MATERIALS ENGINEERING

Review Of Minnesota's
CONCRETE
PAVEMENT DESIGN
TO: R. J. McDonald  
Deputy Commissioner

FROM: P. C. Hughes, Geotechnical Engineer  
Office of Materials Engineering

DATE: March 29, 1985

PHONE: 296-3110

SUBJECT: "A Review of Mn/DOT's Concrete Pavement Design Methods"  
Report by The Concrete Pavement Design Task Force

Attached is the final report entitled "A Review of Mn/DOT's Concrete Pavement Design Methods." The report by the Concrete Design Task Force is in response to your charge of reviewing Mn/DOT's present process in pavement design, including the AASHTO Standards, and any other factors which influence the end product recommendation.

This report is a summary of the review which took place. Based on the review and subsequent discussions, the Task Force determined which factors in the present design process remained valid or required modification, and if new factors needed to be considered. Included in the report are the conclusions and recommendations.

During the review of Mn/DOT's procedures, and in discussions with the Federal Highway Administration, Portland Cement Association, the Minnesota Concrete Pavers Association, and Mn/DOT Staff, it became obvious to the Task Force that Minnesota is a leader in concrete pavement design. The importance of much of which has become standard practice during the last few years in Minnesota is just now starting to be recognized by others throughout the United States. I think Leo Warren should be commended for his forward looking approach to concrete pavement design.

As indicated above, the Task Force's findings and recommendations have been discussed with others outside of Mn/DOT, including the Portland Cement Association and the Minnesota Concrete Pavers' Association. As a result of these discussions, some slight modifications were made in the Task Force's recommendations and those changes are included in this final report.

Upon your approval of this report, the next action to be taken will be implementation of the Task Force's recommendations. Unless otherwise notified, I will assume responsibility for this implementation step.

Finally, I would like to thank you for the opportunity to serve as chairman of this Task Force. The Task Force itself should be congratulated for its diligent efforts in arriving at recommendations which, when fully implemented, will result in significantly improved Minnesota concrete pavement design procedures.

The Recommendations Contained in this Report are Approved For Implementation

R. J. McDonald, Deputy Commissioner
Review of Minnesota’s

CONCRETE PAVEMENT DESIGN

by the

Minnesota Department of Transportation

CONCRETE DESIGN TASK FORCE

MARCH 4, 1985
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I. FOREWORD

This report is the result of a review of the concrete pavement design procedures currently in use by the Minnesota Department of Transportation. The charge given to the Design Review Task Force was to review Mn/DOT's present process in pavement design, including the AASHTO Standards, along with factors that influence the end product recommendation.

This report is a summary of the review which took place. Based on the review and subsequent discussions the Task Force determined which factors in the present pavement design process remained valid, or required modification and if new factors needed to be considered. Included in this report are the conclusions and recommendations of this indepth review.
II. INTRODUCTION

A rigid pavement structure normally consists of two layers, designated as the pavement slab and the subbase course. The design procedure includes the determination of the thickness of the portland cement concrete slab and the design of joints and steel reinforcement.

Design considerations that are essential to satisfactory performance and long life of a rigid pavement are (1) reasonably uniform support for the pavement; (2) the elimination of pumping by use of treated or untreated base, edge drains, open graded bases, etc; (3) adequate joint design; and (4) a thickness that will keep the load stress within safe limits. The overall objective in concrete pavement design is to have a pavement which provides satisfactory performance over its design life (35 years in Minnesota) at the least annual cost.

The objective of this study was to review Mn/DOT's current rigid pavement design procedures, taking into account concrete pavement performance and current state of the art information. The Task Force was also asked to make conclusions as well as recommendations relating to the various factors involved in concrete pavement design. This study included a review of past and present rigid pavement design procedures, field reviews of inplace concrete performance, discussions with Mn/DOT District personnel, input from Mn/DOT Central Office staff and discussions with representatives of the Federal Highway Administration, Portland Cement Association, and the Minnesota Concrete Pavers Association.
III. BACKGROUND

The Department has used a formal rigid pavement design procedure since the early 1920's. Beginning in the mid 1960's the Department adopted a slightly modified Portland Cement Association Design Procedure (PCA Method) which was based on results of the AASHO Test Road at Ottawa, Illinois. Starting in 1982 Mn/DOT began a transition from the PCA Method to the AASHTO Interim Guide Procedures as revised in 1981 (see Appendix A). During the last year all concrete pavement designs have followed procedures outlined in the Mn/DOT Road Design Manual (see Appendix B).

The basic difference between the old design method used by Mn/DOT (modified PCA Method) and the AASHTO procedures currently in use is the thickness obtained at various levels of traffic and loads. Shown below in Figure 1 is a graph relating thickness to loading.

Figure 1. Relationship Between Concrete Pavement Thickness and Traffic Loadings.

As is evident, the AASHTO Interim Guide procedures result in less thick pavements at lower loading levels than does the modified PCA Method. This difference has become an issue of discussion and often time a difference of opinion within the Operations and Technical Services Divisions. In addition to the above, other factors have also received much attention lately including such things as, (1) placement of steel reinforcement; (2) use of widened concrete section (24 feet to 27 feet); (3) use of dowels; (4) urban versus rural design; (5) R-Values versus K-Values; (6) traffic projections; (7) modules of rupture; (8) safety factors; etc.

As a result of the above, R. J. McDonald, Deputy Commissioner, decided to form a special Task Force to review and make recommendations concerning Mn/DOT's rigid pavement design process. The Concrete Pavement Design Task Force members were:

*P. C. Hughes, Geotechnical Engineer
W. N. Yoerg, District Engineer, Brainerd
R. A. Adolfson, Design Standards Engineer
D. J. Flemming, Assistant District Engineer, Golden Valley
L. P. Warren, Chief, Concrete Engineering Section
G. R. Cochran, Subgrade and Base Design Engineer
T. J. Fudaly, Area Engineer, FHWA

*Chairman
The charge to the Task Force was to review Mn/DOT's present process in pavement design, including the AASHTO Standards, along with factors that influence the end product recommendation and to make conclusions and recommendations. The Task Force received its charge and began deliberations in March of 1984. The first item of business was to review the charge given to the Task Force and to assemble and evaluate all concerns related to rigid pavement design. Following this, meetings were held to become educated about the AASHTO interim guide and the modified PCA Method design procedures. An effort was made to become familiar with the factors related to the design of rigid pavements and the sensitivity each has on concrete pavement performance.

A field trip was made to take a look at inplace pavement performance and to discuss concrete pavement design with personnel in Districts 5, 6 and 7. Based on the field trip areas of concern were identified for further investigation. These included traffic projections, modulus of rupture, dowel bars, thickness of pavement, serviceability factors, materials properties, drainage, steel reinforcement, costs of extra inch of concrete, dowel bars in ramps, tied third lane of multi-lane roadways, widened pavement sections, selection of R and K Values, and correction of traffic from 20 to 35 years and the affect of frozen subgrade.

The next step taken was a meeting with Al Pint, Traffic Forecasting Engineer of the Office of Transportation Information and Support, who made a presentation on past, present and future forecasting plans.

Next came a meeting with Leon Noel, who is with the FHWA in Washington, D.C., Roman Dankbar from the FHWA Regional Office in Chicago, and John Bowers from the FHWA in St. Paul. A general discussion was held on all the areas of concern identified and listed previously for rigid pavements as well as a state-of-the art presentation by Mr. Noel. Mr. Noel's field trip report is shown in Appendix C. One of the interesting highlights of Mr. Noel's presentation was the fact that there is a NCHRP project underway which is dealing with a re-do of the AASHTO interim guide. It appears that this study is addressing most if not all of the concerns that Minnesota has with its design procedures. Upon hearing that there was a NCHRP Study underway which appeared virtually the same as the charge given to the Task Force, the Task Force questioned whether or not it should continue with its deliberations or wait until the results of the NCHRP study are known. It was decided that it would be best to continue with the Task Force's charge at this time, issue a final report as soon as possible, and then modify that report, if necessary, based on the results of the NCHRP study when they are known.

During the time that all of the above meetings were taking place, Leo Warren, Concrete Engineer, was keeping the Task Force members aware of what his office was doing in concrete design as well as any other new developments. Of significant note was the development of a Concrete Pavement Evaluation System (COPES) which is also a NCHRP project under the guidance of Dr. Michael Darter of the Department of Civil Engineering at the University of Illinois. A rough draft of Dr. Darter's final report was given to each Task Force member for review. In addition, three reports prepared by the Concrete Technologies Laboratories for Mn/DOT, were distributed. These reports dealt with the effect of frozen support, tridem axles, and tied concrete shoulders on the performance of concrete pavements.
Upon completion of the review and research phase, the Task Force held several meetings to evaluate the input. A rough draft report summarizing the conclusions and recommendations on Mn/DOT's Concrete Pavement Design Procedures was then written. Subsequently the rough draft was reviewed by and discussed with Mn/DOT upper staff, the Portland Cement Association (see Appendix D) and the Minnesota Concrete Pavers Association (See Appendix E). This final report, then, is the result of the foregoing efforts.
IV. DISCUSSION

The following is a discussion of those factors in Mn/DOT's concrete pavement design that were listed by others and/or identified by the Task Force as areas of concern.

Concrete Modulus of Rupture
Data from concrete beam tests and compression tests were reviewed in regard to a determination of an appropriate design working stress value. The design working stress is defined as the expected flexural strength of the concrete divided by a constant $C$. The $C$ term is a safety factor. Mn/DOT currently uses 1.33 as the safety factor.

The present Mn/DOT test procedure for concrete beams requires that the test beams be broke at the center point. The AASHTO test T97-76 (1982) procedure specifies a third point loading. The Portland Cement Association has compared the test results between center point and third point loading and found an approximate 75 psi difference between the two test procedures. If it is assumed that the length of the test beam, moment of inertia, distance to the extreme fibers and total load are all constant for both test procedures, the only variable in flexural strength is the difference in bending moment. The recommended 75 psi difference for normal paving concrete appears to be reasonable from a theoretical as well as empirical procedure as established by the Portland Cement Association. The committee, therefore, recommends that the 75 psi reduction be applied to the present Mn/DOT test data in lieu of purchasing new test equipment and changing to the AASHTO test procedure.

There is currently only a limited amount of 28 day beam test data available because most test beams have been broken at 7 days and 14 days under present Mn/DOT procedures. The available 28 day data show that flexural strengths of 700 to 800 psi can be expected. Using an average of 750 psi and subtracting 75 psi to get the equivalent third point loading flexural strength and dividing by a safety factor of 1.33 gives a working stress of 507 psi. An attempt was made to determine a design working stress from the available 7 day and 14 day data. In order to utilize this data, however, the strength gain of the concrete must be estimated from strength gain curves. This strength gain depends upon temperature, humidity, and the concrete mix design. Good correlation was not found between the available test data for 28 day beam breaks and interpolated data from strength gain charts for 7 day and 14 day breaks.

The committee, therefore, recommends that: (1) a working stress of 500 psi be used as the design value until further 28 day test data indicates a new design value and (2) that future projects require beam tests at 28 days to develop a reliable source of data.

K-Value versus R-Value
Concrete pavement design requires consideration of the modulus of subgrade soil reaction (K-value). This is determined from field plate bearing tests rarely run by any agency today. Mn/DOT's current bituminous design procedure utilizes a laboratory soil test, the Hveem Stabilometer (R-Value). During Investigation 183 fractional plate bearing tests were run at about 50 field test sections. In addition R-values were determined at those same sites. Professor Eugene Skok of the University of Minnesota has analyzed
Figure 2. Relationship Between K-Value and R-Value in Minnesota.

Date: 8/8/84

Scp = K-Value

Correlation r = .798

Best fit Eqn.: K = 1.17 + 63.004 SCP (R)
the plate bearing data and converted it into K-values\(^2\). Harvey Allen, Office of Research and Development, Mn/DOT has, through regression analyses, developed a relationship between the K-values and R-values for these same sections (see Figure 2)\(^3\). This relationship \((-1.17 + 63 \sqrt{R})\) has a standard error of 94 and an \(r^2\) of 0.64. Because the data is all from Minnesota sites and Mn/DOT test procedures, the relationship should be superior for Mn/DOT use to that of other published relationships which typically show neither the data points nor the degree of conservatism applied and in addition are from other localities. As to the degree of conservatism, the committee decided to use the same design R-value in the relationship as used in bituminous design (mean minus one standard deviation).

In summary the committee recommends deriving a K-Value for concrete design by using a design R-Value in the relationship \(K = -1.17 + 63 \sqrt{R}\).

**Effect of Frozen Subgrade**

Currently in Minnesota, the traffic consideration for designing concrete pavements is Concrete Equivalent Single Axle Loads (CESAL). The CESAL's are estimated for 20 years and then adjusted to the 35 year design life by multiplying by a conversion factor of 1.25. This assumes a uniform growth rate factor and takes into account the increase in the subgrade support that occurs during the frozen time of the year.

According to Ball\(^4\), et al., of the Concrete Technology Laboratories, the "... damaging effect of a given single axle or tandem axle load applied in the winter is about 1/7 to 1/9 of that for the same axle load applied during the fall." Ball recommends that "... only 1/7 of the equivalent 18-Kip single-axle loads applied during the winter months need to be considered for thickness design.

Using Ball's recommendation and assuming a frozen period of 4 months, the multiplying factor is determined as follows:

\[
\text{CESAL}_{35} \text{ Adjustment Factor} = \left[ \frac{35 \text{ year}}{20 \text{ year}} \right] \left[ \frac{8 \text{ Mo.}}{12 \text{ Mo.}} + \left( \frac{4 \text{ Mo.}}{12 \text{ Mo.}} \right) \left( \frac{1}{7} \right) \right]
\]

\[
\text{CESAL}_{35} \text{ Adjustment Factor} = 1.25
\]

Hence, the procedure to multiply the 20 year CESAL estimate by 1.25 to obtain the 35 year design life CESALs.

However, Ball also states that "... the current AASHTO design procedure already has built into it the effect of frozen support..."

He further states, that the results of their study can be used in the AASHTO design procedure "... if the difference in severity and duration of winter conditions between Ottawa, Illinois, and the State of Minnesota can be established".

The method used for determining the conversion factor, in effect, assumes that Minnesota has four more months of frozen support than does Ottawa. The literature does not support that assumption.
According to the AASHO Road Test Report 75, there was an average of approximately 3 1/3 months of frozen support during the three winters tested (1958-1961). Studies done on 11 highway projects in southern Minnesota in 1957 by John S. Braun6 for his MSCE thesis showed an average support period of approximately 4 1/3 months - only one month longer than the road test.

Also of interest is the fact that at the Road Test, the period from the start of thawing until total thaw ranged from 1 to 3 weeks in duration. In Minnesota, thawing took about 5 weeks. Therefore, it could be assumed that the Minnesota sections were in a weakened state for a minimum of 2 weeks longer than the road test. However, deflection tests run by Scrivner7 near Ottawa, Illinois and in the Rochester and Duluth, Minnesota areas in 1967 indicate a similar duration of the weakened state. Tests run on six sections near Ottawa showed an average "critical period" of 45.2 days ("critical period" is defined as the period of rapid strength loss plus the period of rapid strength gain as measured by deflection testing.) Tests on six sections near Rochester showed a "critical period" of 40.5 days. Near Duluth the critical period was 43.0 days.

While the data comparing the effect of climate on road strength in Minnesota and Ottawa, Illinois is limited, the following can be concluded.

1. Highways in Minnesota are in a frozen support state for approximately one month longer than highways near Ottawa, Illinois.

2. Highways in Minnesota are in the "critical period" for about the same time duration as highways near Ottawa.

3. The equivalent 18K single axle loads used in the AASHTO design procedure for concrete pavements in Minnesota (CESAL) should be adjusted to reflect the one month difference in the duration of the time of frozen support between Ottawa and Minnesota. The CESALs used in the design procedure for that month should be 1/7 of the actual projection.

Based upon the above conclusions the Task Force recommends that CESALS be projected for the 35 year design life and then adjusted to account for frozen support. If the Traffic Forecasting Unit is not able to project for 35 years the 20 year estimate should be doubled and that value used as the 35 year CESAL. This assumes an average growth rate of 4 percent between the years of 20 to 35.

The adjustment is accomplished by multiplying the 35 year CESAL by an Adjustment Factor of 0.93.

\[
\text{CESAL}_{35} \text{ Adjustment Factor} = \frac{11 \text{ Months} + \frac{1 \text{ Month}}{12 \text{ Months}}}{12 \text{ Months}} \times \frac{1}{7}
\]

\[
\text{CESAL}_{35} \text{ Adjustment Factor} = 0.93
\]
Traffic Projections

During the initial Committee discussions, traffic forecasts were identified as a major concern both by the Districts and the Concrete Office. Both strongly feel that our present forecasting procedures do not ensure "realistic" traffic values for the design period. This feeling was reinforced during a field trip to the Rochester District during which the Committee observed a number of roads which had exceeded the projected 20th year daily Tractor-Semi Trailer (TST) traffic after only 10 years. In another case, on a section of recently reconstructed pavement in the Mankato District, district staff indicated that the TST traffic is already higher than expected due to traffic being drawn to it (being a new pavement) which was not considered in the traffic projection.

Due to the problems identified with traffic projections, a meeting was set up with Al Pint of the Traffic Forecasting Unit of the Office of Transportation Information and Support. During the meeting Al made a presentation on past and present forecasting procedures and future forecasting plans.

Prior to 1976, all traffic forecasts were done in the Central Office. During 1976 and 1977 courses were presented in the districts, except 5 and 9, so they could do their own forecasts. Districts 5 and 9 utilized the Metro Council to obtain forecasts.

It appears that part of the problem in traffic forecasting is that past prediction procedures lacked the sophistication to come up with reliable projections. A few inquiries were made into how the rural districts did traffic forecasts. Typically only a few such forecasts are required in a district each year. The assignment is given to a technician who reviews the procedures developed by the Office of Transportation Information and Support and then proceeds to make the forecasts. Because of the relatively fast changes which have occurred in growth rates, infrequency in making forecasts, and general lack of forecasting expertise, the results are less than desirable in many cases. The results, depending on the wishes of the districts, are reviewed or approved in C.O. or used directly in the surface type determination. At the present time, Al Pint is requesting that all forecasts be sent to him for review, but that may not be occurring.

Al also covered a number of other items in his presentation which he felt would help to improve forecasting results. He indicated that

-- data from weigh-in-motion sites will be used in projects in the future

-- the Traffic Forecasting Unit will be publishing an updated procedures manual by November 1, 1984

-- the Metro Council is no longer making forecasts for Districts 5 and 9 (these districts will also do their own)

-- origin-destination studies which haven't been done for some time, will be initiated again
the load factors for 5 axle semis (ESAL's) are being modified upwards

consultant has developed a statistical probability model to determine confidence levels of traffic forecasts. The model is being tested and if appropriate will be included in the procedures manual. The confidence level used will be based on the level of importance of the roadway.

Based on the discussions of the Committee regarding the traffic forecasting concerns, the following recommendations are made:

1. The districts should continue to make traffic forecasts.

2. Training courses should be given to the district forecasters by the Traffic Forecasting Unit, on an annual basis, to update them on any changes in the procedures which have occurred as well as to increase their expertise.

3. The districts should make a greater effort to communicate with the Traffic Forecasting Unit when making forecasts.

4. Before being used in surface type determination all traffic forecasts should be approved by the Traffic Forecasting Unit of the Office of Transportation Information and Support.

5. The forecasting procedures should include a determination of the traffic drawn to a road due to its reconstruction.

6. A monitoring procedure should be used to compare forecasts to actual traffic growth on roadways of various classifications to determine if the inplace traffic projection procedures are giving reliable results.

Dowel Bars

Mn/DOT currently uses the 20th year daily count (design lane) TST's of 150+ as the value in determining whether or not to require dowels. It is apparent, based on our field review in D-5, D-6 and D-7, that the 150+ TST dowel criteria is too high and soil type and aggregate type must be considered along with TST in evaluating when to use dowels. Presently, there are not enough undoweled pavements entered into Mn/DOT's COPES' data to determine if trends exist which identify a TST, soil type, aggregate or combination of these at which a pavement would probably fault.

Until more conclusive COPES's data become available on undoweled pavements, the committee, based on engineering judgement, recommends the following criteria for dowel bar placement in ramps, loops and mainlines based on the 20th year design lane daily count of TST's:

1. 120 TST's for a granular subgrade when high quality concrete aggregate is used

2. 100 TST's for a plastic subgrade when high quality concrete aggregate is used or for a granular subgrade when carbonate aggregate is used

3. 80 TST's for a plastic subgrade when carbonate aggregate is used.
The above criteria would be subject to change as more COPES' data becomes available.

