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Rapid Soil Identification and Classification for Highway Embankment Construction

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Rapid Soil Identification and Classification for Highway Embankment Construction

ABSTRACT

In response to some recent indications of poor embankment quality, the Iowa DOT has funded an embankment quality research program to identify design specifications and construction methods that may lead to low embankment quality. Some embankments have exhibited slope stability problems, and rough pavements had been observed shortly after being paved and opened to traffic. Based on extensive field testing and by taking into consideration the engineering properties of soils and simple property correlations, the *Iowa Soil Design and Construction (SDC)* chart was developed and designed to improve overall soil identification and to facilitate rapid field identification of fine and coarse-grained soils with plasticity. The identification method is based on liquid limit, plasticity index and fines content (percent passing No. 200 sieve). Relationships between shrink/swell potential (clay content) and frost susceptibility (silt content) were derived for Iowa soils. Furthermore, by considering changes in soil properties with changes in moisture content and determining desired soil properties and constructability, the *Iowa Moisture Content Construction (MCC)* chart was developed. Objectives of the *Iowa MCC* chart are to increase soil uniformity, stability, and overall embankment performance of soils through specifying soil specific minimum and maximum moisture contents. Moisture boundaries are established based on a three-point standard Proctor curve.

KEYWORDS: soil classification, moisture control, embankments, expansive soils, and frost action

INTORDUCTION

An evaluation of Iowa highway embankment quality was initiated in 1997 as a result of internal Iowa Department of Transportation (Iowa DOT) studies that raised concerns about the quality of recently constructed soil embankments. Some embankments reportedly exhibited slope stability problems, and rough pavements had been observed shortly after being opened to traffic (*1*). This raised the question as to whether the current Iowa DOT embankment design and construction methods were adequate. Research began with an investigation of several embankments under construction during summer 1997 through summer 1998. As part of the investigation, Iowa DOT resident construction engineers and field inspectors and earthwork contractors were interviewed to develop an understanding of current knowledge of specifications, engineering soil classification and properties, and desired embankment quality. From these interviews and discussions it became apparent that there were problems with some earthwork specifications and construction methods. Foremost it was observed that the current state-of-knowledge regarding soil identification and placement was lower than anticipated. Field personnel and contractors appeared to be generally conscientious and trying to do a good job, but they were misidentifying soils in the field during construction and lacked the equipment to perform field identifications. Soil identification problems arose when soils were mixed during borrow excavation and when boring data was insufficient. With further field testing, including density and moisture tests and in-situ drilling operations it was discovered that embankment fill materials on several projects, regardless of soil type, were being placed wet of "optimum" and saturated. Fill materials compacted with high moisture contents were frequently observed and caused instability under construction equipment.

For engineering purposes soil classification evolved from the need to group soils with similar properties together for highway and airport construction in the early 1900's. Implicit in a soil classification method is that soils with similar engineering properties can be grouped and that correlations exist between soil properties (*2*). Derived from many years of adjusting soil groups and engineering properties, the most widely used soil classification systems in use today by engineers are the American

Association of State Highway and Transportation Officials (AASHTO) system, the U.S. Department of Agriculture (USDA) textural system, and the Unified Soil Classification (USCS) system. For highway embankment construction the Iowa DOT currently uses the AASHTO system as the primary soil classification method.

Knowledge of shrink/swell behavior, performance under load, frost susceptibility, and permeability are some of the notable engineering properties that may be desired for embankment and subgrade design and construction. In comparison with dam and levee construction, multistory building sites and other sensitive projects, performing expensive and exhaustive laboratory tests on large quantities of soil used in highway embankment construction is currently not practical. With this in mind the following are the objectives of this paper: (1) Show an expedient and improved soil identification method for highway embankment construction termed the *Iowa Soil Design and Construction (SDC)* chart that can be performed by field personnel and can be made from unsophisticated tests in the field; and (2) Improve overall embankment quality (stability and density) by establishing simple soil moisture boundaries from the *Iowa Moisture Content Construction (MCC)* chart based on soil classification and optimum Proctor moisture content.

Over the years, many soil classification systems have been developed based on soil index properties and grain size. It should be noted that in 1927 Terzaghi concluded that the final soil classification for engineering purposes, related to highway construction, should not be based solely on arbitrary data such as liquid limit or plasticity index or even textural classification. Terzaghi believed that the final system of soil classification should be based on the complex behavior of the soil "under various conditions of stress and confinement"(*3*). Terzaghi proposed that the following information about a soil be collected:

1. The volume change produced by a change of the external pressure (compressibility and elasticity of the soil)
2. The speed with which the volume change follows a change of the pressure (coefficient of

consolidation).

3. The permeability of the soil (coefficient of permeability).
4. Volume change due to drying and wetting under standard conditions (shrink/swell potential).
5. Consistency of the soil in two extreme states (Atterberg limits). (*3*)

Through defining these five parameters, achieving final soil classification would require extensive laboratory analysis. However, the result would be a very well defined soil classification and prediction of soil performance. Again, because highway engineers need a simple, less laboratory intensive way to classify soils for highway construction, the *Iowa SDC* chart was developed, as a practical grouping of subgrade soils that would facilitate field identification. The *Iowa SDC* simply relies upon the determination of the liquid limit, plasticity index and fines content (percent passing the No. 200 sieve). Furthermore, because the primary Iowa DOT earthwork specification does not require moisture control as an acceptance criterion except in select treatment areas, the *Iowa MCC* chart was also designed and developed for use in the field. The proposed soil groupings and moisture control limits combine both personal experiences from interviewing several Iowa DOT field personnel and earthwork contractors and from road construction testing and observations and behavior of soils based on collected laboratory data.

