in less than 24 hours. The materials used to meet fast-track strength requirements have been Type I, II, and III Portland cements and certain proprietary cements. One state required the Type III cement to achieve a minimum cube strength at 12 hours of 9.0 MPa in order to be considered for fast-track projects. (31) Type I and II Portland cements had to use chemical admixtures to meet early and long-term strength requirements. Some fast-track projects have used fly ash as a supplement to the hydraulic cement to provide long term strength, increase workability, and finishability of the concrete mix, and to decrease permeability of the hardened concrete. Table 15 lists several projects and specifications in which fast-track concrete practices were employed. The majority of the fast-track projects in Table 15 used Type III cement (9 out of 14). Only one project in Table 15 had a 4-hour strength specification and it used a special blended cement. Several projects achieved strengths of 2.4 MPa (350 psi) in less than 10 hours with Type III cements.

The majority of fast-track projects used curing compound to limit evaporation of mix water and reflect solar radiation to prevent excess heat build up in the concrete surface. On fast-track projects, insulating blankets have been used to aid in the early strength gain especially at lower air temperatures (< 27 C). Construction data from fast-track projects, such as batch plant and paver productivity and length of project, have not yet been fully researched. Further literature reviews to determine the construction requirements in each fast-track project and their corresponding concrete specifications are planned.

9.0 SUMMARY

This report summarizes several design and construction issues that need to be addressed for rigid longer-life pavements. Listed below is a summary of each topic discussed in this report followed by recommendations.

- **Existing Pavement Design Methods**. Several empirical and mechanistic-empirical design procedures for rigid pavements were reviewed and some benefits and drawbacks of each design procedure were identified. The empirically-based AASHTO procedure should be used cautiously in new designs. The PCA design guide is mechanistic-based, but does not allow for analysis of temperature curling, widened lanes, long slab lengths, etc. A mechanistic-empirical design guide similar to the Illinois Department of Transportation concrete design guide has the most potential to analyze many pavement features, environmental conditions, and any axle load and configuration.
- **ESAL versus Load Spectra.** Caltrans ESALs were compared with AASHTO ESALs and were found to be similar with errors increasing with axle type (least error for single axle, increasing with tandem and tridem, respectively). A pavement thickness design comparison between ESALs and load spectra for Southern California traffic volumes and loads was completed. Whether ESALs or load spectra analysis was used, there was no difference in pavement thickness. For the current axle loads and configurations, ESALs and load spectra give the same thickness design, based on fatigue.
- **Longitudinal Cracking.** The appearance of longitudinal cracking on many California rigid pavements was discussed. A brief literature review and simple finite

element analyses were conducted to determine what causes this type of cracking. The literature review and analysis found that longitudinal cracking may occur from fatigue damage at the transverse joint and/or incompressibles entering the joint causing high compressive stresses in the slab.

- **Opening Concrete Strength.** A literature review found that a minimum of 300 psi (2,068 kPa) flexural strength is required to open to truck traffic. This strength requirement increases if the slab thickness decreases, the subgrade stiffness decreases, or the number of ESALs increases. A brief fatigue analysis with the ILLICON program found similar results to the preceding study and the need to determine concrete opening strength depending on the project constraints (materials, traffic, pavement structure). The ILLICON analysis showed that moving the truck wheels away from the edge would reduce the required opening strength.
- **Concrete Construction Productivity.** Each aspect of concrete pavement construction was evaluated in terms of paving lane productivity. Batch plant productivity was determined not to be a limiting factor in pavement construction. The concrete paver was found to be most productive when constructing multiple lanes simultaneously. Ready mix trucks were found to be less productive than end dump trucks due to their slow offload speed and smaller concrete capacity. Other issues that may slow down paving productivity are the use of dowels baskets, removal and replacement of CTB, and use of FSHCC. In order to meet the LLPRS objectives of paving 6 lane-kilometers per weekend, concrete productivity rates higher than existing PCC productivity rates in California will have to be achieved. The time required to pave the 6 lane-kilometers of concrete pavement and the time to clean up

and demobilize the construction site may be the critical scheduling path for construction rather than the required opening concrete strength for traffic.

10.0 RECOMMENDATIONS

Based on the findings of this report and summary in Section 9.0, the following are preliminary recommendations concerning design and construction issues for LLPRS-rigid projects:

1. **Existing Pavement Design Methodologies.** Mechanistic-based design procedures, such as the Illinois Department of Transportation guide, should continue to be used to evaluate the proposed longer life pavement features. Although mechanistic-empirical methodologies have limitations, they are more powerful in their ability to analyze a large number of pavement features that may have never been constructed before. Due to the empirical nature and limitations of procedures like AASHTO and the PCA, caution should be exercised when using these guides given the possibility for erroneous results.

2. **ESALs versus Load Spectra.** For individual projects, load spectra analysis should be used to quantify the effect traffic has on the fatigue resistance of concrete pavement. ESALs should still be used to describe the composite effect that traffic and the environment has on the overall pavement performance.

3. **Joint Sealants.** Given that longitudinal cracking may be caused by incompressibles locking the joint, it may be advantageous to seal all joints as a precautionary measure and an added insurance.

4. **Concrete Construction Productivity.** Further analyses must be completed to determine what construction processes are on the critical path.

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