In addition to TST's, soil type, and aggregate type, the Districts should identify any typical situations which may affect pavement performance such as a gravel pit, landfill, dump site or known future development along or near the corridor.

**Tied Third Lane**

Mn/DOT presently does not tie the third lane to the other lanes as a multi-lane highway. According to Leo Warren, Mn/DOT has no significant problems with an untied third lane but there are problems with keyways and spalls along a longitudinal joint of the lane that is tied into the initial two lane construction.

A field review to confirm Leo's statements was conducted. Where a third lane was tied, and a keyway used, there was bad spalling problems along the longitudinal joint. Where the third lane was untied and located on the outside, faulting and severe lane separation were evident. Where the third lane was untied and located on the inside, some lane separation was noted but no faulting was evident. Because of the above experience, Mn/DOT will always place the untied third lane on the inside. If properly maintained and sealed, a slight lane separation should be no problem. Although it is an FHWA position that the third lane should be tied, at this time it would not appear to be a cost-effective solution in Minnesota.

It is recommended that the untied third lane be placed on the inside and pavements with an untied inside lane be periodically monitored for faulting and separation to determine if there is a potential maintenance problem. If a problem is becoming evident, Mn/DOT should reevaluate the need for tying the third lane, on multi-lane highways.

**Drainage**

Water can be a very troublesome problem with highways. The water may be associated either with the subgrade or the area immediately below the pavement structure. For this discussion, it is assumed that the subgrade water problem has been taken care of by proper design and grading - proper ditch depths, subcuts, good grading soils and compaction, etc. For this discussion, water problems immediately below the pavement itself will be the main concern. The source of this water is primarily from above, that is, it comes through defects in the pavement itself and the shoulders. In addition, some water may come up through capillary action from below.

The water below a concrete pavement slab can reduce the support value under the slab, create non-uniform support for the slab, and cause pumping of fines from below the slab at the pavement joints. All of these problems can reduce the life of the concrete pavement by causing cracking of the concrete at places other than at the planned joint locations and faulting of the joints.
To some extent, water can be prevented from entering from above by:

1. Proper selection of panel lengths so that the joints do not open excessively.
2. Proper sealing and maintenance of the joints.
3. Surfacing and maintenance of the adjacent shoulders.

However, with time, water eventually does get below the slab. This infiltration of water is most detrimental when the subgrade is constructed of plastic soils, the pavement joints are non-doweled, and high traffic volumes with large numbers of trucks present.

Methods for reducing the effects of water infiltration may include:

1. Providing granular bases, either, open graded to provide drainage or dense graded to form a water-shedding layer.
2. Providing treated bases or subgrades such as cement or bituminous treated bases, lime or cement treated subgrades, etc., to shed water.
3. Providing edge drains to remove the water.
4. Design of the slab thickness to keep stresses within safe limits.
5. Providing doweled joints.
6. Providing a widened pavement section.

From the Committee's field trip, the most predominate effect of water below the slab has been faulting of non-doweled pavement joints. This has been the result of pumping of fines under loadings at the joint and resulting loss of support for the slab at the leaving side of the joint. The proposed criteria for using dowels (see Page 11) should help to alleviate this problem.

Although the solution to this problem is beyond the scope of this Committee, it is recommended that Mn/DOT:

1. Continue to provide aggregate bases under concrete pavements as prescribed in Chapter 7 of the Road Design Manual.
2. Conduct research on the use of drained granular backfilled subcuts of varying depth and material, edge drains, and open graded bases.
3. Develop and publish in the Road Design Manual a criteria for the use of edge drains, drained granular backfilled subcuts, and open graded base, based on factors such as:
   a) Type of subgrade soils
   b) Number of trucks
   c) Depth of water table and ditches
   d) Location of grade; low points, supers, flat grades, etc.
   e) Benefit/cost ratios.
Serviceability Factor

One of the charges to the Committee was to evaluate whether or not the concrete pavement design procedure should address urban and rural designs differently. What this is actually asking is should there be different serviceability factors used in the design formula relative to the importance of the road. At the present time, a serviceability factor of 2.5 is used for all designs, urban or rural.

To do the evaluation one first has to determine the typical mode of failure of concrete pavements. In addition there is a need to determine what a change in serviceability factor has on concrete pavement design.

Based on the Committee deliberations and field trips it appears evident that the typical mode of failure of a concrete pavement is joint deterioration rather than pavement fatigue due to loads. The joint deterioration can be manifested in a number of ways including such things as faulting, raveling, blowups, D-cracking (aggregate problem), etc. Rarely, if ever have concrete pavements in Minnesota failed due to inadequate thickness.

To look at the affect of change in serviceability factor on concrete design two concrete designs were compared. Shown below in Table I are the factors used in the designs and the resulting pavement thicknesses when those factors are used in the formula.

| TABLE I - Variation in Concrete Pavement Thickness Due to Change in Serviceability Factor |
|---------------------------------|---------|---------|
| DESIGN FACTOR                  | DESIGN 1 | DESIGN 2 |
| Terminal Serviceability        | 2.5      | 3.0      |
| Subgrade Reaction              | 198      | 198      |
| Soil Support                   | 4.42     | 4.42     |
| Soil Characteristic (R)        | 10       | 10       |
| Rigid                          |          |          |
| Load Transfer                  | 3.2      | 3.2      |
| Modulus of Rupture             | 525      | 525      |
| Modulus of Elasticity          | 4,200,000| 4,200,000|
| N18                            | 5,000,000| 5,000,000|
| Designed Depth                 | 8.435    | 8.782    |

As you can see, the only modification in the input into the design formula was in the serviceability factor which changed from 2.5 to 3.0. The change in serviceability factor resulted only in a thickness variation in the pavement, in this case, a design pavement thickness difference of only 1/3 of an inch which is probably not significant. The change in serviceability factor has no affect on joint design.

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Based on the above discussion, the Committee recommends that a serviceability factor of 2.5 continue to be used in the design formula for both urban and rural roadways. Furthermore, since the general mode of failure of concrete pavements is joint deterioration the Committee recommends extraordinary means be used to improve concrete joint performance on the more important roadways. Extraordinary means would include such things as edge drains, open graded bases, granular backfill in sub-cuts, etc.

Reduction in Traffic for Widened Section or Tied Shoulder
The committee reviewed the reports entitled "Effect of Concrete Shoulders on Concrete Pavement Performance"8 and "Evaluation of Concrete Pavement with Lane Widening, Tied Concrete Shoulders and Thickened Pavement"9 as they related to the AASHTO design procedures. The reports conclude that the damaging effect of a single axle or tandem axle load applied to a pavement with a tied shoulder is only approximately 1/2 that for the same axle loads applied to a pavement without a tied shoulder. In addition, a lane widening of only 16" was found to be as effective as a tied concrete shoulder.

Therefore, the committee recommends that only 1/2 of the equivalent axle loads be considered in the AASHTO design procedure for the thickness design for pavements with tied concrete shoulders or for widened sections.

Widened Concrete Sections
The report entitled "Evaluation of Concrete Pavement with Lane Widening, Tied Concrete Shoulders and Thickened Pavement"9 substantiates that a widened section with the 27'-0" width is structurally adequate. The committee recommends that only a widened section of at least 27'-0" be considered for future concrete pavement unless tied concrete shoulders are used. The widened section has two advantages over the normal 24'-0" sections, namely: (1) the pavement has substantially lower edge stresses and (2) runoff water is drained further away from the traffic wheel path. Water accumulation in the joint between a concrete pavement and a bituminous shoulder has been recognized as a major factor in poor pavement performance. Concrete shoulders and lane widening have the potential for partially alleviating this drainage problem.

In addition to the widened section, based on information presented in the report "Vehicle Shoulder Encroachment and Lateral Placement Study"10 and past performance, the Task Force recommends that stripping continue to be done at 12 feet from centerline and that 3.0 long foot rumble strips be placed from the inside of the stripe into the widened section. The stripping and rumble strips placement recommended above should also be used with tied concrete shoulders. The use of rumble strips significantly lessens the number of vehicles encroaching out onto the widened section or the tied concrete shoulder thereby minimizing edge stresses.

Minimum Thickness for Concrete Pavements
Recent changes in Mn/DOT's design procedures have resulted in an increased number of concrete pavement thickness designs thinner than 7 inches. This has caused concern in various District Headquarters throughout the State. The Task Force feels these concerns are justified for over the years very few pavements have been constructed thinner than 7" on State Highways. Previously Mn/DOT used the PCA design method which is based on Tractor-Semi-Trailer counts (20 one-way T-S-T would require 7" concrete on heavy soils). The AASHTO Interim Design Equation, (now in use) for concrete pavement thickness equates all loads to equivalent 18 kip single axle loads. It is possible to have a $CESAL_{35} = 850,000$ giving a 6.5" thick concrete pavement.
The following items are matters of concern when constructing thin pavements (less than 7 inches) for a 35 year design life:

1. The standard factor of safety of 1.33 (see page 6) may not adequately cover situations where a few extremely heavy loads may cause more damage than the large number of low 18 kip equivalences (See Federal Register/Vol. 47, No. 206).

2. Relatively thin pavements deflect more under loads and these pavements will be subject to a greater potential for faulting. To overcome this potential, thin pavements offer only limited interfacings of aggregates for interlock and minimal depth for cover over dowel bars.

3. The relatively thin pavements also curl and warp to a greater magnitude and as such can offer rough ride at high speed which may or may not be transient in nature.

Therefore, the Task Force recommends that a 7" minimum thickness for concrete pavement on our State Highways be used. This minimum should not apply to county and city roads.

Material Properties

D-cracking has had an enormous impact on concrete performance in this state. Presumably the problem is now solved although the committee did hear that at least one nearby state is requiring higher type aggregate on interstate pavements than Mn/DOT would. In addition, some quasi-D-cracking phenomena was noted on T.H. 169 for which no ready explanation is available.

Based on pavements inspected during committee field trips, pavements residing on granular soils seem to perform better than those on plastic soils. Since the original designs presumably took into account the difference in soil support and since concrete has traditionally been insensitive to subgrade support the real key to the difference in performance may lie in subgrade drainage.

Unfortunately the AASHTO Interim Guide does not appear to delve into the materials area in any great depth. Since it is an area which has significant impact, as seen with D-cracking, on the performance of concrete pavement the Task Force recommends this important area receive additional study and research by the Concrete Section and the Research Office; especially the D-cracking and quasi-D-cracking phenomena, and drainage. COPES data as it becomes available - particularly with an eye toward the effects of drainage should be of great benefit in such an effort.

Reinforced Pavement

According to the AASHTO Interim Guide for Design of Pavement Structures 1972, Chapter 111 Revised, 1981; "The purpose of distributed steel reinforcement in reinforced concrete pavement is not to prevent cracking, but rather to hold tightly closed any cracks that may form, thus maintaining the pavement as an integral structural unit". To provide for an orderly arrangement of the cracking that occurs, contraction joints are provided in the pavement slab. If the joints are properly designed and spaced, cracking of the panel outside of the joint will be held to a minimum.
The Wire Reinforcement Institute's booklet, "Jointed Concrete Pavements Reinforced with Welded Wire Fabric", has a section on slab lengths. It states: "A number of factors affect the choice of slab length. Plain slabs must not crack nor lose load transfer at joints. Slabs reinforced with steel may crack, but the joints must not open excessively to destroy the seals or overtax the load transfer devices at the joints." For plain pavements, when no reinforcing is used, the panel lengths must be of such length that curling stresses are low and the joint opening preserves aggregate interlock. A rule of thumb for determining panel lengths to reduce cracking and not requiring reinforcement is:

1. For Plastic Subgrades
   Slab thickness (in inches) x 2 = panel length in feet.
   Example: 8 x 2 = 16 ft panel length.

2. For Granular Subgrades
   Slab thickness (in inches) x 2 + 20% to 25% (of T x 2) = panel length
   Example: 8 x 2 + (.25 x 16) = 20 ft panel length

From the above, the 27 ft panel length falls into the range of panel lengths requiring reinforcing. The 20 ft panel length falls on the edge of the range requiring reinforcing; needing reinforcing on plastic subgrades and questionable on granular subgrades. From the Committee's field trip, experience with existing concrete pavements tends to support the rule of thumb, 20 ft panels on granular subgrades tend to perform adequately, while 20 ft panels on plastic soils tend to show mid-panel cracking and faulting of joints.

Figure 3 provides annual costs per mile for 20 ft long panels, both 24 ft and 27 ft wide, non-reinforced with doweled joints and 27 ft long panels, both 24 ft and 27 ft wide, reinforced. State-wide average costs were used for these estimates. Figure 4 is a plot of thickness vs. annual cost per mile for both panel lengths. Annual costs were determined using the cost of new construction plus the cost of one joint resealing (during 17 1/2 year) as calculated during the pavement selection cost estimate.

Panel lengths of 20 ft have 264 joints per mile, 27 ft panels have 196 joints per mile. This is a difference of 68 joints per mile or a difference of 34.7%. Annual costs for various thickness of concrete, 27 ft wide, for 20 ft non-reinforced doweled panels and 27 ft reinforced panels indicate little difference in the annual costs per mile for the 20 ft panel vs. 27 ft panel. A reinforced 20 ft panel pavement would cost more than a reinforced 27 ft panel pavement.

The Committee can find no real benefits for the use of either the non-reinforced 20 ft panels with doweled joints or the reinforced 20 ft panels when compared to the 27 ft reinforced panel. Therefore, it is recommended that the 27 ft reinforced plan continue to be Mn/DOT's standard. In addition the standard of skewing joints should be continued.
Figure 3. Annual Costs Per Mile for 20 and 27 Foot Long Panel Pavements.

24' Wide, 20' Panels, Non-reinforced, Doweled

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Annual Cost/Mile</th>
<th>Difference/in</th>
<th>% Increase</th>
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<td>$9,262</td>
<td>$1151</td>
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<td>9.3</td>
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<tr>
<td>9&quot;</td>
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27' Wide, 20' Panels, Non-reinforced, Doweled

<table>
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<th>Thickness</th>
<th>Annual Cost/Mile</th>
<th>Difference/in</th>
<th>% Increase</th>
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</thead>
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<td>9.6</td>
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<td>9&quot;</td>
<td>12,501</td>
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24' Wide, 27' Panels, Reinforced

<table>
<thead>
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<th>Difference/in</th>
<th>% Increase</th>
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</thead>
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<td></td>
</tr>
</tbody>
</table>

27' Wide, 27' Panels, Reinforced

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Annual Cost/Mile</th>
<th>Difference/in</th>
<th>% Increase</th>
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<tr>
<td>9&quot;</td>
<td>12,834</td>
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</tr>
</tbody>
</table>

-19-
FIGURE 4. Relationship Between Thickness and Annual Cost Per Mile For 20 and 27 Foot Long Panel Pavements

ANNUAL COST PER MILE

CONCRETE THICKNESS

27' panels 27' wide
20' panels 27' wide
27' panels 24' wide
20' panels 24' wide
Non-Reinforced Pavement
Mn/DOT's present policy is to use non-reinforced pavement when dowel bars are not required. In order to provide for load transfer in non-reinforced pavements the panel lengths are shortened so that joints do not open excessively. This allows for load transfer to be maintained through aggregate interlock. A 15 foot effective joint spacing (13 ft.-16 ft.-14 ft.-17 ft.) is used for non-reinforced pavement in Minnesota.

Based on information presented to the Task Force by the Concrete Engineering Section and performance of inplace pavements identified during the field trip, the Task Force recommends that the 15 foot effective joint spacing continue to be used as the non-reinforced pavement design. Again, the practice of skewing joints should be continued.

Safety Factors
Considering our 35 year concrete pavement design life it would seem imprudent not to insert some margin for error in our design assumptions. These margins for error take the form of safety factors or conservatism when selecting values for design assumptions. In our concrete design they occur in the model equation in the following terms:

1. Modulus of Rupture - The third point, 28 day beam break test results are divided by 1.33 which is our recommended safety factor. By doing this we essentially design the concrete pavement for in excess of 2.80 times the forecasted traffic. This amounts to a reliability or confidence level in excess of 93%.

2. K Value - Testing of the projects soils are performed which establishes R values. From this set of R values a design R is chosen; this design R is the mean minus one standard deviation. This design R is then converted to the appropriate "K" value.

3. Widened concrete pavements - the adoption of widened concrete pavements permits, according to our studies, reduced design concrete thicknesses by 1.5 to 3.2 inches. We feel a conservative approach is that recommended in report "Effect of Concrete Shoulders on Concrete Pavement Performance"8 which calls for reducing the forecasted CESAL's by one half. This reduces the thickness only about one inch.

4. Traffic forecasting - A statistical probability model to establish confidence levels on traffic forecasts is being developed. It should be available for use this fall. This will result in higher forecasts on more important routes.

AASHTO is currently developing a revised pavement design guide in which the safety factor will be placed in the traffic forecasts. This document is scheduled for publication in April 1986. Our design assumptions should be reevaluated at that time.

Communication
Based on input to and observations by the Task Force, it appears that much of the concern which has arisen over Mn/DOT's present concrete pavement design procedures and the factors involved is the result of inadequate information being passed back and forth between the concerned parties. What the Task Force found in its review would suggest that the Concrete Engineering Section should be commended for their forward looking approach.
to concrete pavement design. Much of which the Task Force recommends for changes in the AASHTO Interim Guide Design Procedures and adoption by Mn/DOT have already been suggested by Leo Warren and some are in the process of being implemented. In the NCHRP Study which will result in a re-do of the AASHTO Interim Guide many of the changes which will result will follow closely work done by the Concrete Office over the last few years. Unfortunately this information has not gotten around the Department and misunderstandings as to the approach the Concrete Office has taken toward concrete pavement design have occurred.

In order to improve on the present situation the Task Force recommends that the Concrete Engineering Section make additional efforts to openly discuss what is happening with concrete pavement design with those, both within and outside of Mn/DOT concerned with pavement design. This should be done to educate all those concerned with pavement design and to open up lines of communication. In addition, similar to the situation which exists with the bituminous pavement design, District Materials Engineers should become very familiar with the concrete pavement design so that they can actually come up with the designs using the project variables as well as being able to comfortably explain how the design procedures work. This could be accomplished by cross training between the Concrete Office and the Districts.
V. RECOMMENDATIONS

Based on its findings during the review of Mn/DOT's concrete pavement design procedures the Task Force recommends the following:

1. The Concrete Engineering Section should be commended for their forward looking approach to concrete pavement design which is being used as a model by others throughout the United States.

2. A design working stress of 500 psi should be used as the design value until further 28 day test data justifies a new design value.

3. Future projects should require beam tests at 28 days to develop a reliable source of data. In lieu of purchasing new test equipment and changing to the AASHTO test procedure (third point loading), a 75 psi reduction should be applied to the 28 day test data from the present Mn/DOT beam break procedures (midpoint loading). As present test equipment becomes unusable, new equipment, meeting the AASHTO standards should be purchased to replace the old equipment.

4. A K-Value for concrete design should be derived using the relationship \( K = -1.17 + 63 R \)-Value where R-Value is the same as that used in the bituminous pavement design (the mean minus one standard deviation).

5. Concrete Equivalent Single Axle Loads (CESALS) for concrete design should be determined by projecting the CESALS for the 35 year design life and then adjusting for frozen support. Until the Traffic Forecasting Unit is able to project for 35 years, the 20 year estimate should be doubled and used as the 35 year CESAL. The adjustment for frozen support can be accomplished by multiplying the 35 year CESALS by an adjustment factor of 0.93.

6. The districts should continue to make traffic forecasts.

7. Training courses should be given to the district forecasters by the Traffic Forecasting Unit, on an annual basis, to update them on any changes in the procedures which have occurred as well as to increase their expertise.

8. The districts should make a greater effort to communicate with the Traffic Forecasting Unit when making forecasts.

9. Before being used in surface type determination all traffic forecasts should be approved by the Traffic Forecasting Unit of the Office of Transportation Information and Support.

10. The forecasting procedures should include a determination of the traffic drawn to a road due to its reconstruction.

11. A monitoring procedure should be used to compare forecasts to actual traffic growth on roadways of various classifications to determine if the inplace traffic projection procedures are giving reliable results.
12. For the present time, dowel bar placement on mainlines, ramps and loops should meet the following criteria using the 20th year design lane daily count of TST's.

a) 120 TST's for a granular subgrade when high quality concrete aggregate is used.

b) 100 TST's for a plastic subgrade when a high quality concrete aggregate is used or for a granular subgrade when carbonate aggregate is used.

c) 80 TST's for a plastic subgrade when carbonate aggregate is used.

The above criteria would be subject to change as more COPES' data becomes available.

13. Until evidence can be gathered to show there is a need to tie the third lane on multi-lane roadways Mn/DOT should continue with its present policy of not tying the third lane and placing the untied lane on the inside of the roadway.