CURRENT PRACTICE

Soil Identification

Soils for Iowa DOT highway embankment construction projects are identified during the exploration phase of the construction process. Borings are taken periodically along the proposed route and at potential borrow pits every 50-150 meters to depths of approximately 6 -12 meters depending on proposed embankment fill heights. Typically, fill heights on any one project across Iowa will vary from about zero to 10 meters. Iowa's land surface consists primarily of Pleistocene loess deposits (40%), glacial till (40%), alluvium (20%), and residual soils over bedrock (<1%) (*4,5*). Soils available for embankment construction in Iowa generally range from A-4 soils, which are very fine sands and silts that are subject to frost action, to A-6 and A-7 soils, which are predominate across the state. Some of the glacial derived A-6 and A-7 groups include relatively high shrink/swell clayey soils. In general these soils rate from poor to fair in suitability as subgrade soils; though, their suitability greatly depends on maintaining a uniform moisture content (*5*). Because of their abundance, economics dictate that these soils must be used on projects even though they exhibit marginal or undesirable properties. It is critical that the embankments built with these marginal soils be placed at the proper moisture content and that the unsuitable expansive and frost prone soils be identified and disposed of properly in the embankment.

Soil samples obtained from the referenced subsurface investigation are tested to determine engineering properties. Atterberg limits, grain-size distribution from hydrometer analysis, carbon content and color are determined and in-situ moisture and density are compared to standard Proctor values that are calculated by the one-point Proctor method (Iowa Test Method Number 103-C). From 1996 to 1999 the Iowa DOT has classified over 12,000 soil samples for an estimated 720 km of completed State, U.S., and Interstate highway embankments. The soil information is reported on soil design sheets for use during earthwork construction. However, even with this large number of soil samples it is impossible to completely characterize soil profiles because of variability between boring locations and more importantly, soil mixing during construction.

Currently, the Iowa DOT uses the soil classification specification shown in Table 1 for determining soil suitability during the exploration phase of the project. As indicated the classification procedure is extensive and requires determination of silt-size fraction from hydrometer analysis, Atterberg limits, maximum Proctor density, group index calculation, carbon content, and a visual classification for peat, muck, shale, or residual soils. Few state DOT's have similar specifications. Ohio bases suitable soil classification on a minimum Proctor density of 1600 kg/m^3 and unsuitable soils as having liquid limit greater than 65 or A-2-5, A-5 or A-7-5 soils. The Michigan DOT recognizes unsuitable frost susceptible materials as materials containing more than 50 percent silt (0.075 to 0.002 mm) combined with a plasticity index less than 10. Louisiana specifies select soils as having a maximum

PI of 15, maximum liquid limit of 35, maximum silt content of 60 percent and organic content less than 2 percent. Despite these findings for specific limits of soil index properties, it was found that the majority of State DOTs specify classification of soil during excavation based on the discretion of the field engineer. This process requires field identification throughout construction of the embankment, for which the proposed *Iowa SDC* chart could be useful.

Completed Embankment Soil Profiles

Based only on appearance and feel, predicting the physical performance and judging the suitability of soils for highway embankment construction is notoriously difficult. Regardless of engineering classification, field personnel often judge soils encountered during embankment construction for suitability based on color and ability to support heavy construction equipment. “Red-dog”, “hardpan”, “tiger”, “blue clay”, “gumbo”, “residual”, “sugar clay” and “muck” are common field terms used to describe soils and to predict engineering performance. The names and properties of these field-described soils vary between earthwork contractors, field inspectors and from region to region across Iowa. For a given field name, the soil may be one of a variety of materials such as a paleosol, glacial till, alluvial deposit, loess, or weathered shale. Design or construction of soils based on these field terms leads to poor embankment quality (*1*). As an example, on a recent embankment project an earthwork contractor used “blue clay” as a select treatment subgrade material because it reportedly became very hard under construction equipment on a previous project, and it made a durable equipment haul road. In fact, the overcompacted, unoxidized “blue clay” was a glacially derived A-7-6(38) (clay loam). Realistically this relatively expansive soil will not perform suitably as a select treatment material and will ultimately increase pavement degradation and maintenance costs.

Based on these observations from recent embankment projects, it was concluded that field personnel and earthwork contractors lack the needed engineering soil identification skills and the equipment to perform classification tests in the field when questions develop. Field personnel and earthwork contractors were relying heavily on the Iowa DOT soil design plan sheets for locating unsuitable, suitable, and select treatment soils. Although the soils design sheets appear accurate, the spacing between borings and soil mixing during construction operations, as illustrated in Figure 1, make it difficult to differentiate soils in the field (*1*). Also, field personnel knowledge of the soil classification systems used on the soils design data sheets was minimal. Consequently, differentiating the suitability, from an engineering standpoint, of an A-7-6 (clay) versus a frost susceptible A-4 (silt loam) on the soil design sheets was difficult, which lead to misplacement of soils and low embankment quality.