14. Although poor drainage results in non-uniform support of the concrete slab the solution to the problem is beyond the scope of the charge given to the Task Force. However, the Task Force does recommend the following:

a) Continue to provide aggregate bases under concrete pavements as prescribed in Chapter 7 of the Road Design Manual (copy attached in Appendix B).

b) Conduct research on the use of drained granular backfilled subcuts of varying depth and material, edge drains, and open graded bases.

c) Develop and publish in the Road Design Manual a criteria for the use of edge drains, drained granular backfilled subcuts, and open graded bases, based on factors such as:

1) Type of subgrade soils
2) Number of trucks
3) Depth of water table and ditches
4) Location of grade; low points, supers, flat grades, etc.
5) Benefit/cost ratios.

15. A serviceability factor of 2.5 should continue to be used in the design formula for both urban and rural. Extraordinary means (i.e., edge drains) should be used to improve concrete joint performance on more important roadways.

16. Only a widened section of at least 27' - 0" should be considered for future concrete pavement constructed in Minnesota unless tied concrete shoulders are used. Striping should continue at 12 ft. from centerline. Rumble strips should be included to minimize vehicle encroachments onto the widened sections or the tied concrete shoulders. If further study shows 27' - 0" is insufficient to handle encroachments on to the shoulder the section should be revised.
17. When using the widened section (27'-0") or tied concrete shoulders, for the purpose of determining pavement thickness only 1/2 of the design CESALS should be used in the AASHTO design procedure.

18. For State Highways a 7" minimum thickness should be used for concrete pavements. This minimum should not apply to county and city roads.

19. As the AASHTO Interim Guide does not delve into the materials area to any great depth, the Concrete Engineering Section and Research Office should continue and even expand its efforts to evaluate material properties particularly with reference to D-cracking, the quasi-D-cracking phenomenon and drainage. COPE's data should be very useful in this area.

20. Mn/DOT should continue to use 27 ft. long reinforced concrete pavement panels as its standard reinforced design and 15 ft. effective (13'-16'-14'-17') panel lengths as its standard non-reinforced design.

21. Mn/DOT should continue its present standards of skewing joints.

22. For the present a safety factor of 1.33 should continue to be used with the modulus of rupture in the design formula. As the AASHTO Interim Guide is revised and the safety factor is moved out of the modulus of rupture and into the traffic forecast area Mn/DOT's design assumptions should be re-evaluated.

23. The Concrete Engineering Section should make additional effort to openly discuss what is happening with concrete pavement design with those, both within and outside Mn/DOT, concerned with pavement design.

24. The Concrete Engineering Section should provide more training which would enable District Materials Engineers to design concrete pavements as they can now do with bituminous pavements.
CONCLUSION

Based on the recommendations contained in this report and if those recommendations are implemented Mn/DOT will be taking a slightly more conservative approach to concrete pavement design. Dowel bars will be used more often, a wider concrete section will be used, and a slightly thicker pavement at higher traffic loadings will result.

Shown in the below Table is a comparison of the thickness of concrete pavements resulting from the various design methods used by Mn/DOT since the mid 1960's. Traffic and soil strengths are shown as the variables. In comparing the last two columns it is evident that the Task Force's recommended design will result in slightly thicker pavements at the higher traffic loadings.

Comparison of Pavement Thicknesses Resulting from Various Mn/DOT Design Methods.

<table>
<thead>
<tr>
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<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
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<td>Low</td>
<td>8.0</td>
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</tr>
<tr>
<td>Low</td>
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<td>6.5</td>
<td>6.0</td>
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<tr>
<td>Medium</td>
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<td>10.5</td>
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<td>10.5</td>
</tr>
</tbody>
</table>
VI - APPENDIX
APPENDIX A

Chapter III—GUIDE FOR THE DESIGN OF RIGID PAVEMENT
Revised, 1981

3.1 — INTRODUCTION

A rigid pavement structure may consist of two layers, designated as the pavement slab and the subbase course. When the roadbed soils are of subbase quality, the subbase course is often omitted. The design procedure includes the determination of the thickness of the portland cement concrete pavement slab, and the design of joints and of steel reinforcement. Also included are recommendations as to the treatment of subbase soils and the type and thickness of subbase required to provide an adequate uniform support for the pavement slab.

3.2 — LIMITATIONS

It is essential that the user of the design procedure in the guide understand its limitations, which are:

1. The design chart scales for working stress \( f_1 \) in concrete, modulus of elasticity of concrete and modulus of subgrade reaction \( k \) are derived from the Spangler modifications of the Westergaard theory of stress distribution in rigid slabs. Further research will be required to fully establish the applicability of the Spangler equation.

2. There is no adjustment in the AASHO Road Test rigid pavement equation for an environmental or regional factor, because it was not possible to measure the effect of variations in climatic conditions over the two-year life of the pavement at the Road Test site. Further research will be required to establish such a factor.

3. There is no provision for the inclusion of an internal drainage system. Such a system aids in reducing the water-related deterioration experienced by many pavements after 15 or 20 years, such deterioration typically progressing exponentially with time. Recent experience has shown that high pore pressures develop under heavy axle loads resulting in scouring of fines from the subbase. Experimental information suggests the use of high permeability layers coupled with pipe edge drains to provide for rapid dissipation of pore pressures and removal of free water without damage to the pavement system.

4. Although the traffic repetitions used in the development of the design relationship were experienced over only a two-year period, the traffic analysis period that must be selected for design is usually considerably longer than two years. The traffic analysis period should not be confused with pavement life, which is affected by other factors in addition to traffic.

5. Two major over-all assumptions have been made in the development of these design procedures, as follows:

(a) That the adequacy of the design will be established by soils and materials surveys and laboratory studies.

(b) That the design strengths assumed for the subgrade and pavement structure will be achieved through proper construction methods.

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3.3 — MATERIALS PROPERTIES AND SPECIFICATIONS

3.3.1 — Subbases

The subbase of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the subgrade and the rigid slab for the following purposes:

1. To provide uniform, stable, and permanent support.
2. To increase the modulus of subgrade reaction (k).
3. To minimize the damaging effects of frost action.
4. To prevent pumping of fine-grained soils at joints, cracks, and edges of the rigid slab.
5. To reduce cracking and faulting.
6. To provide a working platform for construction equipment.

If the subgrade soils are of a quality equal to a subbase, no additional subbase layer is needed.

A number of different types of subbases have been used successfully. These include graded granular materials and materials stabilized with suitable admixtures. Table III-1 sets up specifications for six subbase types that could be used. Other specifications designed to utilize local materials, that give satisfactory results, may be used. Type A, B, C, D, or E subbase material (Table III-1) could be used in the top 4 inches of the subbase layer. Type F subbase material may be used below the top 4 inches. When Type A subbase is used, precautions may be necessary to prevent the intrusion of underlying fine-grained soils. In areas subject to frost action, Types A, B, and F subbase materials should be modified to provide a minimum amount of fines.

The prevention of water accumulations on or in subgrade soils or subbases is essential if satisfactory performance of the pavement structure is to be attained. Drainage should be provided whenever Type A subbase is used, and trench sections should be avoided. The performance of Types B, C, D, E, and F subbases will be improved by drainage. It is recommended that the subbase layer be carried 1 to 3 feet outside the edge of the pavement structure.

Problems with the erosion of subbase material under the pavement slab at joints and at the pavement edge have led some designers to the use of a lean concrete or "econcrete" subbase. This type of subbase should provide better support than conventionally stabilized subbases. However, standard analyses for design of the pavement structure when such subbases are used have not been sufficiently developed and tested. Pending further research, the design of the pavement structure using lean concrete or econcrete subbases should be based on the experience and recommendations of those designers who have such experience.

3.3.2 — Pavement Slab

The basic materials in the pavement slab are portland cement concrete, reinforcing steel, load transfer devices and joint sealing materials. Quality control on the project to see that the materials conform to AASHTO's or the agency's specifications will minimize distress resulting from distortion or disintegration.
### Table III-1
**Types of Subbase**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Type A (Open Graded)</th>
<th>Type B (Dense Graded)</th>
<th>Type C (Cement Treated)</th>
<th>Type D (Lime Treated)</th>
<th>Type E (Bituminous Treated)</th>
<th>Type F (Granular)</th>
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<td>100</td>
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<tr>
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<td>65-100</td>
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<td>5-12</td>
<td>5-20</td>
<td></td>
<td></td>
<td>0-15</td>
</tr>
</tbody>
</table>

(The minus No. 200 material should be held to a practical minimum)

**Compressive Strength:**
- psi at 28 days: 400-750 to 100

**Stability:**
- Hveme Stabilometer: 20 min.
- Hubbard-Field: 1000 min.

**Soil Constants:**
- Liquid Limit: 25 max.
- Plasticity Index**: N.P. 6 max. 10 max. 6 max.
- Type E: 6 max.*** 6 max.

* To be determined by complete laboratory analysis, taking into consideration the ability of the stabilized mixture to resist under-slab erosion.

** As performed on samples prepared in accordance with AASHTO Designation T 87.

*** These values apply to the mineral aggregate prior to mixing with the stabilizing agent.

#### 3.3.2.1 Portland Cement Concrete
- The mix design and material specifications for the concrete should be in accordance with, or equivalent to, the requirements of AASHTO “Guide Specifications for Highway Construction” and “Standard Specifications for Transportation Materials”. Under the given conditions of a specific project, the minimum cement factor should be determined on the basis of laboratory tests and prior experience as to strength and durability.

Air-entrained concrete should be used whenever it is found necessary to provide resistance to surface deterioration from freezing and thawing or from salt or to improve the workability of the mix. Other types of cement should be considered if materials in the area result in adverse reactions.

#### 3.3.2.2 Reinforcing Steel
- The reinforcing steel used in the slab should have deformations or deformation properties adequate to develop the working stresses in the steel. The steel mats required may be assembled on the project or prefabricated, or steel placement may be accomplished by mechanical methods. There are numerous grades available for the requirements of the proposed use. In some cases, very high-strength steels are required, whereas in other cases the bending properties must be considered.
3.3.2.3 – Joint-Sealing Materials – Two basic types of sealants are presently used for sealing joints:

1. Liquid sealants – These include a wide variety of materials of three types: asphalt, hot-poured rubber, and polymers. These materials are placed in the joint in a liquid form and allowed to set. When using liquid sealants, care should be taken to provide the proper shape factor for the movement expected.

2. Preformed elastomeric seals – These are extruded neoprene seals having internal webs that exert an outward force against the joint face. The size and installation width depend on the amount of movement expected at the joint.

3.3.2.4 – Shoulders – Shoulders have been traditionally constructed of flexible base with an asphalt surfacing or of a stabilized base with an asphalt surfacing. The dissimilarity of the main lane and shoulder and the encroachment of heavy wheel loads onto the shoulder has resulted in joint problems between the travel lanes and the shoulder. Research has shown that strengthening of the shoulder and special sealants have helped to alleviate this problem. The use of a tied concrete shoulder or widening with concrete at least three feet or, more have given satisfactory results. Thickening the outside edge of the travel lane or the use of a monolithic curb (where appropriate) also strengthens the pavement edge and reduces the shoulder-joint problem.

3.4 – DEVELOPMENT OF THE RIGID PAVEMENT DESIGN PROCEDURE

The procedure presented herein as a guide for the design of rigid pavement structures is based on data developed by the AASHO Road Test, supplemented and modified by theoretical analysis. The design procedure is presented in this guide in the form of a nomograph (Figure III-1) for ease in solution of the design equation. The equations represented by this nomograph were developed on the basis of the following assumptions:

1. That the basic equations developed from the AASHO Road Test are a valid representation of the relationship between loss in serviceability, traffic, and pavement thickness. In these equations, loss in serviceability is expressed in terms of reduction of serviceability index, traffic is converted to equivalent 18-kip single-axle load applications, and pavement slab thickness is expressed directly in inches of portland cement concrete.

2. That the basic equations developed from the AASHO Road Test for a single type of roadbed soil, may be extended to apply to any roadbed soil by means of a scale of modulus of subgrade reaction (k) developed for this purpose.

3. That the basic equations developed from the AASHO Road Test for repeated applications of uniform traffic loads may be extended to apply to mixed traffic by conversion to equivalent 18-kip single-axle loads.

4. That the basic equations developed from the AASHO Road Test for accelerated applications of traffic during the two-year test period may be extended to apply to applications or traffic during an extended period of time.

5. That the basic equations developed from the AASHO Road Test for a portland cement concrete pavement slab of one type and level of physical properties, may be extended to apply to pavement slabs of other types and levels of physical properties by means of a scale of working stress in concrete (f,) and modulus of elasticity of the concrete developed for this purpose.
6. That uniform and high-quality construction will be obtained, particularly with respect to density, gradation, and quality of materials, and smoothness of the pavement surface, both transversely and longitudinally.

The development of the design equations from the basic AASHO Road Test equations and the assumptions outlined above is presented in detail in Section D.1 of Appendix D. Use of this equation in the design of rigid pavement structures requires an evaluation of the following for the expected conditions: terminal serviceability index \( p_1 \), equivalent 18-kip single-axle loads, modulus of subgrade reaction \( k \), working stress in the concrete \( f_w \), and modulus of elasticity of the concrete.

3.4.1 - Modulus of Subgrade Reaction \( k \)

The basic equation developed from the results of the AASHO Road Test is valid for only one value of modulus of subgrade reaction, that representing the roadbed soils at the Road Test site. In order to make the design procedure applicable to other roadbed soils, it was necessary to develop a scale for modulus of subgrade reaction to represent the variety of soils that would be encountered at other sites.

Westergaard’s modulus of subgrade reaction \( k \) (referred to as “gross k” in AASHO Road Test reports), is used in this guide. It represents the load in pounds per square inch on a loaded area divided by the deflection in inches of that loaded area. The scales for \( k \) included in the design charts are correlated with values obtained by plate loading tests performed in accordance with AASHTO Designation T222 using a 30-inch diameter plate. The “k” value may be estimated on the basis of previous experience, or by correlation with other tests.

3.4.2 - Concrete Properties

The average flexural strength for the concrete on the AASHO Road Test was about 690 psi at 28 days. In order to make the design procedure applicable to concrete of other flexural strengths, it was necessary to develop a scale for this purpose. The modulus of rupture \( S_c \) at 28-days as determined by the test procedure specified in AASHTO Designation T-97, using third-point loading, is the basis for determining concrete flexural strengths. If test data are normally available for other than 28-day strengths, the expected 28-day strengths should be obtained from a time-strength correlation and the extrapolated or interpolated values used. The scale included in the design charts is for working stress \( f_w \) in the concrete. A working stress should be based on the expected flexural strength of the concrete. Recommended working stress can be calculated by the following equation:

\[
f_w = \frac{S_c}{C}
\]

Where \( S_c = 28 \text{ day flexural strength of third point loading, psi} \)  
\( C = \text{constant to determine design working stress.} \)

The C term is a safety factor. The higher the value, the higher the confidence in an adequate design. For freeways and other high volume facilities where closing of a lane for possible rehabilitation will cause the projected traffic to exceed capacity of a probable detour, a higher value for C of up to 2.00 is recommended. Also, for any projected traffic conditions which indicate a relatively low number of total 18 kip equivalencies during the design life
(<1,000,000 for example) but contain above average wheel loads (>10,000#), the factor of safety should be adjusted to prevent damage by one or a few heavy loads. Another case where a different value of C may be required is where local experience has shown that environmental or other factors require that a thicker or thinner pavement be provided. For most conditions a value of 1.33 is recommended. Generally, the safety factor of 2.00 will add between one and two inches to the slab thickness.

The modulus of elasticity may be determined from static compression tests on cylinders (ASTM C469). An average value is acceptable for use in computations. A value of 4,200,000 psi was used as representative of AASHTO conditions for developing the design equations.

The design chart was developed from equations based on Road Test conditions which had only dowel-jointed pavements. The thickness of other types of concrete pavement should be based on local experience or sound engineering analysis. In particular, continuously reinforced concrete pavement should have the same thickness as jointed pavement unless local experience shows that thinner slabs will perform satisfactorily. Where the design analysis indicates a pavement slab thickness of less than 8 inches, careful consideration must be given to environmental conditions, and to the construction problems that may be encountered, before such a lesser thickness is used. Thicknesses less than 8 inches may be satisfactory on minor highway systems carrying light traffic.

3.4.3 - Joints and Load Transfer

3.4.3.1 - Expansion Joints - The primary function of an expansion joint is to prevent the development of damaging compressive stresses due to volume changes in the pavement slab, and to prevent excessive pressures being transmitted to adjacent structures.

In general it is considered that expansion joints are not necessary for rigid pavement, except adjacent to structures. At these locations expansion joints may be used when protected with satisfactory load transfer devices and suitable preformed joint fillers. A 3/4- to 1-inch width should be used. Consideration might be given to the use of suitable terminal anchorage devices in combination with expansion joints.

Where it is necessary to provide for more than 1 inch of expansion space, a series of 1-inch joints may be installed at intervals of approximately 20 feet. Other types of joints that provide adequate control may also be used.

Unusual conditions, such as cold weather construction or the use of materials with a high coefficient of expansion, may require special consideration. Suitable modifications of the previous recommendations may be made based on a study of the particular situation.

3.4.3.2 - Contraction Joints - The purpose of contraction joints is to provide for an orderly arrangement of the cracking that occurs. If the joints are properly designed and spaced, a minimum of cracking outside the joint would be expected.

Contraction joints may be sawed in the hardened concrete or formed by plastic inserts if performance indicates they are satisfactory. The depth of joint
should not be less than 1/4 of the thickness of the pavement slab. The design of the joint should be related to the expected joint opening and the elongation of the joint filler used. Adequate load transfer through mechanical means should be provided at all joints. Aggregate interlock may be adequate for load transfer under some conditions.

A 4-foot to 5-foot skew on a 24-foot width pavement will result in only one wheel crossing the joint at any one time. This skew results in better load transfer and improved riding quality across the joint.

Random spaced joints have been used to prevent rhythmic or resonant reaction to a moving vehicle. Skewed joints have successfully been coupled with randomly spaced joints.

3.4.3.3 - Longitudinal Joints - Longitudinal joints are used to prevent the formation of irregular longitudinal cracks, and may be keyed butt joints or mechanically formed or sawed grooves. Adjacent lanes should be kept from separating and faulting by steel tie bars or connectors spaced in accordance with Table III-2. The depth of formed or sawed grooves should not be less than 1/4, the thickness of the pavement slab.

3.4.3.4 - Load-Transfer Devices - Mechanical load-transfer devices should possess the following attributes:

1. They should be simple in design, practical to install, and permit complete encasement by the concrete.
2. They should properly distribute the load stresses without overstressing the concrete at its contact with the device.
3. They should offer little restraint to longitudinal movement of the joint at any time.
4. They should be mechanically stable under the wheelload weights and frequencies that will prevail.
5. They should be resistant to corrosion when used in those geographic locations where corrosive elements are a problem.

A commonly used load-transfer device is the plain round steel dowel conforming to AASHTO Designation M31-Grade 60 or higher. The minimum design requirements for round dowels should be as follows:

<table>
<thead>
<tr>
<th>Pavement Thickness in.</th>
<th>Dowel Diameter in.</th>
<th>Dowel Length in.</th>
<th>Dowel Spacing in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>3/16</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>9</td>
<td>1 1/4</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>10</td>
<td>1 1/4</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>11</td>
<td>1 1/4</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>12</td>
<td>1 1/4</td>
<td>18</td>
<td>12</td>
</tr>
</tbody>
</table>

Other mechanical devices that have proven satisfactory in field installation may be used.
3.4.3.5 - Tie Bars — Tie bars, either deformed steel bars or connectors, are designed to hold the faces of abutting slabs in firm contact. Tie bars are designed to withstand the maximum tensile forces required to overcome subgrade drag. They are not designed to act as load-transfer devices.

Deformed bars should be fabricated from billet or axle steel of Grade 40 conforming to AASHTO M-31 or M-53. The bar sizes, lengths and spacings for different pavement conditions are given in Table III-2.

Other approved connectors may also be used. The tensile strength of such connectors should be equal to that of the deformed bar that would be required. The spacing of these connectors should conform to the requirements given for deformed bars.

Consideration should be given to the use of corrosion-resistant materials where salts are to be applied to the surface of the pavement.

3.4.4 - Reinforcement Criteria

The purpose of distributed steel reinforcement in reinforced concrete pavement is not to prevent cracking, but rather to hold tightly closed any cracks that may form, thus maintaining the pavement as an integral structural unit. The pavement slab tends to shorten when its temperature drops or its moisture content decreases. This contraction is resisted by the subgrade through friction and shear between it and the slab. The resistance to movement must be balanced by the tensile resistance of the steel crossing a crack. The maximum stress will occur at a crack at mid-length of a slab. Reinforcement is designed for the stress developed in this condition.