To cite an example of soil misplacement, the liquid limit and plasticity index of soils encountered in a boring through a completed embankment is shown in Figure 2. Throughout the embankment, the liquid limit varied from 33% to 58% with respective plasticity index values of 16% to 41%. Most of the embankment was constructed of soils classified as select glacial till. However, as shown the upper lift consists of highly plastic unsuitable soil. This unsuitable material was misidentified by the earthwork contractor and Iowa DOT field personnel and should have been placed at least 1.5 meters below grade according to the current Iowa DOT disposal practices.

Moisture Control

Except for subgrade treatment materials, the primary Iowa DOT specification does not set moisture control limits. Iowa DOT specification states: "It will be the contractor's prerogative to determine if moisture content of the material is excessive or suitable for satisfactory compaction." Furthermore, "satisfactory compaction" is based on a maximum 75-mm sheepsfoot roller stud penetration under a minimum 1400 kPa foot contact pressure. In order to establish acceptable compaction moisture conditions, the current field practice relies heavily upon good judgement and considerable experience on part of the earthwork contractors and Iowa DOT field inspectors (*1*). Interestingly, the above referenced moisture specification was found similar to most other state DOT's. Table 2 summarizes the primary moisture content requirements of all of the DOT's. The surveyed DOT specifications include a wide range of required moisture contents. Variations in moisture control result from different soil types and different philosophies as to what moisture content provides the best soil properties. Some states limit the moisture content to not exceed optimum, while other DOT's specify optimum moisture content as the minimum. Among practitioners in geotechnical engineering, there is constant debate about whether to compact soils wet or dry of optimum moisture content. The answer depends upon the material, engineering property requirements, and the practicality of obtaining those properties, which are often competing in highway embankment construction. High strength and density, low permeability, low shrink swell behavior, and low compressibility are all desired outcomes related to soil moisture content.

Moisture-Density Field Tests

Recent field tests performed on an embankment, constructed using the current Iowa DOT specifications, with A-7-6 soils of glacial origin are indicated in Figure 3. These moisture-density relationships indicate that soils excavated from borrows are typically very wet and when compacted are highly saturated, which causes low density and stability. High moisture contents can cause variable settlement and increased consolidation time for the embankment and add to the potential of slope failure. In addition, rutting during placement of wet soil builds in shear planes and sets the stage for potential slope failure. Essentially, the *Iowa MCC* chart objective is to better control moisture content and desirable soil properties.

IOWA SOIL DESISGN AND CONSTRUCTION (SDC) CHART

SDC Classification Procedure

By taking into consideration the engineering properties of fine and coarse-grained plastic soils and simple property correlations, the *Iowa SDC* chart was developed to improve overall soil design and to facilitate field identification during construction. The *Iowa SDC* chart, as shown in Figure 4, classifies soil based on three simple tests (1) liquid limit, (2) plasticity index, and (3) fines content (percent passing the No. 200 sieve). Recently, this classification method has been utilized on an Iowa DOT pilot project. Research has shown that the classification method is an effective tool to use when soils are being mixed in the borrow excavation or not identified on the soil design sheets. Once a soil sample is obtained from the borrow (or grade during construction), the liquid limit, plasticity index, and fines content are determined. In order to perform these tests in the a field, a lab trailer was equipped with a liquid limit device, small glass plate for plastic limit, microwave, scale, and No. 40 and 200 sieves. In addition a 5000-liter water tank was used for sieve washing. Total testing time to perform these tests averaged approximately one hour for an efficient technician.

Once these tests are performed the Iowa SDC chart is then used to designate the soil as either *select treatment material*, *suitable soil*, or *unsuitable soil* as described in the following:

1. Plot the liquid limit and plasticity index on the *Iowa SDC* chart shown in Figure 4.
2. Determine in which designated region the soil plots; for example LL =56 and PI = 37 plots in the *High plasticity inorganic clay* region.
3. Determine if the fines content (percent passing the No. 200 sieve) is less than or greater than the Fineness Designation Number as shown on the Iowa SDC chart.
4. Use guidelines shown in Table 3 to classify soil as *select treatment material*, *suitable soil*, or *unsuitable soil*.

In general terms, *select treatment materials* are those soils placed directly under the pavement structure (0 to 0.6 m) to provide adequate volumetric stability, low frost potential, and good bearing capacity. *Suitable materials* underlay the select treatment materials and are usually susceptible to seasonal moisture changes such as wetting and drying cycles. *Unsuitable materials* are commonly characterized as highly plastic clays or highly compressible frost prone silts and are buried (1.0 to 1.5 m) beneath the top of subgrade. Several examples and comparison classifications between the *Iowa SDC* chart and conventional soil index properties are shown in Table 4.

With current soil classification limitations in mind, shrink/swell potential and susceptibility to frost heave were critical engineering properties incorporated in the *Iowa SDC* chart. Further, to better define the relationship between clay and silt materials the Casagrande A-Line and U-line were utilized. Also, the new group index empirical formula (AASHTO 145-91) was used as a means of weighting the effects of plasticity index, liquid limits, and percent passing the No. 200 sieve. Brief descriptions of the critical engineering properties used to develop the *Iowa SDC* are discussed in the following sections.