Resistance to movement will vary with the amount of movement, size of slab, rate of temperature change, and characteristics of the subgrade. The maximum resistance is reached when actual sliding of the slab on the subgrade occurs. A coefficient of resistance of 1.5 is recommended for design purposes. This coefficient may vary between 1 and 2.

The cross-sectional area of steel \( A_s \) required per foot of slab width is:

\[
A_s = \frac{FLw}{2f_t}
\]

Where:
- \( A_s \) = Cross-sectional area of steel per foot width of slab, sq. in.
- \( F \) = Coefficient of resistance between slab and subgrade.
- \( L \) = Distance between free transverse joints or between free longitudinal edges, ft.
- \( w \) = Weight of pavement slab, lb/sq ft.
- \( f_t \) = Allowable working stress in the steel, psi.

The formula is used for both longitudinal and transverse steel, and is solved graphically in Figure III-2.

The percentage of longitudinal steel in a continuously reinforced pavement has been established by experience in experimental installations. It varies between 0.5 and 0.8 percent of the cross-sectional area of the pavement. Such variables as tensile strength of concrete, yield strength of steel, seasonal...
### Table III-2

**Recommended Tie Bar Spacings**

<table>
<thead>
<tr>
<th>Type and grade of steel</th>
<th>Working Stress, psi</th>
<th>Pavement thickness in.</th>
<th>Minimum overall length in.*</th>
<th>Maximum Spacing, in.*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Lane Width 10 ft.</td>
<td>Lane Width 11 ft.</td>
</tr>
<tr>
<td>Grade 40 billet or axle steel</td>
<td>30,000</td>
<td>6</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>48</td>
<td>48</td>
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<td></td>
<td></td>
<td>8</td>
<td>43</td>
<td>44</td>
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<td>9</td>
<td>38</td>
<td>39</td>
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<td>10</td>
<td>35</td>
<td>35</td>
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<tr>
<td></td>
<td></td>
<td>11</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>32</td>
<td>29</td>
</tr>
</tbody>
</table>

*It is recommended that spacing of tie bars should not exceed 48 inches

**350 psi assumed for bond stress (u).

Length includes 3-inch allowance for centering.
\[ A_s = \frac{FLW}{2fs} \]

\( A_s \) = Area Steel - Sq. In./Ft. Slab Width

\( F \) = Friction Coefficient - 1.5 Assumed

\( L \) = Slab Length - Feet

\( f_s \) = Working Stress of Steel - psi

\( W \) = Weight of Slab - Per Sq. Ft.

(150) Lbs. Per Cu. Ft. Weight Assumed

---

**Figure III-2** Distributed Steel Requirements, Jointed Rigid Pavements
variations in temperature, and judgment based on experience should all be correlated in determining the percentage of steel.

3.5-DETERMINATION OF PAVEMENT SLAB THICKNESS

Determination of the thickness of the pavement slab is accomplished by the use of the design chart (Figure III-1) in the following steps:

1. Using a straightedge, draw a line from the estimated equivalent total or daily 18-kip single-axle loads on the left scale, through the applicable value of working stress \( f_c \) of the concrete on the second scale, to intersect turning line 1.

2. With a second application of the straightedge, draw a line from the subgrade reaction \( K \) on the far right scale through the adjacent scale for concrete modulus on to turning line 2.

3. Draw a line connecting the two turning lines, and the intersection of this line with the curved slab thickness scale gives slab thickness in inches. This thickness should be rounded to the next whole inch.
Appendix D—DEVELOPMENT OF FACTORS PERTAINING TO DESIGN OF RIGID PAVEMENT STRUCTURES
Revised, 1981

D.1—DEVELOPMENT OF RIGID PAVEMENT DESIGN CHARTS

The General AASHO Road Test equation is:

\[ G_t = \beta \left( \log W_t - \log \rho \right) \]  

(D-1)

where:

- \( G_t \) = a function (the logarithm) of the ratio of loss in serviceability at time \( t \) to the potential loss taken to a point where \( p_t = 1.5 \).
- \( \beta \) = a function of design and load variables that influence the shape of the \( p \)-versus-\( W \) service-ability curve.
- \( W_t \) = axle load applications at end of time \( t \).
- \( \rho \) = a function of design and load variables that denotes the expected number of axle load applications to a serviceability index of 1.5.
- \( p_t \) = serviceability at end of time \( t \).

At the AASHO Road Test, the terms \( \beta \) and \( \rho \) were found to have the following relationship to load and pavement variables for rigid pavements:

\[ \beta = 1.00 + \frac{3.63(L_1 + L_2)^{5.20}}{(D + 1)^{8.46}L_2^{3.52}} \]  

(D-2)

and:

\[ \log \rho = 5.85 + 7.35 \log (D+1) - 4.62 \log (L_1 + L_2) + 3.28 \log L_2 \]  

(D-3)

where:

- \( L_1 \) = load on one single axle or on one tandem axle set, kips.
- \( L_2 \) = axle code (1 for single axle and 2 for tandem axle).
- \( D \) = thickness of slab, inches.

Since the equations for both \( \beta \) and \( \rho \) contain the terms \( L_1, L_2 \) and \( D \), the solution of equation (D-2) for \( D \) is an involved iterative process. The solution of the equation is simplified if all load factors are expressed in terms of a common denominator. The common denominator used in this guide is an 18,000-lb single-axle load. For these conditions (\( L_1 = 18 \) kips, \( L_2 = 1 \)), equation (D-2) becomes:

\[ \beta = 1.00 + \frac{3.63(18+1)^{5.20}}{(D + 1)^{8.46}} \]  

or:

\[ \beta = 1.00 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}} \]  

(D-4)
and:
\[ \log \rho = 5.85 + 7.35 \log (D+1) - 4.62 \log (18+1) \]

or:
\[ \log \rho = 7.35 \log (D+1) - 0.06 \]  

Rewriting equation (D-1) as:
\[ \log W_1 = \log \rho + G_1/\beta \]  

then inserting equations (D-4) and (D-5) into equation (D-6) gives:
\[ \log W_1 = 7.35 \log (D+1) - 0.06 + G_1/\beta \]

where:
\[ G_1 = \log \left( \frac{4.5 - p_i}{4.5 - 1.5} \right) = \log[0.333 (4.5 - p_i)] \]

Equation (D-6) provides a basis for developing design charts involving the factors \( W_{18}, D \) and \( p_1 \) (a value of 2.5 is used in this guide). However, these charts would be applicable only to conditions similar to those at the Road Test such as:

1. Modulus of elasticity of concrete, \( E_c \).
2. Modulus of rupture of concrete, \( S_c \).
3. Modulus of subgrade reaction, \( k \).
4. Environmental conditions.
5. Life span (two years on the Road Test).
6. Load transfer characteristics.

Each chart would have values for total weighted load applications (\( W_{18} \)), for slab thickness (\( D \)), and for a single point on a scale for modulus of subgrade reaction representative of Road Test conditions.

To account for conditions other than those which existed at the Road Test, it was necessary to modify the general Road Test equation using experience and theory. This was accomplished by comparing stresses calculated from strain measurements on the Road Test pavement slabs with stresses calculated using the theoretically based formulas of Westergaard, Spangler and Pickett. Although the absolute values of stress differed, the Westergaard and Spangler equations linearized the Road Test measurements, and because of its simplicity the Spangler equation was selected to extend the Road Test equation to other conditions.

The Spangler equation is given as:
\[ \sigma = \frac{JP}{D^2} \left( 1 - \frac{a_1}{I} \right) \]  

where:
\( \sigma \) = maximum tensile stress in concrete, psi. (Multiply \( \sigma \) psi, by 47.88 to obtain pascals Pa.)
\( P \) = wheel load, lb.
\( D \) = slab thickness, inches.

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a1 = distance from corner of slab to center of load, inches. (equals \( a \sqrt{2} \), where \( a \) is the radius of a circle equivalent to the tire contact area).

\( J = \) load transfer coefficient (equals 3.2 for protected corner).

\[
l = \left( \frac{ZD^3}{12 (1 - \mu^2)} \right)^{0.25}
\]

\( Z = \frac{E}{k} \).

\( E = \) Young's modulus of elasticity of concrete, psi.

\( k = \) Modulus of subgrade reaction, psi/in.

\( \mu = \) Poisson's ratio for concrete.

In order to simplify the equation, without damage to theory, Poisson's ratio (\( \mu \)) was fixed at 0.20 and \( a_1 \) was set equal to 10 inches (the average for Road Test loads). For these values the radius of relative stiffness (\( l \)) becomes

\[
l = \left( \frac{ZD^3}{11.52} \right)^{0.25}
\]

No other modifications were made to the Spangler equation. The coefficient \( J \) requires additional analysis before it can be used in design; however, it cancels out in the present equations for design of pavements whose slab continuity does not differ from those used at the Road Test (protected corner).

Stresses were then calculated for different combinations of Road Test variables using equation (D-8). The ratio of the modulus of rupture to the calculated stresses (\( S_c/\sigma \)) was subsequently compared with axle load applications. These comparisons indicated that for any given load and terminal serviceability level (\( p_1 \)), the following relationship exists:

\[
\log W_t = a + b \log F \quad (D-9)
\]

where:

\( W_t = \) number of applications of a given load to a terminal serviceability level \( p \).

\( F = S_c/\sigma \).

\( S_c = \) modulus of rupture, psi.

\( \sigma = \) stresses calculated from equation (D-8).

\( p_t = \) terminal serviceability selected for design.

\( a = \) a constant.

\( b = \) slope of the \( \log W_t \) vs \( \log S_c/\sigma \) curve.

To determine the difference in life between a pavement with physical properties described by \( F \) (the Road Test pavement) and one with different physical properties \( F' \), equation (D-9) was differentiated to give:

\[
d(\log W_t) = (b_{p_t})d(\log F)
\]

or:

\[
\log W_t' - \log W_t = b_{p_t}(\log F' - \log F)
\]

Rearranging terms:
\[
\log W'_t = \log W_t + b_{p_t} (\log F' - \log F) \tag{D-10}
\]

where:

\[ W_t \] = number of load applications to reach serviceability \( p_t \) for a pavement whose physical properties are similar to Road Test pavements.

\[ F = \frac{S_c}{\sigma} \] for Road Test pavements.

\[ W'_t \] = number of load applications to reach serviceability \( p_t \) for pavements with physical properties described by \( F' \)

\[ F' = \frac{S_{c'}}{\sigma'} \] for physical properties different from those at the Road Test.

Further examination of the Road Test data indicated that for a range of \( p_t \) between 1.5 and 2.5, the following relationship between \( p_t \) and \( b \) existed:

\[ b_{p_t} = 4.22 - 0.32 p_t \tag{D-11} \]

Combining equations (D-10) and (D-11):

\[
\log W'_t = \log W_t + (4.22 - 0.32p_t) (\log F' - \log F) \tag{D-12}
\]

Before further development can take place, two important assumptions must be made:

1. The variation in pavement life (W) for different loads at the same level of \( S_c/\sigma \) is accounted for by the basic Road Test equation and is covered in this design procedure by the traffic equivalence factors given in Section D.2.

2. Any change in F resulting from changes in the physical constants \( E, k, D \) and \( S_c \) will have the same effect on \( W_t \) as varying slab thickness, and this relationship is defined by equation (D-9).

With the above assumptions, it is possible now to incorporate theory into the Road Test results. Combining equations (D-7) and (D-12) gives:

\[
\log W'_t = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + 1.624 \times 10^7 (D + 1)^{0.46}}
+ (4.22 - 0.32p_t) (\log F' - \log F) \tag{D-13}
\]

where:

\[
\log F' - \log F = \log \left( \frac{S_{c'}}{\sigma'} \cdot \frac{\sigma}{S_c} \right) = \log \left( \frac{S_{c'}}{S_c} \cdot \frac{\sigma}{\sigma'} \right)
\]

or:

\[
\log \left( \frac{S_{c'}}{S_c} \cdot \frac{\sigma}{\sigma'} \right) = \log \left[ \left( \frac{S_{c'}}{S_c} \right)^{\frac{1}{3}} \left( \frac{D^{0.8} - 18.42/Z^{0.25}}{D^{0.8} - 18.42/Z^{0.25}} \right) \right]
\]

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Equation (D-12) can be written:

\[ \log W'_{t,s} = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} + (4.22 - 0.32p_t) \log \left[ \frac{\left( \frac{S'_{c}}{S_{c}} \right)}{\left( \frac{J}{J} \right)} \left( \frac{D^{0.75} - 18.42/Z^{0.75}}{D^{0.75} - 18.42/Z^{0.75}} \right) \right] \] (D-14)

Substituting in equation (D-14) the values for the physical constants (Z = E/k, S, and J) that represent Road Test condition as follows:

- \( E = 4.2 \times 10^6 \) psi (static test @ 28 days)
- \( k = 60 \) pci (gross, 30-in.-dia. plate)
- \( S = 690 \) psi (28 day, 1/3-point loading)
- \( J = 3.2 \) (assumed value for protected corner)

This gives:

\[ \log W'_{t,s} = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} + (4.22 - 0.32p_t) \log \left[ \frac{\left( \frac{S'_{c}}{S_{c}} \right)}{\left( \frac{J}{J} \right)} \left( \frac{D^{0.75} - 1.132}{D^{0.75} - 18.42/Z^{0.75}} \right) \right] \] (D-15)

This equation is that used to develop the design chart given in Chapter III, Figure III-1. When using this equation, it is necessary to employ physical constants determined in the manner employed at the Road Test (i.e., E, 28-day static; k, gross value with 30-inch (762mm) diameter plate; S, 28-day, 1/3-point loading). If other tests are used, it will be necessary to adjust the value by correlation.

As mentioned in the original guide, the Road Test equation is based on pavements which have a sand-gravel subbase. It is possible to include a term (Q) in equation (D-15) to permit variation of subbase quality. The term (Q) would be arbitrary and based on experience and the data available from Experiment Design 3 at the Road Test. Such a term would account for potential changes in k during the life of the pavement and would require further research to evaluate properly the subbase performance term.

Also, Equation (D-15) does not provide for variations in pavement life \( W \) which may result from changes in environment or weather. This could be an important factor in areas different from the Road Test site. If desirable, a regional factor (R) could be included in equation (D-15) to account for differences in frost penetration, rainfall, daily temperature variation, and other weather factors.

Although these factors are not included in the design chart, future efforts should be aimed at establishing their effects so that at some later date each can be included.
D.2-DETERMINATION OF TRAFFIC EQUIVALENCE FACTORS FOR RIGID PAVEMENTS

To use the rigid pavement design procedure presented in this guide, mixed traffic must be converted to an equivalent number of 18-kip (80kN) single-axle loads. The procedure for accomplishing this conversion includes:

1. Derivation of Traffic Equivalence Load Factors.
2. Conversion of mixed traffic to equivalent 18-kip (80kN) single-axle load applications.
3. Lane distribution considerations.

To express varying axle loads in terms of a common denominator, it is necessary to develop traffic equivalence load factors. These factors, when multiplied by the number of axle loads within a given weight category, give the number of 18-kip (80kN) single-axle load applications which has an equivalent effect on the performance of the pavement structures. Analysis of the AASHO Road Test design equations permits the determination of such factors.

The design equation for rigid pavements developed in Section D.1 may also be written as:

$$\log W_1 = 5.85 + 7.35(\log D + 1) - 4.62(\log L_1 + L_2) + 3.28 \log L_2 + G_f/\beta$$  \hspace{1cm} (D-16)

All terms in this equation are as defined in Section D.1. If $L_1$ equals 18 kips and $L_2$ equals 1, for single axles, equation (D-16) becomes

$$\log W_{1s} = 5.85 + 7.35 \log (D + 1) - 4.62 \log (18 + 1) + G_f/\beta_{18}$$  \hspace{1cm} (D-17)

For any other axle load $L_1$ equal to $X$, equation (D-16) becomes:

$$\log W_{1x} = 5.85 + 7.35 \log (D + 1) - 4.62 \log (L_x + L_2) + 3.28 \log L_2 + G_f/\beta_x$$  \hspace{1cm} (D-18)

Subtracting equation (D-17) from equation (D-18) gives:

$$\log \frac{W_{1x}}{W_{1s}} = 4.62 \log (18 + 1) - 4.62 \log (L_x + L_2) + 3.28 \log L_2 + G_f/\beta_x - G_f/\beta_{18}$$  \hspace{1cm} (D-19)

For single axles ($L_2 = 1$), equation (D-19) reduces to:

$$\log \frac{W_{1x}}{W_{1s}} = 4.62 \log (18 + 1) - 4.62 \log (L_x + 1) + G_f/\beta_x - G_f/\beta_{18}$$  \hspace{1cm} (D-20)

or, for tandem axles ($L_2 = 2$), to:

$$\log \frac{W_{1x}}{W_{1sh}} = 4.62 \log (18 + 1) - 4.62 \log (L_x + 2) + 3.28 \log 2 + G_f/\beta_x - G_f/\beta_{18}$$  \hspace{1cm} (D-21)

The ratio of $W_{1x}/W_{1s}$ gives the relationship between any axle load $X$, single or tandem, and an 18-kip single-axle load. As before, the ratio is an equivalence factor and is evaluated by solving equations D-20 and D-21 for any other load $X$. Because the term $\beta$ is a function of $D$ as well as $L_x$ (Section D.1), the equivalence factor also varies with $D$. The computer equivalence factors for a
Table D.2-1
Traffic Equivalence Factors, Rigid Pavement

Single Axles, $p_1 = 2.5$

<table>
<thead>
<tr>
<th>Axle Load</th>
<th>Kips</th>
<th>D - Slab Thickness - inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>8.9</td>
<td>0.0002 0.0002 0.0002 0.0002 0.0002 0.0002 0.0002 0.0002</td>
</tr>
<tr>
<td>4</td>
<td>17.8</td>
<td>0.003 0.002 0.002 0.002 0.002 0.002 0.002 0.002</td>
</tr>
<tr>
<td>6</td>
<td>26.7</td>
<td>0.01 0.01 0.01 0.01 0.01 0.01 0.01 0.01</td>
</tr>
<tr>
<td>8</td>
<td>35.6</td>
<td>0.04 0.04 0.03 0.03 0.03 0.03 0.03 0.03</td>
</tr>
<tr>
<td>10</td>
<td>44.5</td>
<td>0.10 0.09 0.08 0.08 0.08 0.08 0.08 0.08</td>
</tr>
<tr>
<td>12</td>
<td>53.4</td>
<td>0.20 0.19 0.18 0.18 0.18 0.18 0.18 0.18</td>
</tr>
<tr>
<td>14</td>
<td>62.3</td>
<td>0.30 0.29 0.28 0.28 0.28 0.28 0.28 0.28</td>
</tr>
<tr>
<td>16</td>
<td>71.2</td>
<td>0.45 0.44 0.44 0.44 0.44 0.44 0.44 0.44</td>
</tr>
<tr>
<td>18</td>
<td>80.1</td>
<td>0.60 0.59 0.59 0.59 0.59 0.59 0.59 0.59</td>
</tr>
<tr>
<td>20</td>
<td>89.0</td>
<td>0.75 0.74 0.74 0.74 0.74 0.74 0.74 0.74</td>
</tr>
<tr>
<td>22</td>
<td>97.9</td>
<td>0.90 0.89 0.89 0.89 0.89 0.89 0.89 0.89</td>
</tr>
<tr>
<td>24</td>
<td>106.8</td>
<td>1.05 1.05 1.05 1.05 1.05 1.05 1.05 1.05</td>
</tr>
<tr>
<td>26</td>
<td>115.7</td>
<td>1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20</td>
</tr>
<tr>
<td>28</td>
<td>124.6</td>
<td>1.35 1.35 1.35 1.35 1.35 1.35 1.35 1.35</td>
</tr>
<tr>
<td>30</td>
<td>133.4</td>
<td>1.50 1.50 1.50 1.50 1.50 1.50 1.50 1.50</td>
</tr>
<tr>
<td>32</td>
<td>142.3</td>
<td>1.65 1.65 1.65 1.65 1.65 1.65 1.65 1.65</td>
</tr>
<tr>
<td>34</td>
<td>151.2</td>
<td>1.80 1.80 1.80 1.80 1.80 1.80 1.80 1.80</td>
</tr>
<tr>
<td>36</td>
<td>160.1</td>
<td>1.95 1.95 1.95 1.95 1.95 1.95 1.95 1.95</td>
</tr>
<tr>
<td>38</td>
<td>169.0</td>
<td>2.10 2.10 2.10 2.10 2.10 2.10 2.10 2.10</td>
</tr>
<tr>
<td>40</td>
<td>177.9</td>
<td>2.25 2.25 2.25 2.25 2.25 2.25 2.25 2.25</td>
</tr>
</tbody>
</table>