Shrink/Swell Potential

One of the most significant engineering properties of a soil that results in pavement degradation is soil volume change. Although soil volume changes can be attributed to numerous factors, the most significant component is the soil's propensity to shrink and swell with changes in moisture content. According to Mitchell (*6*), swelling and shrinking of clayey soils is a direct result of wetting and drying, which are functions of clay particle mineralogy, particle arrangement, initial moisture content, and confining pressure. As a perspective, it has been reported that damage costs from shrinking and swelling soils on buildings and pavement structures are greater than that produced by any other natural hazard including floods, hurricanes, tornadoes, and earthquakes (*7*). Furthermore, it is estimated that damages from expansive soils in the United States reaches \$10 billion annually (*8*). To design against problems associated with expansive soils, soil properties and the conditions that contribute to moisture changes in the soils must be identified (*7*).

Several relationships have been developed to show that clay type and amount is directly related to swell potential. However, no single test, such as free swell or x-ray diffraction, is feasible or fully reliable for predicting actual field behavior of soil. Although swell potential relates to the performance of structures like building foundations, it is not normally carried out during embankment design.

Consequently, correlations between swell potential and factors that relate to clay mineral composition, such as plasticity index and liquid limit, were incorporated into the *Iowa SDC* chart. For Iowa soils a relationship exists between plasticity index and clay content (< 0.002 mm) as shown in Figure 5. For most Iowa soils, clay content can be estimated by the following equation:

$$\text{(\% Clay-size fraction, by weight)} = 0.95(PI) + 6.75 \quad [1]$$

Other comparisons of correlations by different authors (*9, 10, 11, 12, 13, 14, 15*) between swelling potential and both plasticity index and liquid limit are shown in Table 5.

Essentially, a plasticity index of ≥ 35 combined with a liquid limit of ≥ 50 indicates approximately 40 percent clay content and a high to very high swell potential. A plasticity index of 10 or less indicates a clay content of approximately 15 percent or less and appears to be a boundary value for low swell potential and provides an indication of susceptibility to frost action. As clay content decreases permeability increases allowing increased capillary water movement. Critical liquid limit values of 50 and plasticity index values of 10 and 35 were used to develop the select, suitable, and unsuitable soil boundaries in the *Iowa SDC* chart for low to medium to high plasticity clays.

Frost Susceptibility

Preventing frost susceptible soils from being incorporated into the upper portion of the pavement subgrade was also a major consideration while developing the *Iowa SDC* chart. Field experience with concrete and asphalt has shown that frost damage occurs if all of the following conditions are present: (1) a supply of water, (2) freezing temperatures penetrating the ground, and (3) frost-susceptible soils. Eliminating just one of the conditions will prevent frost damage. The most practical way to prevent frost damage is to identify and eliminate frost susceptible soils, because precipitation and fluctuating water tables supply water and frost penetration in Iowa ranges from 3 feet in the south to 5 feet in the north (*16*). Based on hydraulic principles such as soil permeability and capillary action, low plasticity silts and fine sands are the most frost susceptible soils; whereas, gravels and heavy clays are the least susceptible.

To better define frost susceptible soils the Department of the Army (*17*) developed a guideline for classification of frost susceptible soils and conducted laboratory heave measurements on several remolded samples. From this investigation ML, ML-OL, CL, CL-ML, and SM-SC were found to have high to very high frost susceptibility. Identifying these inorganic silts and low to medium plasticity clays can eliminate the majority of frost susceptible soils from the subgrade. Based on the referenced plasticity index versus clay content relationship for Iowa soils and a determination of the fines content (F_{200} - percent passing the No. 200 sieve), the silt-size fraction can be estimated by the following equation:

$$\text{(\% Silt-size fraction, by weight)} = F_{200} - [0.95(PI) + 6.75] \qquad [2]$$

Well-graded silty or fine sandy Iowa soils with plasticity index ≤ 10 and fines content of 70% or greater indicates the potential for frost action and should not be placed directly under the pavement structure.

Group Index Weighting

Because not all soils with high plasticity index and liquid limits have high swell potential (some glacial tills of Iowa) the group index was used to weight the liquid limit, plasticity index and the percentage of materials passing the No. 200 sieve. The original empirical formula was designed to weight the maximum influence of each of the three variables in the ratio of 8 for percent passing the No. 200 sieve, 4 for liquid limit, and 8 for plasticity index (*18*). Furthermore, the quality of a subgrade material was assumed an inverse to its group index. Based on the new AASHTO M-145-91 group index formula, critical values of 0-15 for select soils, 15-30 for suitable soils and greater than 30 for unsuitable soils were determined based on typical Iowa soils and taking into consideration the previously discussed critical values of plasticity index and liquid limit. Although the group index provides an indication of clay mineral activity and volume of fines fraction, it was not used in the *Iowa SDC* chart to evaluate frost susceptibility because frost susceptible soils are typically low plasticity materials.