Table D.2-2
Traffic Equivalence Factors, Rigid Pavement

Tandem Axles, $p_1 = 2.5$

<table>
<thead>
<tr>
<th>Axle Load</th>
<th>Kips</th>
<th>D - Slab Thickness - inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>44.5</td>
<td>0.01 0.01 0.01 0.01 0.01 0.01 0.01 0.01</td>
</tr>
<tr>
<td>12</td>
<td>53.4</td>
<td>0.03 0.03 0.03 0.03 0.03 0.03 0.03 0.03</td>
</tr>
<tr>
<td>14</td>
<td>62.3</td>
<td>0.06 0.06 0.06 0.06 0.06 0.06 0.06 0.06</td>
</tr>
<tr>
<td>16</td>
<td>71.2</td>
<td>0.09 0.09 0.09 0.09 0.09 0.09 0.09 0.09</td>
</tr>
<tr>
<td>18</td>
<td>80.1</td>
<td>0.12 0.12 0.12 0.12 0.12 0.12 0.12 0.12</td>
</tr>
<tr>
<td>20</td>
<td>89.0</td>
<td>0.15 0.15 0.15 0.15 0.15 0.15 0.15 0.15</td>
</tr>
<tr>
<td>22</td>
<td>97.9</td>
<td>0.18 0.18 0.18 0.18 0.18 0.18 0.18 0.18</td>
</tr>
<tr>
<td>24</td>
<td>106.8</td>
<td>0.21 0.21 0.21 0.21 0.21 0.21 0.21 0.21</td>
</tr>
<tr>
<td>26</td>
<td>115.7</td>
<td>0.24 0.24 0.24 0.24 0.24 0.24 0.24 0.24</td>
</tr>
<tr>
<td>28</td>
<td>124.6</td>
<td>0.27 0.27 0.27 0.27 0.27 0.27 0.27 0.27</td>
</tr>
<tr>
<td>30</td>
<td>133.4</td>
<td>0.30 0.30 0.30 0.30 0.30 0.30 0.30 0.30</td>
</tr>
<tr>
<td>32</td>
<td>142.3</td>
<td>0.33 0.33 0.33 0.33 0.33 0.33 0.33 0.33</td>
</tr>
<tr>
<td>34</td>
<td>151.2</td>
<td>0.36 0.36 0.36 0.36 0.36 0.36 0.36 0.36</td>
</tr>
<tr>
<td>36</td>
<td>160.1</td>
<td>0.39 0.39 0.39 0.39 0.39 0.39 0.39 0.39</td>
</tr>
<tr>
<td>38</td>
<td>169.0</td>
<td>0.42 0.42 0.42 0.42 0.42 0.42 0.42 0.42</td>
</tr>
<tr>
<td>40</td>
<td>177.9</td>
<td>0.45 0.45 0.45 0.45 0.45 0.45 0.45 0.45</td>
</tr>
</tbody>
</table>

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wide range of axle loads (single and tandem) are summarized in Tables D.2-1 and D.2-2 for D ranging from 6 to 12 inches and a $p_1$ value of 2.5.

The prediction of traffic for design purposes must rely on information from past traffic, modified by factors for growth or other expected changes. Most states accumulate past traffic information in the form of loadometer data of the Federal Highway Administration W4 loadometer tables, which are tabulations of number of axles observed within a series of axle-load groups, with each group usually a 2,000-lb (8.9kN) interval. These tabulations are in a convenient form for conversion, since the number of vehicles in each load group may be multiplied by the appropriate traffic equivalence factor to give $W_{t,i}$ for each load group. This can be accomplished as follows:

\[
W_i = N_i \cdot e_i = N_i \cdot P_i \cdot e_i
\]  

where:

- $W_i$ = equivalent 18-kip (80kN) single-axle loads for load group $i$.  
- $N_i$ = number of axles expected for load group $i$.  
- $N_n$ = total number of axles.  
- $P_i$ = percent of axles in load group $i$.  
- $e_i$ = traffic equivalence factor for load group $i$.

The number of equivalent 18-kip single-axle loads for all axle groups is then summed to give one number representative of mixed traffic:

\[
W_{t,s} = W_1 + W_2 + \ldots + W_i + \ldots + W_n
\]

or

\[
W_{t,s} = \sum_{i=1}^{n} W_i
\]  

or

\[
W_{t,s} = N_i \sum_{i=1}^{n} P_i e_i
\]

Equations (D-20) and (D-21) are used to compute the traffic equivalence factors ($e_i$). These factors are, however, a function of $D$. Therefore to arrive at the design $W_{t,s}$ it is necessary to assume a value for $D$, use the equivalent load factors listed for the assumed value of $D$ and then solve equation (D-23). If, after completion of the design problem, the computed $D$ is appreciably different from the assumed value, a new assumption should be made, the design traffic number ($W_{t,s}$) recomputed, and $D$ determined for the new $W_{t,s}$. This procedure should be continued until the assumed and computed values of $D$ are reasonably close.

The number of equivalent axle loads derived in the preceding section represents the total for all lanes and both directions of travel. This number must be distributed by direction and lanes for design purposes.

Directional distribution is usually made by assigning 50 percent of the traffic to each direction, unless special considerations warrant some other distribution. In regard to lane distribution, most states assign 100 percent of the traffic in each direction to the design lane. Some states have developed lane
Table D.2-3

Lane Distribution Factors on Multilane Roads

<table>
<thead>
<tr>
<th>Number of Lanes In both Directions</th>
<th>Percent of $W_{t,\infty}$ In Design Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>80-100</td>
</tr>
<tr>
<td>6*</td>
<td>60-80</td>
</tr>
</tbody>
</table>

distribution factors for multilane facilities. The range of factors used is given in Table D.2-3.

If lane or directional distribution factors are used and pavements are designed on the basis of distributed traffic, consideration should be given to the use of variable cross sections. Heavier structural sections should be provided in the outside lanes when warranted on the basis of the lane distribution analysis.

D.3 — DESIGN EXAMPLES, RIGID PAVEMENTS

This Section presents examples of the use of the Guide for design of rigid pavement structures. A typical Rural interstate highway problem is used to illustrate use of the regular procedure.

Example Problem No. 1.

Total equivalent 18-kip (80kN) single-axle loads for the design lane are determined from the estimate of the type and amount of traffic expected over the period and the appropriate traffic equivalence factors. The procedure used to reduce mixed traffic to a design traffic number is given in Section D.2. Because selection of the equivalence factors to be used to convert mixed traffic to total 18-kip (80kN) single-axle loads depends on the slab thickness (D), a value for D must be assumed for the initial conversion. The use of an average or assumed D value for the determination of 18-kip (80kN) single-axle equivalence factors instead of the D value determined by design, normally will not result in significant design differences. If, however, the value of D resulting from solution by the design chart differs appreciably from the assumed value, a new value should be assumed, the number of 18-kip single-axle loads recalculated, and the design operation repeated.

For this example, it is assumed that the total 18-kip (80kN) single-axle loads expected in the design lane during the next 20 years will equal 9,000,000 (1,233 daily applications).

The modulus of rupture ($S_c$) at 28 days, as determined by the test procedure specified in AASHTO Designation T-97, using third-point loading, is the basis for concrete flexural strength. The working stress is determined by the equation $f_1 = S_c/C$ (Section 3.4.2). Since this is a rural interstate and traffic volumes are not expected to exceed the capacity of a probable detour, a factor of safety of 1.33 is considered satisfactory. For this example, the modulus of rupture ($S_c$) is assumed to have been determined to be 650 psi (4.48MPa). Thus, the working stress ($f_1 = S_c/1.33$) for use in the design chart is 490 psi (3.10MPa). The modulus of elasticity may be determined
from static compression tests on cylinders. An average value is acceptable for use in computations. A value of 4,200,000 psi (29GPa) is assumed for this example.

Westergaard's modulus of subgrade reaction (k), referred to as "gross k" in Road Test reports, is used in this guide. It represents the load in pounds per square inch on a loaded area, divided by the deflection in inches of that loaded area. The scales for k in the design charts are correlated with values obtained by plate loading tests performed in accordance with AASHTO Designation T-222 using a 30-inch (762mm) diameter plate. For this example, k is assumed to have been determined as 200 pci.

The determination of the thickness of the pavement slab is accomplished by use of the design chart, Figure III-1. A line is drawn from the estimated total equivalent total 18k single axle load applications for 20 years (1,233 daily applications) on the left scale through the applicable value of working stress (f_t = 490 psi) of the concrete on the second scale, to intersect turning line 1. A line is then drawn from the modulus of subgrade reaction \(K_e = 200 \text{pci}\) on the far right scale through the adjacent scale for concrete modulus \(4.2 \times 10^6 \text{psi}\) on to turning line 2. A line connecting the two turning lines is drawn intersecting the slab thickness scale at 9.6 inches. The design thickness for this pavement slab is 10 inches.

The design charts are based on protected corner conditions for jointed pavements. (An alternate design procedure which provides for the design of continuously reinforced concrete pavements is presented in Section D.4.) A discussion of joints and of the design of load transfer devices and tie bars is presented in Chapter III. For this example, it is assumed that mechanical load transfer devices will be used in transverse contraction joints, the bars in longitudinal joints and reinforcement in the slabs.

For this design example, and for plain round steel dowels as a load transfer device the minimum requirements for a 10-inch (250mm) pavement slab (from Section 3.4.3.4) are as follows:

- **Dowel Diameter**: 1 ½ inches (32mm)
- **Dowel Length**: 18 inches (457mm)
- **Dowel Spacing**: 12 inches (305mm)

Other mechanical devices that have proven satisfactory in field installations may also be used for load transfer in lieu of plain round dowels.

Tie bars are used to prevent slabs in adjacent lanes from separating. The requirements for tie bars may be determined from Table III-2. For this example, it is assumed that 5/8-inch (16mm) diameter bars with a working stress of 30,000 psi (207MPa) are to be used. Tie bar requirements are then as follows:

- **Minimum Over all Length**: 30 inches (762mm)
- **Maximum Spacing**: 48 inches (1.22m)

Other approved connectors may also be used in lieu of deformed bars. The tensile strength of such connectors should be equal to that of the deformed bar that would be required, and the maximum spacing should conform to the requirements for deformed bars.

Distributed steel reinforcement is designed for the maximum steel stress which would occur in resisting movement at a crack at mid-length of a slab. The cross-sectional area of steel \(A_s\) in inches\(^2\) required per foot of slab width is:
This equation is used for both longitudinal and transverse steel, and may be solved graphically by means of Figure III-2, where the factors in the formula are defined. For this example, the following are assumed: Slab length, 40 feet (12.19m); slab width, 24 feet (7.31m); working stress of steel ($f_s$), 45,000 psi (310MPa). The resulting areas of steel ($A_s$) square inches per foot width of slab required for the 10-inch (25mm) thick pavement slab are: longitudinal, 0.09 (58mm$^2$); transverse, 0.05 (32mm$^2$).

Example Problem No. 2

The use of a higher safety factor for certain conditions are allowed in Chapter III. Assume the same values used in example problem I except considering the location is an urban area where closing a lane for possible rehabilitation will cause the projected traffic to exceed capacity of a probable detour, thereby utilizing the higher recommended C value of 2.0.

- Modulus of Subgrade Reaction—200 pci
- Concrete Elastic Modulus—4,200,000 psi
- Daily Equivalent 18 kip single axle load—1,233
- Working Stress—($650/2) = 325$ psi

This results in the design slab thickness of 12 inches.

Using the same load transfer devices, tie bars and reinforcement in the slab, minimum requirements for a 12-inch pavement slab are as follows:

- Dowel Diameter: 1½ in.
- Dowel Length: 18 inches
- Dowel Spacing: 12 inches
- 5/8 inch diameter Tie Bars
- Minimum Overall length: 30 inches
- Maximum Spacing: 41 inches
- Reinforcement Area of Steel
  - Longitudinal: 0.10 in$^2$/ft.
  - Transverse: 0.06 in$^2$/ft.

D.4 – ALTERNATE PROCEDURE FOR THE DESIGN OF RIGID PAVEMENT STRUCTURES

This alternate procedure for the design of rigid pavement structures provides means for considering additional variables. The same basic limitations apply to this procedure as to the procedure presented in Chapter III, and the same recommendations and assumptions regarding materials properties and specifications are also applicable. The two methods will give identical results for conditions representative of the AASHO Road Test. This alternate procedure should also be considered interim in nature and subject to periodic review and revision as may be found necessary through practice and further research.

This design procedure is based on the equations developed from the AASHO Road Test as presented in Section D.1, with modifications as described below. It provides for the design of subbase thickness, pavement slab thickness, pavement slab reinforcement, and joint details. The steps required may be summarized as follows:
1. Select pavement and subbase type.
2. Determine a composite k-value at top of subbase that accounts for the effective supporting capability of the subgrade and subbase.
3. Determine the pavement thickness using the composite $k_c$-value.
4. Determine slab dimensions.
5. Determine the reinforcement design, if reinforcement is used.

**Subbase Design**

The subbase stiffness and the modulus of subgrade reaction are used with Fig. D.4-1 to estimate a composite k-value. Also required is the selection of a subbase thickness by the designer based on other criteria such as experience, construction control, and gradeline requirements.

The chart is used by entering with the subbase thickness on the vertical scale and projecting horizontally to the expected subbase stiffness. See Table D.3-1 for range of stiffness for some typical subbase materials.

From this intersection a line is projected vertically until it intersects the appropriate subgrade modulus value. (Interpolation will be required for values other than those shown on the chart.) This point is then projected horizontally until it intersects the vertical axis. The composite $k_c$-value at the top of the subbase is then read on the vertical scale.

The composite $k_c$-value obtained from Fig. D.4-1 is used in pavement thickness determinations. Although there are no specific procedures for reducing support value to account for losses due to pumping, erosion, consolidation, etc., a design agency should consider such reduction based on its own experience. When untreated granular materials are used or the pavement is placed directly on the subgrade, substantial reductions in the k-values might be considered. If subbases are stabilized with additives, little or no reduction may be required.

<table>
<thead>
<tr>
<th>Material</th>
<th>Stiffness Range, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granular</td>
<td>(55-138MPa)</td>
</tr>
<tr>
<td>Cement stabilized base</td>
<td>(3.5-6.9GPa)</td>
</tr>
<tr>
<td>Cement stabilized soil</td>
<td>(2.8-6.2GPa)</td>
</tr>
<tr>
<td>Asphalt treated base</td>
<td>(2.4-6.9GPa)</td>
</tr>
<tr>
<td>Asphalt-emulsion treated base</td>
<td>(28-2.1GPa)</td>
</tr>
</tbody>
</table>

**Pavement Slab Thinckness**

The pavement thickness design procedure presented herein is based on data developed in the AASHO Road Test, supplemented and modified by theoretical analyses. (See Section D.1 for a detailed development of the pavement thickness design equation.) The pavement serviceability-performance concept developed in connection with the AASHO Road Test is accepted as the basic philosophy of this design.

Figure D.4-2 is a nomograph for solving the alternate design equation developed. Values for the following variables are entered on the scales indicated:
Figure D.4-1 Chart for Estimating Composite K-Value

May be determined by means of the laboratory test method for Modulus of Resilience \( (M_R) \) described in NCHRP Report 128.
1. Design lane total equivalent 18-kip single-axle loads (Scale 1).
2. Level of serviceability (Scale 2).
3. Pavement continuity term, or pavement type (Scale 3).
4. Concrete working stress (Scale 4).
5. Modulus of elasticity of concrete (Scale 6).
6. Composite \( k_c \)-value (Scale 7).

In using the chart to solve for pavement thickness, Scales 1, 2, 3, and 4 are consecutively projected through turning lines 1 and 2 to turning line 3. Scales 6 and 7 are then connected to establish a point on turning line 4. By connecting turning lines 3 and 4, the required pavement thickness \( D \) is determined from Scale 5. Values are then rounded to the next whole inch.

The thickness from Fig. D.4-2 applies to all slabs having provisions for adequate load transfer through mechanical devices or aggregate interlock, whether reinforced or nonreinforced.

Where the design analysis indicates a slab thickness of less than 8 inches, careful consideration must be given to environmental conditions, and to the construction problems to be encountered, before such lesser thickness is used. Thicknesses less than 8 inches may be satisfactory on minor highway systems carrying light traffic.

**Serviceability Index**

The designer has the option of specifying the level of service during the design life through the selection of the terminal serviceability index. In general, the higher the service level desired during the design life the thicker the pavement, thus the greater the cost. The present serviceability rating development study for the AASHO Road Test found that 100 percent of the rating panel felt that a pavement with a \( p_1 \) of 1.5 should not be on the highway system. Hence, failure was defined as \( p_1 = 1.5 \) at the Road Test. For major highways (e.g., Interstate and primary) a value of 2.5 is recommended for \( p_1 \). This value may be too low, depending on state practices, as the experience of some states indicates a value of 3.0 to be more realistic.

Figure D.4-3 illustrates the options available to the designer. For pavement structure A, it is evident that \( W_{1.5} > W_{2.0} > W_{2.5} \). If the designer wishes to design for total traffic of \( W_t \), and a \( p_1 = 2.5 \), pavement structure A is inadequate, and a structure such as represented by performance curve B must be used. Another important parameter is the initial value of present serviceability index.

**Concrete Properties**

The concrete properties required in the design process are modulus of elasticity, flexural strength, and tensile strength. The first two properties are required for the pavement thickness determination and the latter for reinforcement design. In the design process the flexural strength and tensile strength must be converted to working stresses.

The modulus of elasticity may be determined from static compression tests on cylinders (ASTM C469). An average value is acceptable for use in computations. A value of 4,200,000 psi (29GPa) was used as representative of AASHTO conditions for developing the design equations.

A scale for working stress in the concrete is included in the design charts to permit thickness adjustments for flexural strengths \( S_c \) different than those 115
Figure D.4-2 Design Chart, Alternate Procedure for Design of Rigid Pavements
Figure D.4-3 Illustration of Level of Service Concept in Pavement Structure Design
obtained on the Road Test. The scale is based on 28-day strengths as determined in the test procedure specified in AASHTO Designation T-97, using third-point loading. If test data are normally available for other than 28-day strengths, the expected 28-day strengths should be obtained from a time-strength correlation and the extrapolated or interpolated values used. If the strengths are based on center-point loading, the values should be reduced to be compatible with the charts. The average flexural strength for the concrete on the AASHO Road Test was about 690 psi (4.8MPa) at 28 days.

The flexural working stress should account for concrete variability by applying a statistical adjustment to the flexural strength data. The use of an average value means that 50 percent of the data are below the design value. An alternate means to arrive at the working stress may be accomplished by applying a safety factor to the mean flexural strength. The working stress may be computed by either of the following equations:

\[
\begin{align*}
ft &= \frac{S_c}{F_s} \\
ft &= S_c - c\sigma_c
\end{align*}
\]

where:
- \(S_c\) = mean flexural strength from a series of tests, psi.
- \(\sigma_c\) = standard deviation of flexural strength tests, psi.
- \(c\) = factor from Table 0.4-1 to establish the confidence level in design.
- \(F_s\) = safety factor in design. See section 3.4.2 (Chapter III) for recommended values.

The selection of a value for \(c\) establishes the relative number of tests below the design value that a designer is willing to accept. A \(c\) value of 1.645 indicates that the designer is willing to accept the fact that 5 percent of the tests will be less than the design value. The standard deviation of flexural strength may vary from about 40 to about 125 psi (.28 to .86MPa), depending on the level of construction quality control provided.

The tensile strength \(S_t\) of concrete may be determined from direct tensile tests or from splitting tensile tests (AASHTO T-198). The tensile strength is used for design of reinforcement for continuously reinforced concrete pavement, with a greater value requiring more reinforcement. A strength value at seven days is recommended, because the basic crack pattern is established during the first week of pavement life. The design value may take concrete variability into account by the following:

\[
f = (S_t + C\sigma_t)
\]

where:
- \(S_t\) = mean value of tensile strength, psi.
- \(\sigma_t\) = standard deviation of tensile strength tests, psi.
- \(C\) = as previously defined.
- \(f\) = tensile working stress of concrete.

The correction for variability is plus in this case, as the larger value requires a greater steel percentage.
## Table D.4-1

Multiplication Factors to Establish Confidence Level of Design

<table>
<thead>
<tr>
<th>Confidence Level in %</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>0.675</td>
</tr>
<tr>
<td>80</td>
<td>0.842</td>
</tr>
<tr>
<td>85</td>
<td>1.037</td>
</tr>
<tr>
<td>90</td>
<td>1.282</td>
</tr>
<tr>
<td>95</td>
<td>1.645</td>
</tr>
<tr>
<td>99</td>
<td>2.326</td>
</tr>
</tbody>
</table>

## Table D.4-2

Friction Factor of Subbase for Use in Empirical Design Equation

<table>
<thead>
<tr>
<th>Subbase Type</th>
<th>Subbase Friction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface treatment</td>
<td>2.2</td>
</tr>
<tr>
<td>Lime stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Asphalt stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Cement stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>River gravel</td>
<td>1.5</td>
</tr>
<tr>
<td>Crushed stone</td>
<td>1.5</td>
</tr>
<tr>
<td>Sandstone</td>
<td>1.2</td>
</tr>
<tr>
<td>Natural subgrade</td>
<td>0.9</td>
</tr>
</tbody>
</table>
NOMOGRAPH

SOLVES: $P_s = \frac{LF}{2L_g} \times 100$

Scale 1

Scale 2

Scale 3

Scale 4

EXAMPLE PROBLEM:

$L = 45$ FT.

$F = 1.5$

$f_s = 52,000$ PSI

ANSWER: $P_s = 0.0078$

WHERE:

$P_s =$ REQUIRED STEEL PERCENTAGE - $S$

$L =$ WIDTH OF SLAB - FEET

$F =$ FRICTION FACTOR OF SUBBASE

$f_s =$ ALLOWABLE WORKING STRESS IN STEEL - PSI

$0.75$ OF YIELD STRENGTH RECOMMENDED,

THE EQUIVALENT OF SAFETY FACTOR OF $1.33$

Figure 9.4-4 Chart for Determining the Steel Percentage for Jointed Reinforced Concrete Pavements

120
Reinforcement Criteria

The procedure for reinforcement design depends on the pavement type and the dimension being considered. In this section, the design is divided into procedures for jointed and for continuously reinforced concrete pavement (CRCP). For the design of transverse steel, the procedure for jointed reinforced pavement should be used, regardless of pavement type. Also included in this section is a procedure for establishing bar size and spacing for a given steel requirement.

**Jointed Reinforced Concrete Pavement** — The purpose of distributed steel reinforcement is not to prevent cracking, but rather to hold tightly closed any cracks that may form, thus maintaining the pavement as an integral structural unit. A pavement tends to shorten when its temperature drops or its moisture content decreases. This contraction is resisted by the subgrade through friction and shear between it and the slab. The resistance to movement must be balanced by the tensile resistance of the steel crossing a crack. (This is commonly referred to as the Subgrade Drag Theory). The maximum steel stress will occur at a crack at mid-length of a slab. Reinforcement is designed for the stress developed in this condition.

Figure 0.4-4 is a nomograph for determining the required steel percentage (cross-sectional area) for given conditions. Values determined for the following variables are entered on the scales indicated, to solve the equation shown on the chart:

1. The distance between free edges (Scale 1).
2. The friction factor of subbase (Scale 2).
3. The working stress in the steel (Scale 4).

Scales 1 and 2 are connected and projected to the pivot line. The pivot line and Scale 4 are connected and the required steel, expressed as percentage of cross-sectional area, is read on Scale 3.

On multilane roadways, it may be economically feasible to reduce the amount of transverse steel in the outside lanes. Figure D.4-5 may be used as a guide in making the necessary computations. The figure may also be used to compute the required steel percentage at the longitudinal joints, hence the tie bar size and spacing may also be computed.

**Continuously Reinforced Concrete Pavement** — The purpose of reinforcing steel in CRCP is to hold transverse volume change cracks that occur in the pavement to a minimum width.

The minimum percentage of longitudinal steel should be .6 percent. Exceptions should be made only where experience has shown that a smaller percentage of steel has given satisfactory service. In areas of large seasonal temperature changes or where periods of extreme low temperature occur, the use of .7 percent steel may be advantageous.

The reinforcing steel should be deformed steel bars or deformed welded steel fabric that meet the requirements as set out in AASHTO Specifications. Part I, AASHTO M 31, M 42, M 53, M 54, or M 221. The tensile requirements for bars should conform to grade 60 for longitudinal bars and to either grade 40 or grade 50 for transverse bars.
Using the influence diagram, the steel percentage required for any area may be computed as follows:

**EXAMPLE PROBLEM:**

\[ \delta = 12' \]
\[ L = 36' \]
\[ F = 1.5 \]
\[ f_s = 52,500 \text{ psi} \]
\[ P_s = 0.051\% \]
\[ P_a = 0.034\% \]

\[ P_a = 2 \frac{P_s \delta}{L} \]

**WHERE:**

\( \delta \) = Distance from a free edge to the most interior point for the area under consideration.

\( L \) = Width of slab (ft)

\( P_a \) = Required steel percentage (% (from Fig. D.4-4)

By definition the term \( \delta \) must satisfy the following:

\[ \delta \leq \frac{L}{2} \]

**Figure D.4-5** Procedure for Reducing the Distributed Steel Percentage as a Function of Width
Bar Size and Spacing — The steel percentage from Figs. D.4-4 may be used with Fig. D.4-6 to determine bar size and spacing. Values determined for variables are entered on the scales indicated, to solve the equation shown on the chart:

1. Steel percentage (Scale 1).
2. Pavement thickness (Scale 2).
3. Bar Size (Scale 3).

Scales 1 and 2 are connected and extended to the pivot line. The points on the pivot line and Scale 3 are connected and extended to Scale 4, where the required spacing is read. In some cases, such as the use of deformed wire, where the cross-sectional area can be specified, the spacing may be fixed and the chart used to determine the required bar area.

D.4.1 — Alternate Design Procedure

Example Problem No. 1

This alternate procedure will result in the same design thickness as would be obtained by using the Design Chart Fig. III-I contained in the Guide for the Design of Rigid Pavement Structures when values of modulus of elasticity of concrete and continuity factor which are representative of AASHO Road Test conditions are used. To illustrate this, these values of modulus of elasticity and continuity factor with the values assumed for design in the example in D.3.3.2 will be used with this alternate procedure. These values are:

- Modulus of Elasticity of Concrete — 4,200,000 psi (29GPa)
- Continuity Factor — 3.2
- Serviceability Index — 2.5
- Concrete Working Stress — 650 \( \times 0.75 \) — 490 psi (3.4kPa)
- kg Value — 200 pci
- Traffic — 9,000,000 total (1,233 daily) equivalent 18-kip (80kN) single-axle loads

The resulting value of pavement slab thickness (D) is 10.0 inches (250mm), the same as obtained in the example in D.3.

Example Problem No. 2

Presented in the following is an example of the use of the alternate design procedure described in section D.3. One design problem is included, based on conditions representative of a typical Interstate Highway. Three design alternates are presented for use of three pavement types, as follows:

1. Nonreinforced jointed concrete pavement.
2. Reinforced jointed concrete pavement.

Determination of Pavement Slab Thickness—For this design example, it is assumed that the total number of 18-kip (80kN) single-axle loads expected in the design lane during the next 20 years will be 10,000,000 (1,370 daily) and the Terminal Serviceability Index is 2.5.

In this alternate procedure, the three pavement types are represented by an appropriate continuity factor. The pavement slab thickness will vary with this
Figure D.4-6 Nomograph for Determining the Bar Spacing Design
continuity factor, with a lower value resulting in lesser slab thickness. For this example, a value of 3.2 is assumed to be representative of jointed pavements, both with and without reinforcement, and for continuously reinforced concrete pavement (CRCP).

The concrete properties required for pavement slab thickness determination are modulus of elasticity and flexural strength. For this example the average modulus of elasticity, as determined from a series of static compression tests, is assumed to be 4,000,000 psi (27.6GPa) and the flexural strength, as determined according to AASHTO Designation T-97, is assumed to be 600 psi (4.1kPa).

The composite modulus of subgrade reaction \( k_c \) for design is a function of:

1. The \( k \)-value of the subgrade.
2. The type of subbase.
3. The thickness of subbase.

For this example, it is assumed that the subgrade soil may be classified as poor, with a \( k \)-value of 125 pci. Six inches of a high-quality cement-treated subbase (modulus of elasticity \( E \) of 1,000,000 psi (6.9GPa)) is also assumed. To obtain the design \( k \)-value (at the surface of the subbase), Figure D.4-1 is entered for the subbase thickness of 6 inches (152mm), \( E \) of 1,000,000 (6.9GPa), and subgrade \( k \)-value of 125, to obtain a design \( k_c \)-value of 330 pci.

To determine the pavement slab thickness, the design chart (Figure D.4-2) is entered with the following values for each scale, as determined above:

1. Traffic \( 10,000,000 \) total (1,370 daily) equivalent 18-kip single-axle loads
2. Serviceability Index 2.5
3. Continuity Factor 3.2 (jointed pavements and CRCP)
4. Concrete Working Stress \( 600 \times 0.75 = 450 \) psi \( (3.1 \text{ kPa}) \)
5. \( k_c \)-value 330 psi
6. Modulus of Elasticity \( 4,000,000 \) psi (27.6GPa)

The resulting values of pavement slab thickness \( D \) are 10.0 inches (250mm) for jointed pavements, and for CRCP.

Joints, Load Transfer, and Reinforcement

The pavement slab thicknesses determined above are only one part of the over-all design. Separate consideration must be given to slab dimensions, joints, and design of steel for each of the alternate pavement types.

Nonreinforced Jointed Concrete Pavement

Transverse contraction joints are spaced at specified intervals to minimize cracking outside the joint. For this alternate, joints are assumed to be spaced at intervals of 15 ft and smooth round steel dowels used as load transfer devices. Section 3.4.3.4 (Chapter III) presents the procedure for design of load transfer devices. For this alternate \( D = 10 \) inches the minimum requirements would be:

1. Dowel diameter, inches \( 1\frac{1}{4} \) (45mm)
2. Dowel length, inches 18 (457mm)
3. Dowel spacing, inches 12 (305mm)

Longitudinal joints are used to prevent the formation of irregular longitudinal cracks. For this alternative, longitudinal joints are assumed to be placed along the edge of each 12-ft (3.7m) lane of a 3-lane slab (36-ft [11.0m] width.) Tie bars serve to hold the abutting slabs together and must be designed to withstand tensile forces required to overcome subgrade drag. The selection of percent steel is determined from Figure D.4-4. For this example, the maximum steel percent is determined to equal 0.051%, if \( f_s = 52,500 \text{ psi} (362\text{MPa}) \) and \( F = 1.5 \). However, the maximum is required only at the center of the 36-ft (11.0m) slab width. Figure D.4-5 can be used to reduce the steel percent to correspond to the stresses occurring along the longitudinal joints, where the tie bars are actually placed. As indicated in this figure, the percent steel is reduced from 0.051 to 0.034. For this percentage, tie bars having a cross-sectional area of 0.16 in.\(^2\) (103mm\(^2\)) may be placed at a center-to-center spacing of 33 inches (834mm) (see Figure D.4-6).

**Reinforced Jointed Concrete Pavement**

The purpose of distributed steel reinforcement is not to prevent cracking, but to hold together any cracks that may form. The maximum steel stress will occur at the mid-point of the slab width (for transverse steel) and slab length (for longitudinal steel). Reinforcement is designed for this condition.

For this alternate, the slab length is assumed to be 40 ft (12.2m), the width 36 ft. (11.0m) and the friction factor of the subbase 1.9. Figure D.4-4 is entered for each slab dimension. The resulting steel percentage is 0.072% for longitudinal steel and 0.065% for transverse steel. (It may be economically feasible to reduce the amount of steel in the outside lanes. Figure D.4-5 may be used as a guide to do this.)

The steel percentage determined in Figure D.4-4 is used with Figure D.4-6 to determine the bar size and spacing. If it is assumed that a bar with cross-sectional area of 0.16 in.\(^2\) (103mm\(^2\)) is used in both directions, transverse steel may be spaced 24.5 inches (623mm) and longitudinal steel 22.2 inches (564mm).

**Continuously Reinforced Concrete Pavement**

The purpose of reinforcing steel in CRCP is to hold to a minimum the transverse volume change cracks that occur in the pavement. Longitudinal steel is assumed at 0.6 percent since no adverse experience has shown any need for a change in the steel requirement.

In order to assure minimum crack widths, only deformed bars should be used. Assuming that a No. 6 bar is to be used for the 10 inch slab thickness, the required spacing as determined by Figure D.4-6 is 7 inches.

For the design of transverse steel, the procedure used for jointed reinforced concrete pavements is also applicable for CRCP. Transverse bars may be included or omitted. In those cases where transverse steel also performs the function of tiebars, omission of transverse steel would call for the use of tie bars.
7-4.0 RIGID PAVEMENT DESIGN

7-4.01 General

A rigid pavement structure normally consists of two layers—the pavement slab and the subbase course. The design procedure includes the determination of the thickness of the slab and subbase and the treatment of subgrade soils. Also included is the design of the joints and reinforcement steel in the slab.

Design considerations that are essential to satisfactory performance and long life of a rigid pavement are (1) reasonably uniform support for the pavement; (2) the elimination of pumping by use of a treated or untreated base course; (3) adequate joint design; and (4) a thickness that will keep load stress within safe limits. The overall objective is to determine the minimum thickness that will give the least annual cost.

The material in this section sets forth the policy procedures of Mn/DOT with regard to the design of rigid pavements, including steel reinforcement, joints and special requirements.

7-4.02 Policy

Rigid pavement structural design is based on the tractor semi-trailer (TST) component of heavy commercial average daily traffic and/or Concrete Equivalent Single Axle Load (CESAL) over a 20-year period and the evaluation of the soil subgrade support value based upon AASHTO soil classification. Design will be based on 1-way traffic volume and design-lane TST and/or CESAL. The estimated service life for rigid pavements is 35 years. During this time all transverse joints are to be resealed at approximately 17½ years. These values are an average which is based on historic information. It is used by the Pavement Selection Committee. The actual practice is dependent on observed conditions in the field. At 5 years, a 2-ft wide bituminous shoulder wedge will be placed next to the pavement on both sides of a 2-lane highway and on the outside only of multi-lane roadways. This should be specified for bituminous shoulders only.

There are two basic joint spacing designs (both skewed) that are currently being used for rigid pavement:

**Design “A”**—Roads that have concrete mainline and bituminous shoulders with skewed joints at 27-ft spacing. They are reinforced and have dowels. (See Standard Plan Sheet 5-297.217 and Standard Plate 1019)

**Design “B”**—Roads that have concrete mainline and bituminous shoulders with skewed joints at 15-ft effective spacing. They are non-reinforced and may or may not have dowels at the joints. (See Standard Plan Sheet 5-297.217 and Standard Plate 1016).

Surfacing plans should be simple and explicit as to the designer's intent. Pavement details should not be obscured with distractive and irrelevant details such as meaningless topography and alignment data. The function of pavement sheets is to show type of pavement, type of reinforcement, joints, pavement widths, medians, curb and gutter and the like. Proper and clear identification will eliminate most inquiries and confusion. Where it is impossible to avoid some congestion, use enlargements and specific details.

Occasionally to facilitate construction, panel and joint layouts as shown in the plans require changing in the field. A note should be included in the plan on the first paving layout sheet as follows: “Panel layouts and joint types in this plan may be changed if approved by the Engineer.” The Engineer should contact the Concrete Engineer's Office for approval of the change.

7-4.03 Design Procedure

The design procedure includes the determination of the thickness of the pavement slabs, and design of joints and steel reinforcement by the Concrete Engineer. Also included are recommendations as to treatment of subgrade soils and the type and thickness of base required by the Subgrade and Base Design Engineer.

The general objective of the design procedure is to determine the minimum thickness that will give the least annual cost.

For rigid pavement design a determination will be made of the following items:

1. pavement thickness;
2. base type and thickness;
3. expansion joints;
4. contraction joints;
5. longitudinal joints; and
6. reinforcement steel.

**7-4.03.01 Pavement Thickness**

The thickness of the mainline roadway is determined by the value of modulus of subgrade reaction (K) and the TST and/or CESAL volume of the 20-year projected ADT. Values for TST and/or CESAL, ADT, and 1-way design
7-4.0 RIGID PAVEMENT DESIGN

7-4.01 General

A rigid pavement structure normally consists of two layers—the pavement slab and the subbase course. The design procedure includes the determination of the thickness of the slab and subbase and the treatment of subgrade soils. Also included is the design of the joints and reinforcement steel in the slab.

Design considerations that are essential to satisfactory performance and long life of a rigid pavement are (1) reasonably uniform support for the pavement; (2) the elimination of pumping by use of a treated or untreated base course; (3) adequate joint design; and (4) a thickness that will keep load stress within safe limits. The overall objective is to determine the minimum thickness that will give the least annual cost.

The material in this section sets forth the policy procedures of Mn/DOT with regard to the design of rigid pavements, including steel reinforcement, joints and special requirements.

7-4.02 Policy

Rigid pavement structural design is based on the tractor semi-trailer (TST) component of heavy commercial average daily traffic and/or Concrete Equivalent Single Axle Load (CESAL) over a 20-year period and the evaluation of the soil subgrade support value based upon AASHTO soil classification. Design will be based on 1-way traffic volume and design-lane TST and/or CESAL. The estimated service life for rigid pavements is 35 years. During this time all transverse joints are to be resealed at approximately 17½ years. These values are an average which is based on historic information. It is used by the Pavement Selection Committee. The actual practice is dependent on observed conditions in the field. At 5 years, a 2-ft wide bituminous shoulder wedge will be placed next to the pavement on both sides of a 2-lane highway and on the outside only of multi-lane roadways. This should be specified for bituminous shoulders only.

There are two basic joint spacing designs (both skewed) that are currently being used for rigid pavement:

Design “A” — Roads that have concrete mainline and Bituminous shoulders with skewed joints at 27-ft spacing. They are reinforced and have dowels. (See Standard Plan Sheet 5-297.217 and Standard Plate 1019)

Design “B” — Roads that have concrete mainline and bituminous shoulders with skewed joints at 15-ft effective spacing. They are non-reinforced and may or may not have dowels at the joints. (See Standard Plan Sheet 5-297.217 and Standard Plate 1016).

Surfacing plans should be simple and explicit as to the designer’s intent. Pavement details should not be obscured with distracting and irrelevant details such as meaningless topography and alignment data. The function of pavement sheets is to show type of pavement, type of reinforcement, joints, pavement widths, medians, curb, curb and gutter and the like. Proper and clear identification will eliminate most inquiries and confusion. Where it is impossible to avoid some congestion, use enlargements and specific details.

Occasionally to facilitate construction, panel and joint layouts as shown in the plans require changing in the field. A note should be included in the plan on the first paving layout sheet as follows: “Panel layouts and joint types in this plan may be changed if approved by the Engineer.” The Engineer should contact the Concrete Engineer’s Office for approval of the change.

7-4.03 Design Procedure

The design procedure includes the determination of the thickness of the pavement slabs, and design of joints and steel reinforcement by the Concrete Engineer. Also included are recommendations as to treatment of subgrade soils and the type and thickness of base required by the Subgrade and Base Design Engineer.

The general objective of the design procedure is to determine the minimum thickness that will give the least annual cost.

For rigid pavement design a determination will be made of the following items:

1. pavement thickness;
2. base type and thickness;
3. expansion joints;
4. contraction joints;
5. longitudinal joints; and
6. reinforcement steel.

7-4.03.01 Pavement Thickness

The thickness of the mainline roadway is determined by the value of modulus of subgrade reaction (K) and the TST and/or CESAL volume of the 20-year projected ADT. Values for TST and/or CESAL, ADT, and 1-way design
Notes:
1. See Section 7-6.0 for shoulder design.
2. See Section 4-2.01 for roadbed subcuts.
4. On 4-lane highways, the left shoulder width is 4 ft.; on 6-lane highways, the left shoulder width is 10 ft.

RIGID PAVEMENT DESIGN - INTERSTATE HIGHWAYS AND HIGHWAYS
OVER 1,000 ADT OR OVER 200 DHV (1-WAY)
Figure 7-4.03A

RIGID PAVEMENT DESIGN - HIGHWAYS UNDER 1,000 ADT AND UNDER
200 DHV (1-WAY)
Figure 7-4.03B
INSET R-1
(For Soil Classes A1a and Stable A3)
Stabilizing Aggregate - Spec. 2105 (30' wide) - incorporated in subgrade at a rate of 150 lb./yd.² or as recommended by the District Materials and/or Soils Engineer in conjunction with the C.O. Soils Section.

INSET R-2
(For Soil Classes A1b and A2)
3" Aggregate Base - Class 5 - Spec. 2211 (30' wide) or stabilizing Aggregate - Spec. 2105 (30' wide) incorporated in subgrade at a rate of 150 lb./yd.² or as recommended by the District Materials and/or Soils Engineer in conjunction with the C.O. Soils Section.