IOWA MOISTURE CONTENT CONSTRUCTION (MCC) CHART

MCC System Procedure

The objectives of the *Iowa MCC* chart are to ensure stability and increase soil uniformity and overall embankment performance for soils through specifying soil specific minimum and maximum moisture contents. Acceptable moisture content ranges are based on soil classification according to the *Iowa SDC* classification procedure and standard Proctor "optimum" moisture content and "maximum" dry density. The *Iowa MCC* chart, as shown in Figure 6, can be utilized to establish the acceptable soil moisture content range. The chart is divided into two sections, one for select and suitable soils and one for unsuitable soils. In order to establish the proper moisture boundaries for a soil the following procedures are followed:

1. Obtain a soil sample from the borrow or grade during construction and wet-grind the material through a No. 4 sieve.
2. Split the sample into three portions of about 2500g each.
3. Immediately perform a standard Proctor test on the wet material while allowing the other two samples to aerate and begin drying.
4. Next, use the two remaining samples at consecutively dryer moisture contents to establish a three-point Proctor curve.
5. Plot the results and calculate the "optimum" Proctor moisture content

6. Lastly, based on the *Iowa SDC* soil classification and standard Proctor "optimum" moisture content, determine the appropriate moisture boundaries from the *Iowa MCC* chart shown in Figure 6.

Equipment required for determination of the moisture boundaries includes a Proctor mold and standard hammer, No. 4 sieve, microwave, and scale. The estimated time, as performed on recent Iowa DOT pilot projects, for this procedure is approximately 1.5 hours, but testing time is greatly dependent upon having the proper laboratory equipment.

Moisture Boundaries for Select and Suitable Soils

The acceptable moisture content range for a select or suitable soil is fixed at –1% to +3% of "optimum" Proctor moisture content. The objective for select and suitable soils, which are placed in the upper portion of the embankment, is to provide the appropriate moisture content that minimizes swell potential, produces uniform density, and provides adequate stability for equipment and paving operations.

Moisture Boundaries for Unsuitable Soils

A second moisture content boundary has been established for unsuitable soils. The amount of water to be used in compacting unsuitable high plasticity clay soils shall not deviate from optimum on the dry side by more than 90% and not more than 120% on the wet side. However, as shown in Figure 6, if the optimum Proctor moisture content of the unsuitable material is over 20% (based on dry weight) then the minimum allowable moisture content is –2% and the maximum is +4%. Furthermore, at low optimum moisture contents the minimum moisture range is –1% to +3%. These moisture boundaries are set to better represent specific soil properties. For example, a high optimum moisture content typically exhibits a flatter Proctor curve while material with a low optimum moisture content has a sharper curve, which indicates that changes in moisture more greatly affects dry density. When used during construction the *Iowa MCC* chart can greatly increase the uniformity of density and stability. Also, the *Iowa MCC* chart will minimize low embankment shear strength zones caused by very wet and saturated materials observed on recent projects. Table 4 indicates moisture boundary results for various types of *select treatment materials*, *suitable soils*, and *unsuitable soils*.

CONCLUSIONS

- Design or construction of soils based on geologic origin, field terms and misidentification of soil as a result of soil mixing, or lack of complete subsurface information on the design sheets, leads to poor embankment quality.
- The *Iowa SDC* chart takes into account engineering properties such as swell potential, frost susceptibility, and group index weighting and is based on simple field tests to facilitate rapid field identification.
- By considering changes in soil properties from moisture content and determining desired soil properties and constructability, the proposed Iowa Moisture Construction Chart (MCC) was developed. Objectives of the MCC chart are to increase soil stability, uniformity and overall embankment performance for soils through specifying soil specific minimum and maximum moisture contents.
- Acceptable moisture content ranges are based on soil classification per the *Iowa SDC* chart and standard Proctor "optimum" moisture content and "maximum" dry density.

ACKNOWLEDGMENTS

The Project Development Division of the Iowa Department of Transportation and the Iowa Highway Research Board sponsored this study. The support of these agencies is greatly acknowledged. The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the Iowa Department of Transportation. The Iowa SDC and MCC charts are being reviewed by the Iowa Department of Transportation for implementation.

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TABLE 1 Iowa DOT soil classification specification

Select (cohesive) Treatment Materials[a]	Suitable Soils[b]	Unsuitable Soils[c]
• 45 percent or less silt size fraction (0.075 – 0.002 mm)	• 1500 kg/m^3 or greater density (AASHTO T 99 Proctor Density)	• Slope dressing only - Peat or muck - Soil with plastic limit ≥ 35 - A-7-5 or A-5 having density < 1350 kg/m^3
• 1750 kg/m^3 or greater density (AASHTO T 99 Proctor Density)	• Group index < 30 (AASHTO M 145-91)	• Disposal 1 m below subgrade - All soils other than A-7-5 or A-5 having density <1500 kg/m^3 - All soils other than A-7-5 or A-5 containing ≥ 3.0% carbon
• Plasticity index > 10		• Disposal 1.5 m below subgrade - A-7-6 (30 or greater) - Residual clays overlying bedrock regardless of classification
• A-6 or A-7-6 soils of glacial origin		• Disposal 1.5 m below subgrade with alternate layers of suitable soil - Shale - A-7-5 or A-5 soils having density from 1350 kg/m^3 to 1500 kg/m^3

[a]Must satisfy all conditions and typically used in top 0.6 m of subgrade.
[B]Must satisfy all conditions and used throughout fill except for top 0.6 m of subgrade.
[c]Restricted use as indicated.