INSET R-3
(For Soil Class Unstable A3)
3" Aggregate Base - Class 5 - Spec. 2211 (30' wide) and stabilizing Aggregate - Spec. 2105 (30' wide) incorporated in subgrade at a rate of 150 lb./yd.² or as recommended by the District Materials and/or Soils Engineer in conjunction with the C.O. Soils Section.

INSET R-4
(For Soil Classes A4, A6 and A7)
5" Aggregate Base, Class 5 - Spec. 2211 (30' wide) or may be changed to 2" aggregate base - Class 5 and 3" aggregate base - Class 3 when recommended by the District Materials and/or Soils Engineer in conjunction with the C.O. Soils Section.

INSET R-5
(For Soil Classes A4, A6 or A7 when projected 1-way design-lane TST < 40)

Note: See Table 7-4.03A for pavement thickness.

RIGID PAVEMENT DESIGN - INSETS
Figure 7-4.03C
excavation will be required and subcuts designed to correct these conditions. Requirements for designing compaction subcuts and subgrade corrections on mainline roadway and ramps and loops are found in Section 4-2.01.

Figure 7-4.03C sets typical base thicknesses for five subgrade types that can be used. Other subgrade types which use local materials and give satisfactory results may be used at the discretion of the District Materials and/or Soils Engineer in conjunction with the Central Office Soils Section.

7-4.03.03 Reinforcement Requirements

The purpose of distributed steel reinforcement is not to prevent cracking, but rather to hold tightly closed any cracks that may form, thus maintaining the pavement as an integral structural unit.

No wire reinforcement is required for "Design B" where the panel length is 15 ft except where the pavement crosses a culvert or as determined by the District Soils Engineer. In such cases bar reinforcement will be designed in accordance with Standard Plate Number 1070. For "Design A" where panel lengths are 27 ft, wire reinforcement is required for all thicknesses. For 27-ft reinforced panels over culverts either bar reinforcement or wire mesh fabric can be used.

Reinforcement requirements for both Design A and B are given in Table 7-4.03B. For special conditions where additional reinforcement may be required, supplemental steel will be placed in accordance with Standard Plan 5-297 .217.

A discussion of the criteria used in determining the cross-sectional area of steel required for reinforced concrete pavement can be found in the AASHTO Interim Guide for Design of Pavement Structures, 1972.

### Table 7-4.03B

#### RIGID PAVEMENT REINFORCEMENT

<table>
<thead>
<tr>
<th>15' EFFECTIVE PANELS FOR ALL ADT'S (DIVIDED OR SINGLE ROADWAY)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PAVEMENT THICKNESS</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>6&quot; - 8&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>27' PANELS FOR OVER 1000 ADT (DIVIDED OR SINGLE ROADWAY)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PAVEMENT THICKNESS</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>≤ 8&quot;</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>≤ 9&quot;</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>≤ 10&quot;</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

Note: All reinforcement and tie bars shall meet the requirements of Grade 60 for AASHTO M-31 or M-53.
7-4.04 Warrants for Reinforced Panels

Reinforced panels are used to minimize cracking of concrete pavement at culverts or other locations where subgrade conditions warrant. Generally, the cracking at these locations is due either to settlement of the soils adjacent to the culvert or movement of the culvert and soil because of frost action.

Reinforced panels over a culvert shall be required where the height of fill (HF) from the top of the culvert to the bottom of the pavement is 10 ft or less.

In addition to the location of reinforced panels determined by the above criteria, the District Soils Engineer may recommend additional sites for the use of reinforced panels. These recommendations may be incorporated into his "Soils Recommendations" or special recommendations made after reviewing the project when grading has been completed.

7-4.05 Ramps and Loops

Structural design of rigid pavement for ramps and loops will be as determined by the Concrete Engineering Section.

7-4.05.01 General Considerations

The pavement type on the ramps and loops should be the same as that of the mainline roadway. Pavement design will be based on the CESAL and/or TST component of the HCADT volume.

The cross-section elements for the acceleration lanes, deceleration lanes, escape lanes, auxiliary lanes, directional connections, and gore areas adjacent to the through lanes shall be the same as the mainline. The transverse joints in ramp pavement to the gore point shall coincide with the joints in the mainline.

For typical ramp and loop cross sections, see Section 6-3.04.04 of Chapter Six, Interchanges. For structural design of ramp and loop shoulders refer to Section 7-6.0, Shoulder Structural Design.

7-4.05.02 Base and Soils

The Soils Engineer will recommend base depths and aggregate classes for pavement and shoulders based on soil type and modulus of subgrade reaction. As with the mainline roadway it may be necessary to replace unacceptable subgrade material. In such cases subcuts and subgrade corrections for ramps and loops will be made according to Section 4-2.01.

7-4.05.03 Pavement Width and Placement

For the purpose of pavement design, ramps and loops are classified rural or urban in the same manner as mainline roadways. Rural design is identified by the absence of curbs. Urban design contains curbs, and can be in both urban and rural areas as appropriate. Section 6-3.0 of Chapter Six identifies the basic criteria for the use of urban or rural designs for ramps and loops.

The concrete pavement on ramps and loops intersecting a bituminous crossroad or street shall extend to the edge of the through lane of the crossroad or street. Pavement widths on ramps and loops should be designed in whole-foot increments from back-to-back of integrant curbs. When design B or D Integrant Curb is used, the inches (fractions of a foot) shall be added to or subtracted from the traffic lane width to arrive at a total pavement width in whole-foot increments. Pavement widths on auxiliary lanes with integrant curb may be designed in increments of feet and inches; e.g., a 12-ft lane with a B6 Integrant Curb would have a total pavement width of 12-8 ft in. To facilitate construction and keep costs at a minimum, every effort should be made to provide for uniform widths for ramps and loops, and variable pavement widths should be kept to a minimum.

The designer should check the final design against the layout and review with Preliminary Design and Traffic Engineering to include the latest concepts in complex areas. Changes from an approved layout must be reviewed by the Preliminary Design Engineer who may approve minor changes and arrange for Staff Approval of significant changes.

7-4.05.04 Pavement Thickness

Table 7-4.05A shall be used for the design of pavement for ramps and loops. Panel lengths shall be either 27 ft or 15 ft effective depending on traffic volume. Supplemental reinforcement shall be placed in all panels with pavement widths between 16 and 22 ft.

Requirements for joints and reinforcement can be found in Standard Plate 1015 - Concrete Pavement Without Longitudinal Joints and Standard Plans 5-297.200 series.

7-4.05.05 Storage Areas

Figure 7-4.05A provides details for both reinforced and non-reinforced concrete pavement sections of ramps with a storage area. The normal rural ramp width of 16 ft is tapered out to two 12-ft lanes at the intersection of the crossroad. For urban loops the normal width is 18 ft. The center joint will be an L1T for rural design. For urban design the joint will be an L2KT. Curb design will be...
### Table 7-4.05A

<table>
<thead>
<tr>
<th>TST-HCADT</th>
<th>PAV'T. THICK</th>
<th>PAV'T WIDTH</th>
<th>PANEL LENGTH</th>
<th>FABRIC REINF.</th>
<th>SUPPLE. REINF.</th>
<th>SKEWED CONST. JOINTS</th>
<th>LONG JOINT</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 TO 150</td>
<td>8&quot;</td>
<td>0 THRU 16'</td>
<td>15' EFFECTIVE</td>
<td>NONE</td>
<td>NO</td>
<td>C5-1</td>
<td>NONE</td>
</tr>
<tr>
<td>0 TO 150</td>
<td>8&quot;</td>
<td>OVER 16' THRU 22'</td>
<td>15' EFFECTIVE</td>
<td>NONE</td>
<td>YES</td>
<td>C5-1</td>
<td>YES</td>
</tr>
<tr>
<td>0 TO 150</td>
<td>8&quot;</td>
<td>OVER 22'</td>
<td>27'</td>
<td>NONE</td>
<td>NO</td>
<td>C5-1</td>
<td>YES</td>
</tr>
<tr>
<td>OVER 150</td>
<td>8&quot;</td>
<td>0 THRU 16'</td>
<td>27'</td>
<td>YES</td>
<td>NO</td>
<td>C1D</td>
<td>NONE</td>
</tr>
<tr>
<td>OVER 150</td>
<td>8&quot;</td>
<td>OVER 16' THRU 22'</td>
<td>27'</td>
<td>YES</td>
<td>YES</td>
<td>C1D</td>
<td>YES</td>
</tr>
<tr>
<td>OVER 150</td>
<td>8&quot;</td>
<td>OVER 22'</td>
<td>27'</td>
<td>YES</td>
<td>NO</td>
<td>C1D</td>
<td>YES</td>
</tr>
<tr>
<td>FAI TO FAI</td>
<td>6</td>
<td>0 THRU 16'</td>
<td>27'</td>
<td>YES</td>
<td>NO</td>
<td>C1D</td>
<td>NONE</td>
</tr>
<tr>
<td>FAI TO FAI</td>
<td>6</td>
<td>OVER 16' THRU 22'</td>
<td>27'</td>
<td>YES</td>
<td>YES</td>
<td>C1D</td>
<td>YES</td>
</tr>
<tr>
<td>FAI TO FAI</td>
<td>6</td>
<td>OVER 22'</td>
<td></td>
<td>YES</td>
<td>NO</td>
<td>C1D</td>
<td>YES</td>
</tr>
</tbody>
</table>

**NOTES:**

1. Tractor semi-trailer, heavy-commercial average daily traffic (includes only traffic categories 3, 4 and 5 of traffic assignment), based on one-way traffic volume.

2. If pavement thickness 8" use 12 x 12 D5.8 x W4.0 of 12 x 12 W6.2 x W4.0
   - If pavement thickness 9" use 12 x 12 D6.5 x W5.0 or 12 x 12 W7.0 x W4.0
   - If pavement thickness 10' use 12 x 12 D7.2 x W4.0 or 12 x 12 W7.8 x W4.0

3. For widths greater than 15' a L1T joint should be provided 4' from the edge of ramp on the drivers left. Tie bar size and spacing shall be as follows:
   - 15' Effective Panels
     - No. 4 x 2' 6" Tie Bars spaced at 3' 0" c. to c. for ≤ 8"  
     - 27' Panels
     - No. 4 x 2' 6" Tie Bars spaced at 3' 0" c. to c. for ≤ 8"  
     - No. 4 x 2' 6" Tie Bars spaced at 2' 6" c. to c. for ≤ 9"  
     - No. 5 x 3' 0" Tie Bars spaced at 3' 0" c. to c. for ≤ 10"

4. Supplemental reinforcement shall consist of 5 No. 4 reinforcement bars spaced at 4-ft centers (2 ft from C1 joints, and placed at the same skew as the C1 joints.)

5. Supplemental reinforcement shall consist of 7 No. 4 reinforcement bars spaced at 4-ft centers and placed at the same skew as the C1D joints.

6. Same as thinnest mainline concrete pavement.
RIGHT TURN WIDENING

1. Place two # tie bars at end of L1T joint.
2. Reinforcement shall be in accordance with Standard Plan 5-297.217.

RIGID PAVEMENT DESIGN FOR WIDENED RAMPS AND LOOPS WITH SKEWED JOINTS

Figure 7-4.05A
D424 and will be carried through the tapered section. In the tapered section the joint can be terminated at a point where the taper width equals 10 ft or extended to a point near the end of the taper reaching 4 ft as noted in the figure. This area shall contain supplemental reinforcement for both 15-ft and 27-ft panels.

7-4.06 Bridge Approach Treatments and Panels

7-4.06.01 General

The design of bridge approach treatments and approach panels are the responsibility of the Road Design Unit. Approaches are broken down into two groups depending upon whether the structure is designed for low abutments or high abutments. Normally bridge approach treatments will be provided at every bridge that has bridge approach panels unless recommendations to the contrary are provided by the District Soils and/or Materials Engineers in conjunction with the Central Office Soils Units. Discussion of the use and design of bridge approach panels is found in Section 7-4.06.03.

7-4.06.02 Low Abutments

A low abutment is defined as an abutment whose height is less than 15 ft measured from the top of the deck to the bottom of the footing. Details for this design are shown in Figure 7-4.06A and also in Standard Plan Sheet 5-297.225. For low abutments, the following requirements must be met:

1. When the grading, paving and bridge are all under one contract, the same sequence of construction as shown on the standard plan sheet should be followed (i.e., zone Ω should be backfilled after construction of the bridge).

2. Perforated drainage pipe may be required in zone ález when recommended by the District Materials and/or Soils Engineer. This will normally only be required where seepage through the expansion device at the bridge end may be a problem.

3. On replacement bridges where abutment locations are close to the locations of the old abutments (settlements of existing approach fills and subsoils have already occurred) only zone Ω construction, including possible drainage pipe, will be required.

4. The slope between the grading material and zone ₁ should be 3:1 or flatter so that the break point to the 20:1 slope falls behind the sill.

5. Temporary drainage is required during construction of the abutment and zone Ω.

6. Special recommendations for erosion protection of the end slope when the treatment is used at a river bridge should be requested from the District Materials Engineer in conjunction with the Central Office Soils Unit.

7-4.06.03 High Abutments

A high abutment is defined as an abutment whose height is 15 ft or greater measured from the deck to the bottom of the footing. Details for this design are shown in Figure 7-4.06B and also in Standard Plan Sheet 5-297.226. For high abutments, the following requirements must be met:

1. When the grading, paving and bridge are all under one contract, the same sequence of construction as shown on the standard plan sheet should be followed (i.e., zone ₃ should be backfilled after construction of the bridge).

2. On replacement bridges even where the new abutments are located at the old abutments, the full treatment, according to Standard Plan Sheet 5-297.266, should be provided. This will allow the most favorable soil pressure conditions for the wall design of the high abutment.

3. The slope between the grading material and zone ₁ should be 2:1 or flatter so that the break point to the 20:1 slope falls behind the sill.

4. The 4-in. perforated pipe may be located on top of the abutment footing rather than as shown on Figure 7-4.06B if drainage can be provided through the wing walls.

5. Temporary drainage is required during construction of the abutment and zone ₃.

6. Special recommendations for erosion protection of the end slope when the treatment is used at a river bridge should be requested from the District Materials Engineer in conjunction with the Central Office Soils Unit.

7-4.06.04 Bridge Approach Panels

Bridge approach panels are used to connect the pavement structure with the bridge. They are provided for both rigid and flexible pavement conditions. Bridge approach panels are to be constructed under the paving contract.

Bridge approach panels are detailed on Standard Plan Sheets 5-297.223 and 5-297.224 for concrete and bituminous pavements respectively. The panel is designed with a 4-in. pressure relief joint and passive soil pressure keys.
APPENDIX C

FIELD TRIP REPORT

(See Instructions on reverse)

<table>
<thead>
<tr>
<th>TO</th>
<th>FROM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mr. Ronald E. Heinz</td>
<td>Leon M. Noel</td>
</tr>
<tr>
<td>Chief, Highway Design Division</td>
<td>Chief, Pavement Branch</td>
</tr>
</tbody>
</table>

INCLUSIVE DATES

From July 24, 1984 To July 26, 1984

ITINERARY

Washington, D.C., to St. Paul, Minnesota, and vicinity and return.

PURPOSE

To consult with the Division and State at their request about Portland Cement Concrete procedural changes being considered by the Minnesota Concrete Design Committee.

PRINCIPAL CONTACTS

See Attachment

ACCOMPLISHMENTS OR RESULTS

See Attachment

SUBSEQUENT ACTIONS TAKEN

None

RECOMMENDATIONS

See Attachment

OTHER PERTINENT ITEMS
ACCOMPLISHMENTS/RESULTS AND RECOMMENDATIONS

Upon arrival in St. Paul, representatives of the Division and I went directly to a weigh-in-motion installation on I-494 where State personnel demonstrated the capabilities of this equipment. Both lanes of a two-lane roadway were instrumented to provide speed data, truck axle configuration, and weights on a continuing basis. The equipment was from the "Advanced Digital Engineering Corporation" and was developed in Canada. The speed and truck data are automatically recorded by a computer housed at the outer edge of the ROW. The computer was programmed to provide equivalency factors, summaries, etc., and could be unloaded into central office equipment via telephone. This is a very impressive installation costing $250,000 and is undoubtedly what is needed to get good truck weight information. The State people reported that truck weight applications secured by this equipment were much greater than those secured by the normal truck weighing operations used by Planning in the past. As a result, records of actual truck applications and projected applications should be much improved.

The remaining 2 days were spent looking at various pavement designs in the field, reviewing pavement activities underway in the State, and then discussing with the State/Division Design Review Committee those specific rigid pavement design issues the committee had identified for review.

I think the organization the State has for handling pavements is somewhat unique and appears quite effective. For example, there is a rigid pavement engineer who has responsibilities for design, construction, and maintenance activities related to rigid pavements. There is a parallel flexible pavement engineer and
then an engineer responsible for subgrade and base design. The significant point is that one engineer can concentrate on one pavement type and develop compatible design, construction, and maintenance strategies. The end result of this process is that the State is getting excellent performance from its rigid pavements. They believe they can get in excess of 30 years life and no doubt they are getting excellent performance.

We spent 1 day looking at various pavement designs and pavement conditions in the Minneapolis-St. Paul area and in southern Minnesota. For heavy duty pavements, their design has evolved to provide for a 27-foot panel length, dowelled joints with neoprene seal, tied longitudinal joints (except when three or more lanes, in which case one joint is left untied) with rubberized asphalt seal, and widened lanes (13 or 14 feet) with rumble strips to keep trucks off the pavement edge. The ride quality, of particularly the new pavements, is excellent which the State attributes to a new rideability specification being used. The base or subbase used is an unstabilized gravel subbase with limited fines which is daylighted. One interesting aspect which may be highly significant in the good performance they experience is that the grade line is high and the ditch line is about 5 feet below the surface of the pavement. This provides for good drainage although it may be somewhat at cross purposes with the desire for flat slopes for safety. Also, the maintenance of joints is good which has the effect of resisting water entry into the pavement structure. The State’s limited experimental use of stabilized bases has not shown very good performance.

The State has made significant progress in implementing the Concrete Pavement Evaluation System (COPES) for its Interstate and primary highway systems. Data are being collected by survey crews using hand held equipment which can then be dumped automatically into the computer (IBM-PC) by modem. Data coding and manipulation is minimized. The COPES approach is extremely impressive and should provide for the kind of analysis to evaluate various design approaches. If at all possible, it would be desirable to have someone from Minnesota attend our LTM Conference in October 1984 to explain their COPES activities.

The primary purpose of my trip was to consult with the Division and Minnesota DOT on Portland Cement Concrete Pavement issues and changes in State procedures being considered, some of which were prompted by Research’s studies of the benefits of slab edge strengthening by widened lanes or concrete shoulders, the beneficial effect of frozen subgrade support, and new equivalencies derived for tridem axles.

I discussed the development of the new Pavement Design Guide being done by the AASHTO Joint Task Force on Pavements, the background leading to this effort, and the schedule for completion which calls for a draft of the new Guide by January 1985. Issues under consideration by MnDOT are also under consideration by AASHTO and I suggested that significant procedural changes by the State at this time were premature. Likewise, the State’s pavement performance data being collected under COPES and from historical records will be available for analyzing about February, if all goes well with this effort. This should provide for a much greater in-depth evaluation of actual performance of
various design features. While no definite decision was made at the time, State personnel agreed that the timing for significant changes was not right and they should wait until other efforts are complete. (I subsequently learned that the State postponed any significant changes until 1985.)

On other issues discussed, the State has experienced premature faulting on its medium to low volume, plain undowelled pavements, particularly with quarried limestone and low hardness aggregates. The slab panels in this design are variable with about 15 foot effective spacing. They think soil, traffic, panel length, and the hardness of concrete aggregate used all influence this faulting. In the past, the PCA recommended a level of 150 heavy trucks per day was used as the level above which dowels would be used. The level now proposed is as follows:

<table>
<thead>
<tr>
<th>Heavy TST Trucks (one lane)</th>
<th>Subgrade</th>
<th>Concrete Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>Granular</td>
<td>Good</td>
</tr>
<tr>
<td>100</td>
<td>Plastic/Granular</td>
<td>Good/Soft</td>
</tr>
<tr>
<td>80</td>
<td>Granular or Plastic</td>
<td>Soft</td>
</tr>
</tbody>
</table>

This appears to be a good approach and may be worthy of consideration by others.

The State will continue its research/experimentation with open graded drainable subbases and edge drains with the intent of establishing criteria for their use. They will continue to routinely use gravel/stone subbases which have demonstrated excellent performance.

Finally, there was limited discussion about the need sometime in the future to strengthen the State's pavement life cycle cost analysis model. The model does not now include consideration of maintenance, salvage, and user costs. The State has been a leader in involving industry and refining its pavement type selection procedures. Unfortunately, when the economic analysis is used primarily as the sole basis for selection, pressures are exerted by industry to economize or lessen the cost of their product through cut backs in design. Since we do not have the ability in pavement design to always quantify the benefits of various design features such as dowel bars, it makes equating designs of different materials difficult if not impossible. I see this as a very significant problem for pavement managers in the future. Perhaps the solution to this dilemma is to form a pavement type selection committee or group to review all relevant factors such as those in the AASHTO Guide for Pavement Type Selection (including economic analysis) and arrive at a consensus selection.

As a general overall observation, I was very impressed with the interest, enthusiasm, and efforts of both the State and our Division with respect to the design, rehabilitation, and overall management of pavements. It appears that their efforts are paying off because they seem to be experiencing good pavement performance, concrete performance projected in excess of 30 years, which is the end result the bottom line.
January 10, 1985

Concrete Pavement Design Task Force
Minnesota Department of Transportation
Transportation Building
St. Paul, Minnesota 55155

Re: A Review of Mn/DOT's Concrete Pavement Design Methods

Gentlemen:

We want to first of all thank the Task Force for the opportunity to comment on your review of Mn/DOT's concrete pavement design methods. Your review is occurring at a time when traffic needs and demands are increasing at a very rapid rate and decisions must be made to obtain the best investment for the traveling public. Also the focus is on reconstructing existing highways to meet these increasing needs which brings along new challenges for the highway engineer. The exchange of technology among the highway industry will result in the most efficient way to accomplish our goals to meet the demands on our highway transportation system.

A. TRAFFIC

The construction or reconstruction of a highway project represents a long term investment to meet the transportation needs of the future. The biggest uncertainty in the design of future pavements is projecting what the future traffic demands will be. If we look at the historical records over the past thirty years we have seen an increasing growth of both the volume and allowable load limits. There is nothing today that indicates that this trend will change, and if anything, will continue to increase at a more rapid rate. The Task Force's concern over this matter is expressed in thirty-percent of their recommendations.

Figure 1 shows the truck types and the equivalent loads they are carrying. This information was taken from Mn/DOT's Truck Weight and Classification Studies and indicates the tractor semi-trailer units yield the greatest loads on Minnesota highways. Figure 2 indicates the relationship of tractor semi-trailer units and number of E18 loadings based on present outstate truck distribution.

I believe the most significant recommendation made by the Task Force is the use of a 35 year traffic forecast for the design of new pavements. This recommendation will result in the most economical pavement to meet the transportation needs of the future at the least expense to the traveling public. This recommendation should not be limited to the design of concrete pavements only.
Included with this recommendation was the adjustment for frozen support by multiplying the 35 year traffic forecast by a factor of 0.93. Technically this is correct but seems like a redundant exercise knowing the industries present ability to forecast the 35 year design traffic. It reduces the design thickness by only a 0.10 of an inch and we cannot, as of yet, project traffic that accurately.

Since the traffic forecast is the biggest unknown for any pavement design, we recommend that safety factors be applied to traffic projections rather than material properties and that they apply to the design for both materials.

B. Design

Figure 3 compares three design methods using average daily two-way tractor-semitrailer traffic projected over the conventional 20 year design life. The Minn/DOT curve represents the thickness requirements from Table 7-4.03A in the Road Design Manual attached to your review. The AASHTO,OST curve represents the thickness requirements of the AASHTO Interim Guide for Design of Pavement Structures for the mix of traffic that has been experienced on out state Minnesota highways. The NCR PLAIN curve represents the thickness requirements using the North Central Region's plain pavement performance model. This curve was included because of its extensive 30-year performance data on plain pavements.

Below the 600 TST level the Minn/DOT curve would require thicker slabs for the same traffic than the other two methods. This is reversed above that level. Above the 2000 TST level Minn/DOT would not require pavements thicker than 10" but the other two methods require thicker pavements with increasing loads.

The NCR PLAIN curve requires greater thickness above the 600 TST level than the other two methods. This is due to the necessity of plain pavement to maintain faulting within acceptable limits during the design life of the pavement. Faulting is directly related to the cross-sectional area and spacing of the transverse joints. Joints must be spaced a maximum of fifteen feet to provide load transfer by aggregate interlock and the cross-sectional area, controlled by thickness, must be adequate to efficiently transfer the loads applied. In order for a pavement to successfully perform as a plain pavement it must be designed and built as a plain pavement. This curve also indicates that if slabs are built thinner for the same level of traffic, alternate load transfer systems must be used. If for any reason these alternate load transfer systems fail, initially or after the long term affects of the environment, unacceptable faulting will occur.

Figure 4 compares these same three design methods using a 35 year design life recommended by the Task Force. The 35 year design increases the Minn/DOT thickness about one-half inch and requires the maximum 10" thickness at the 1,500 TST level. The AASHTO thickness is increased an average of 1".
FIG. 3 – PCC THICKNESS vs TST LOADING
20yr DES LIFE, k=150, WK STRESS=500psi

FIG. 4 – PCC THICKNESS vs TST LOADING
35yr DES LIFE, k=150, WK STRESS=500psi
The 35 year design life has its biggest affect on plain pavement design. This thickness along with a maximum joint spacing of 15' is required to maintain faulting within acceptable limits and slabs thinner than recommended by this curve can be expected to fault if the jointing system fails.

Two important conclusions can be drawn when comparing the two design lives. One, the greatest influence for increasing load carrying capacity is the increase in pavement thickness and two, we are reaching the limits of our present design methods when we consider a 35 year design life and the loads we are experiencing on our highways.

C. Joints

We agree with the Task Force conclusion that "Rarely, if ever have concrete pavements in Minnesota failed due to inadequate thickness". The successful performance of concrete pavements, and there are many, is related to the successful performance of the jointing system. Both reinforced and plain pavements.

Joint problems related to the use of "D"-crack susceptible aggregate have been solved. Minn/DOT research has developed solutions that will eliminate this problem in both new and reconstructed highways.

Neither reinforcement or load transfer dowel bars are a factor that allows an increase in the load carrying capability of concrete pavements. There has been a trend in recent years to place doweled joints closer and closer together. This may or may not include the use of distributed reinforcement. This is an effort to try to reduce and control spalling and faulting at joints and mid-panel cracks. There has not been adequate experience to determine whether or not this trend will produce favorable results.

When dowel bars become locked, either initially due to improper construction procedures or after the long term effects of moisture and deicing chemicals, the pavement section must act as a plain pavement. If the distance between the mid-panel cracks exceeds 15' and/or the cross-sectional area is not adequate to carry the loads applied faulting will occur.

Plain pavement jointing systems result in thicker pavements than we have become conventionally accustomed too. They have been successfully used on heavy truck routes and economically compare very favorably to other designs.

We suggest that the panel lengths in non-reinforced pavements be limited to a maximum of 15 feet.

D. Concrete Shoulders

Minn/DOT has been a leader in the research on the benefits of tied concrete shoulders. They improve the safety and maintenance benefits over conventional shoulders while contributing to the strength of the mainline pavements. Figure 5 shows the effect on the thickness of the mainline pavement by reducing the design loadings one-half. This reduction averages one inch. Eliminating this reduction
would not add a lot of additional costs and would provide assurance that the new pavement would be able to carry unanticipated loads in the future. It would also provide the needed thickness to allow for future diamond grinding to improve safety and smoothness while maintaining its structural ability to carry heavy loads.

**FIG. 5 – PCC THICKNESS vs TST LOADING**

35yr DES LIFE, k=150, WK STRESS=500psi

E. Minimum Thickness

The use of a 7" minimum thickness for State Highways assures adequate load carrying capacity on low volume roads while providing assurance that the pavement will carry unforeseeable traffic or provide an extended service life for the design traffic.

Determination of a minimum thickness is best determined by the agency responsible for the pavement's maintenance. While a minimum standard of 7 inches for concrete may be desirable for traffic loads experienced on state highways it may not be the best minimum for local county and city pavements because of different traffic distribution. We recommend that the 7" minimum apply to state highways only.
Minimum thickness for both concrete and asphalt pavements should be based on equivalent load carrying capacity. Based on Minn/DOT's Road Design Manual a minimum of 10.5 inches of asphalt will be required to comply with a 7 inch minimum concrete thickness standard. We recommend that Minn/DOT adopt a 10.5 inch minimum asphalt thickness standard.

F. Concrete Properties

In our region the most critical property of concrete pavements is durability. Minn/DOT research has developed specifications that make concrete resistant to the effects of freeze-thaw and de-icing chemical environments. Recent research of "D"-crack susceptible aggregates in Minnesota has resulted in recommendations that will avoid this problem for future pavements. When we design concrete to be durable in our freezing environment we find we have adequate strength to carry the loads.

Concrete can be designed to meet any predetermined strength requirement necessary and have a very small cone of variation. Your own data can verify this. Since concrete strength is the most predictable factor in pavement design it is not good practice to apply a safety factor to it. We are conserving something of value that has been paid for and not being used.

The AASHTO Design Guide applies a safety factor to concrete but conversely does not to asphalt materials. The asphalt design curve is a performance curve and has a 50-50 chance of failure. The 500 psi working stress for concrete is a realistic value but we recommend using a 750 psi design stress for thickness determinations and apply the safety factor to the largest unknown value - traffic.

Warping and curling is a characteristic of concrete slabs that is anticipated and designed for. Warping and curling is caused by moisture and temperature differences between the top and bottom of the slab. In environments where seasonal temperature differentials will reach 150 degrees-F and wet and dry periods are experienced, warping and curling stresses can be anticipated. In plain pavements these stresses are kept within the limits of the concrete by using a joint spacing in feet equal to twice the thickness of the slab in inches but never exceeding a maximum of 15 feet.

In mesh-dowel pavements reinforcement is placed in the slab to hold the cracks together that are caused by warping and curling stresses.

Warping and curling stresses can be aggravated by a saturated subgrade or subbase that keeps the bottom of the slab continually moist.

G. Subgrade and Subbases

The most important contribution towards the successful performance of pavements is the uniform support of the base, subbase and subgrade. Developing subgrade support values is important for asphalt thickness determinations and acts as a
FIG. 6 - SUBGRADE VALUE RELATIONSHIP

FIG. 7 - PCC THICKNESS vs SUBGRADE VALUE

35yr DES LIFE, WK STRESS=500psi

AAS-k150
AAS-k300
method of measuring the uniformity obtained for slab support. Figure 6 is a graphical representation of the k-value and R-value relationship proposed by the Task Force.

Proper drainage of subsurface water to improve pavement performance represents a considerable investment. Free water in a base that contains fines that will go into suspension can be expected to pump these fines from the base when the two-way truck count reaches 300 TST’s per day. This may result in undesirable faulting occurring in plain pavements and at the midpanel cracks of mesh dowel pavements. Also excessive moisture in the base may accelerate the deterioration of "D"-crack susceptible aggregate and aggravate warping and curling stresses.

If undesirable subsurface water is anticipated a uniform 4' to 6 inch aggregate base with edge drains has been effective in removing this free water. The day lighting of the base to the shoulder or edge drains to assure positive drainage is critical for concrete pavements. Inadequate drainage of the base will create a bathtub for the pavement to sit in and in our adverse environment create more problems then it will solve.

Using "extraordinary means" such as filling deep subcuts with granular materials under concrete pavements is neither desirable nor beneficial and may result in the premature failure of the pavements. Good drainage is critical to the successful performance of a base. Deep subcuts filled with granular material allows for the collection of water that repeatedly creates a drainage problem. Often times deep subcuts would be below the bottom of parallel drainage ditches where the water would be expected to drain.

Concrete pavements are insensitive to subgrade support for structural capacity. Figure 7 shows the affect on the concrete thickness when the subgrade k value is doubled. This increase allows for less than a one-half inch reduction in concrete thickness. Figure 8 compares the load carrying ability of both concrete and bituminous pavements with the increasing thickness of the subbase, As can be seen from the graph, increasing the subbase thickness under a 8 inch pcc pavement to 12 inches provides only a 14% increase in load carrying capability. By contrast the increased subbase greatly adds to the load carrying capability of the bituminous pavement. We know increasing the slab thickness from 8" to 9" will double the load carrying ability and extend the service life of the concrete pavement.

Kinn/DOT’s Road Design Manual indicates however, that deep granular bases under bituminous pavements offers a structural benefit. Adequate drainage of deep subbases is still a critical concern to assure expected performance.

Deep subcuts filled with granular material should not be designed or built under concrete pavements. If it is felt they are needed for bituminous designs to make them perform they should be considered as part of the bituminous pavement structure and the costs included in the pavement type determination for the bituminous alternate.
M. Conclusion

Historical records indicate to us that we should be more conservative in our traffic projections for the future. Minnesota's use of a 35 year design life is a positive step in that direction.

We would suggest that the safety factor for design be removed from the concrete equation and be applied to traffic and used in the design of both pavement types.

Recommended plain pavement joint spacing in feet is equal to twice the thickness of the pavement in inches but never exceeding a maximum of 15 feet.

Minimum design standards should apply to all pavement alternates and based on load carry capacity. Adoption of a 7 inch minimum standard for concrete on state highways in Minnesota requires asphalt to be a minimum of 10.5 inches.

Our transportation demands are reaching the limits of our present design methods. This illustrates the need for continued monitoring of pavement performance and updating design procedures to meet the transportation needs on the highway system. Minn/DOT has accomplished this for concrete pavements through the efforts of this Task Force. This same effort should be made for asphalt pavement design as well.

Minn/DOT research has provided the entire concrete highway industry with mix designs that will prevent the "D"-cracking phenomena from occurring in future PCC pavements.
These thick granular subbases are not needed under concrete pavements for load-carrying capacity. If used they can materially add to the problem of maintaining positive constant drainage in subbases to the detriment of pavement performance of either concrete or asphalt. This is particularly true in our environment where the freezing of water in deep open-graded subgrade materials becomes a constant possibility and a threat to performance. This added problem of drainage requirements takes on special importance where pavements might be built below existing grade.

Except in a few cases where excessive non-uniform subgrades were present or "D"-cracking aggregates were used concrete pavements have carried the load and performed extremely well.

Minn/DOT has been a leader in updating pavement designs in order to obtain the best investment for the traveling public. Upgrading concrete pavements to meet future demands is a good step forward. Our industry feels that alternate pavement designs should also be upgraded to meet the same demands that are being placed on concrete. This is important if the Minnesota tax payer is going to realize the greatest benefit from his tax dollar.

Sincerely,

Norm Nelson
Regional Paving Engineer
North Central Region
PORTLAND CEMENT ASSOCIATION
February 22, 1985

Minnesota Department of Transportation
Concrete Pavement Design Task Force

Re: A Review of Mn/DOT's Concrete Pavement Design Methods

Gentleman:

We would like to thank the Task Force and the Department for the opportunity to comment on the findings of your Review of Mn/DOT's Concrete Pavement Designs. We commend your efforts in looking for the best possible Concrete Pavement Designs, balancing the engineering, safety, and economic factors. The Concrete Paving Industry is experiencing a resurgence on both the national and local level, and we must strive to build the best possible pavements that we can.

Leo Warren and the rest of the Concrete Office should be proud of their accomplishment over the last few years. The development of the thinner, widened section, and Concrete Pavement Restoration techniques have given the driving public a much better return on their highway investment, and we appreciate the opportunity we have had to give our input.

We would now like to address the issues in order:

DESIGN VARIABLES

The present thickness design formulas, based on the AASHTO Interim Guide appear to be functioning, as evidenced by the lack of structural failure in existing pavements, and we question the need to adopt a more conservative working stress of 500 psi. The safety factor or C, of 1.33 results in a load safety factor of 2.67, versus the asphalt load safety factor of 1.0; and we feel that the present value of 525 psi, or even a higher value, should be used pending the results of test data and the new AASHTO design revisions, which are expected to take the safety factor out of the modulus of rupture entirely.
We agree with use of the R-values determined for the bituminous design, to derive a K-value for the concrete design. This takes one more assumption out of the design, and puts the two designs on equal footing in relation to subgrade values.

TRAFFIC

We believe that inaccurate traffic forecasting, has been the biggest factor in the premature deterioration of pavements, both concrete and bituminous, and support the Task Force recommendations wholeheartedly. Central monitoring and approval by the Traffic Forecasting Unit, along with further training, are a big step in the right direction. It is a very crucial factor in pavement performance and additional expertise is needed. The ever increasing truck weights and numbers, make it even more critical. We would also like to see the safety factors taken out of the other variables, and applied to the traffic projections, for both concrete and bituminous designs. This would result in more equivalent designs.

DOWEL BARS

Due to the above mentioned problems with traffic forecasts, and the increasing truck volumes and weights, we support the lowered requirements for doweled joints. The lack of dowels on some projects where the TST count was underestimated, has led to premature faulting, and given our industry a black eye.

UNTIED THIRD LANE

Although the national trend is toward tying all lanes and shoulders together, leaving the third or inside lane untied, doesn't appear to be a problem in Minnesota. We feel that the situation should be monitored, and further information be sought as it becomes available. If there is no problem with longitudinal cracks in pavements which are tied in other states, it should be considered in Minnesota.

DRAINAGE

Drainage appears to be the biggest unknown in pavement and subgrade design. We support the recommendation for more study. The 4 foot subcut with granular backfill, that has been designed into reconstruction jobs, has not been proven as an economically viable design when you look at the concrete thickness designs; and benefits the bituminous alternate to a much greater degree. We feel that better drainage is needed, but the subcut and granular backfill, is not the best solution. It creates a "bathtub" under the pavement and necessitates a complex, functioning subdrain
system, which can be prone to plugging, breaks, etc. Edge drains and open graded bases appear to be more promising alternates.

**WIDENED SECTION / TIED CONCRETE SHOULDERS**

The widened section (27 foot) and tied concrete shoulders offer many benefits, and we agree with their adoption as the standard. We feel that concrete shoulders should be used more, but they can not compete with the rural bituminous shoulder designs economically. The existing bituminous design is not sufficient to handle the loads, and should be strengthened; but consideration should also be given to adding concrete shoulders to a project that has been determined as concrete with bituminous shoulders, as an engineering decision, even if it is not the cheapest alternate. The additional money would be well spent. The 7 inch minimum thickness for state highways also is a good idea, considering the unknowns in design, i.e. traffic, truck weights, etc.

**MATERIALS**

Materials have been a problem in many past projects, but the recent advances in joint sealants, and the recognition of the D-cracking problems and subsequent tightening of the aggregate specs, should prevent most of the problems from reoccurring. The Department should be commended for its efforts in implementing the COPE's program. This should result in further refinement of material and construction specs.

**JOINTING AND REINFORCING**

Most of the state highway concrete pavements built in the last few years have utilized the 27 foot joint spacing, mesh-dowel design; and they seem to perform very well. With the lowered dowel requirements, this will virtually be the only design we build, and we don't have any reservations about it. The present non-reinforced 15 foot effective joint spacing design, also seems to perform very well in the absence of high TST's. We also support the continued use of skewed joints.

**CONCLUSIONS**

To sum up, we agree with the Task Force's recommendations for the most part, and feel that the Review is a very good idea. We would like to suggest that the same type of review be undertaken with the Mn/DOT bituminous designs, and the safety factors be equalized between the two alternates. In the absence of changing the presents designs, we would like to see the new AASHTO Design
Guide adopted as soon as it becomes available. Our greatest immediate concern is the subcut and granular backfill, issue on both reconstruction and new construction. We would like to see some guidelines for its use, and have the opportunity to comment on them. There doesn't appear to be any underlining reasoning in the decision on whether to use it, and we would like to see any extra costs involved included in the bituminous alternate, for the Pavement Selection Determination.

We also support the idea of greater communication between the Concrete Office and the Districts. There hasn't been much concrete built in some of the districts over the last few years, and with the increased construction program, they will need to re-learn some of the design and construction techniques. Communication with the Concrete Office will play an important part in the learning process.

In closing, we would again like to thank the Task Force for the opportunity to comment on your findings, and look forward to working with all of you in the future.

Sincerely,

CPAM PAVEMENT DESIGN COMMITTEE

Ed Egan, Allstate Paving, Inc.
Joe Janowiak, Arcon Construction Co., Inc.
Jerry Ingman, Progressive Contracting, Inc.
Eno Olson, Shafer Contracting Co., Inc.
Arnie Laingen, CPAM
REFERENCES


2. Skok, E. L. Jr., Letters to L. P. Warren, Mn/DOT Concrete Engineer, dated October 12 and November 18, 1983, prepared by the Department of Civil and Mineral Engineering, University of Minnesota.

3. Allen, H. S., Memo to George Cochran, Mn/DOT Subgrade and Base Design Engineer, dated October 11, 1983, prepared by the Office of Research and Development, Mn/DOT.