TABLE 2 State DOT primary embankment moisture control specifications

Moisture Control Specification	Number of State DOTs
Adequate moisture to achieve specified compaction	31
±5	1
-4 to 0	1
-4 to +2	3
-4 to +5	1
±3	1
-2 to 0	1
-2 to +1	1
±2	5
0 to +3	1
0 to +5	1
≤ +2	1
≤ +3	1
≤ 115% of Optimum	1

TABLE 3 Iowa Soil Design and Construction (SDC) chart guidelines

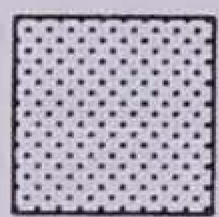
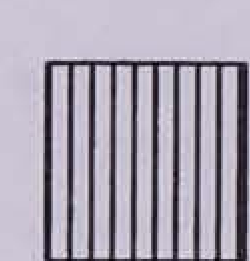
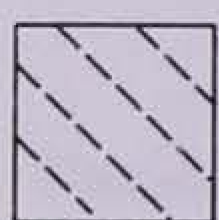
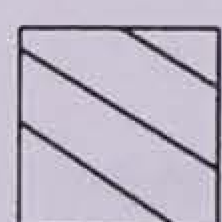
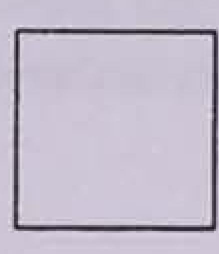
Legend	Designated Soil Regions
	<u>Low plasticity clays</u> • Select ≤ 45% passing the No. 200 sieve, F_{200}, and ≤ 70% F_{40} • Suitable 46% - 70% F_{200} • Unsuitable > 70% F_{200}
	<u>Low/Medium plasticity inorganic clays</u> • Select ≤ 60% F_{200} • Suitable 61 - 70% F_{200} • Unsuitable ≥ 70% F_{200}
	<u>Medium plasticity inorganic clays</u> • Select – Plots above A-Line (PI=0.73(LL-20), and F_{200} ≤ fineness designation • Suitable - F_{200} > fineness designation
	<u>High plasticity inorganic clays</u> • Suitable - Plots above A-Line (PI=0.73(LL-20), and F_{200} ≤ fineness designation • Unsuitable - F_{200} > fineness designation
	<u>Inorganic silts of medium compressibility</u> • Unsuitable - Plots below A-Line (PI=0.73(LL-20), • Dispose of below frost line
	<u>Highly compressible inorganic silts and high plasticity organic clays</u> • Unsuitable - (Slope dressing only)

TABLE 4 Comparison of Iowa SDC, Iowa MCC, and soil index properties

Liquid Limit (%)	Plasticity Index (%)	Fines Content (< 0.075 mm) (%)	Silt Fraction (%)	Clay Fraction (<0.002mm) (%)	Group Index	Standard Proctor		Iowa SDC	Iowa MCC Limits (%)
						Dry Density (kg/m^3)	Moisture Content (%)		
35	21	61	35	26	10	1890	13	Select	12-16
36	23	71	43	28	14	1787	16	Select	15-19
28	4	44	28	16	2	1918	12	Select	11-15
40	19	99	71	28	20	1868	13	Suitable	12-16
33	11	62	47	15	5	1758	16	Suitable	15-19
33	13	62	47	15	6	1866	13	Select	12-16
33	18	65	38	27	9	1820	15	Select	14-18
45	21	99	71	28	24	1696	18	Suitable	17-21
41	24	100	74	26	25	1705	18	Suitable	17-21
49	25	95	60	35	27	1670	19	Suitable	18-22
48	27	99	64	35	30	1651	20	Suitable	19-23
48	33	98	69	29	34	1523	22	Suitable	21-25
80	37	92	50	42	35	1536	23	Unsuitable	21-27
76	58	91	36	55	58	1515	26	Unsuitable	24-30
97	71	95	12	83	78	1430	27	Unsuitable	25-31

TABLE 5 Several relationships between swell potential and plasticity index and liquid limit (*9, 10, 11, 12, 13, 14, 15*)

Swell Potential	LL	PI	LL	PI	PI	PI	PI	PI
Low	20-35	< 15	< 50	< 25	< 12	< 20	< 15	< 15
Medium	35-50	10-35	50-60	25-35	12-23	12-34	10-30	15-24
High	50-70	20-55	> 60	> 35	23-32	23-45	20-55	25-46
Very High	> 70	> 35	(a)	(a)	> 32	> 32	> 40	> 46

[a] Classification not defined

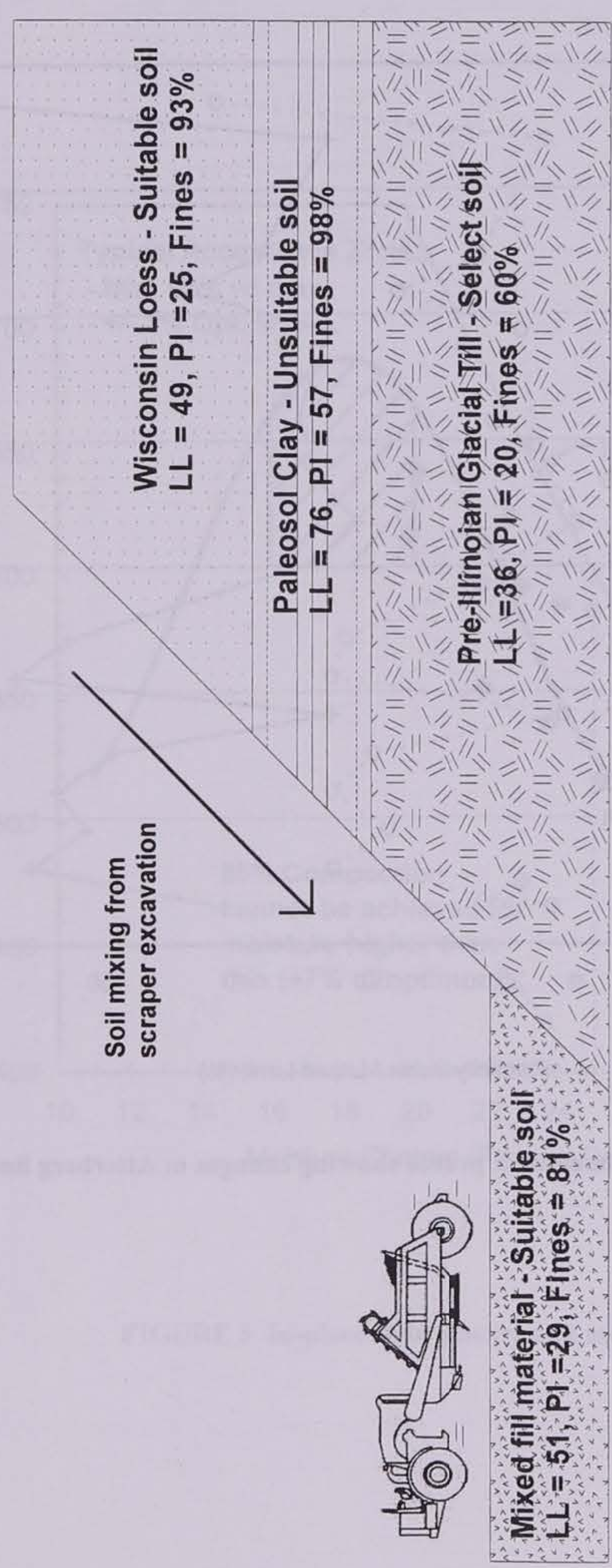


FIGURE 1 Soil mixing during excavation

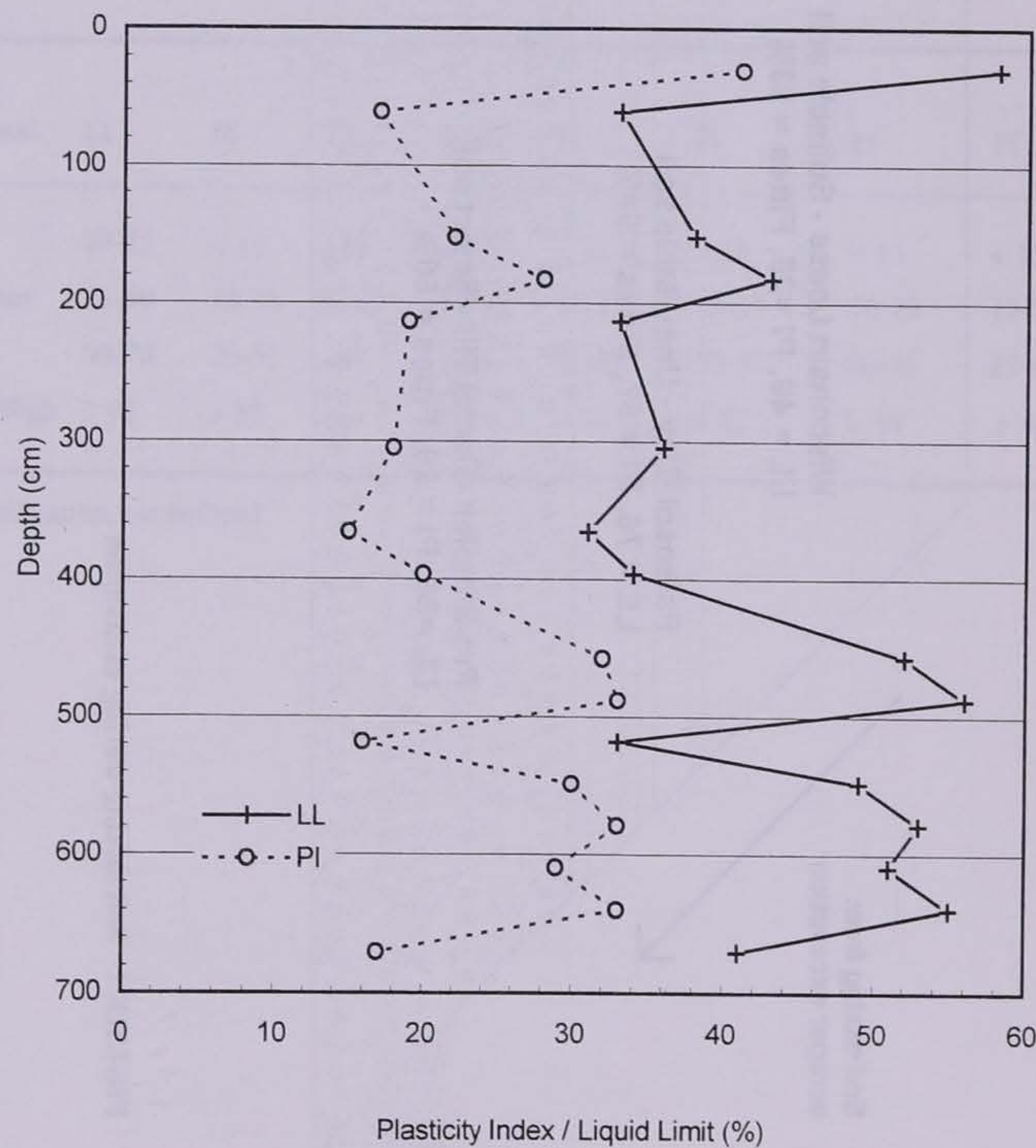


FIGURE 2 Embankment profile showing changes in Atterberg limits

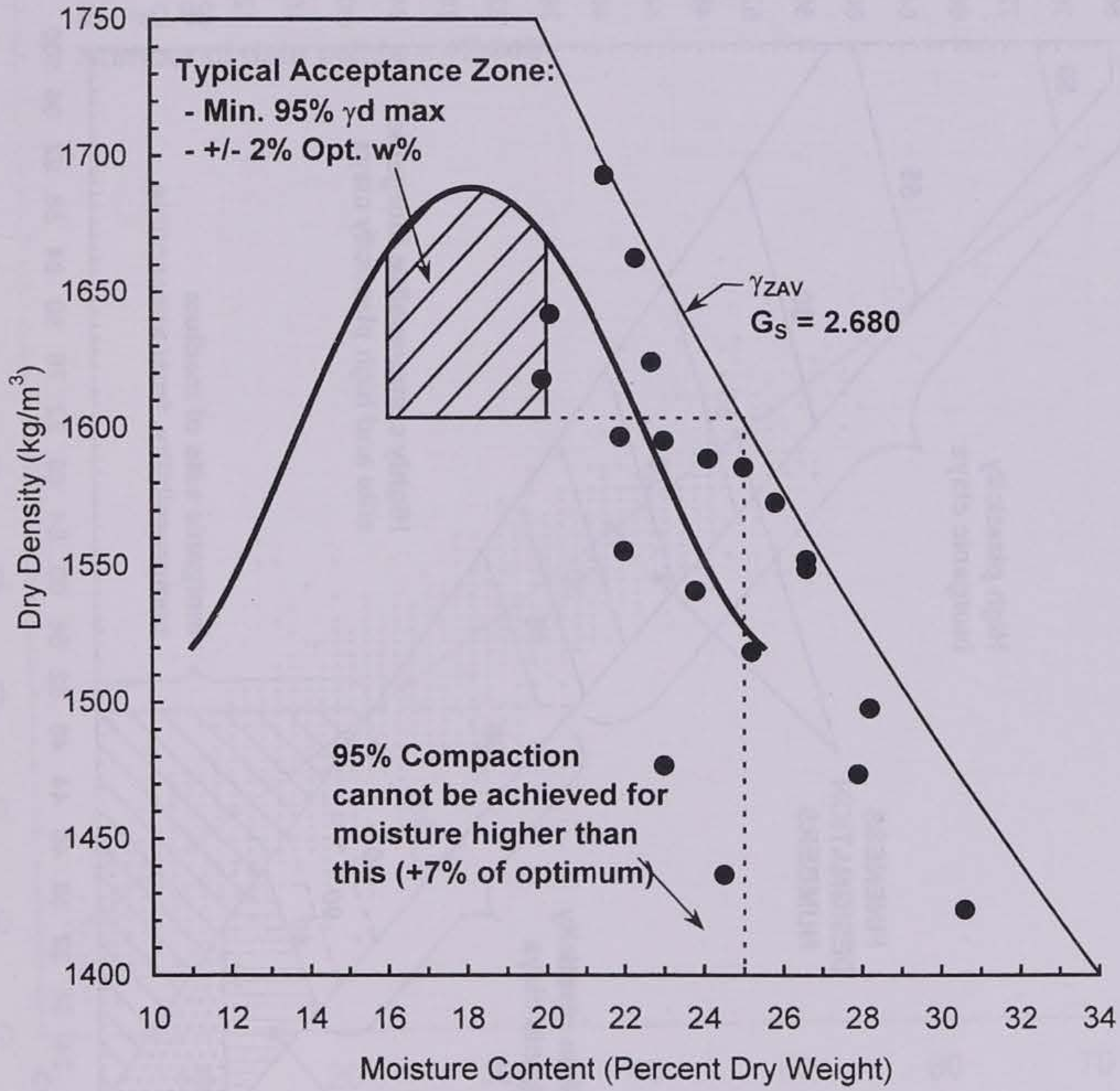


FIGURE 3 In-place field density and moisture results

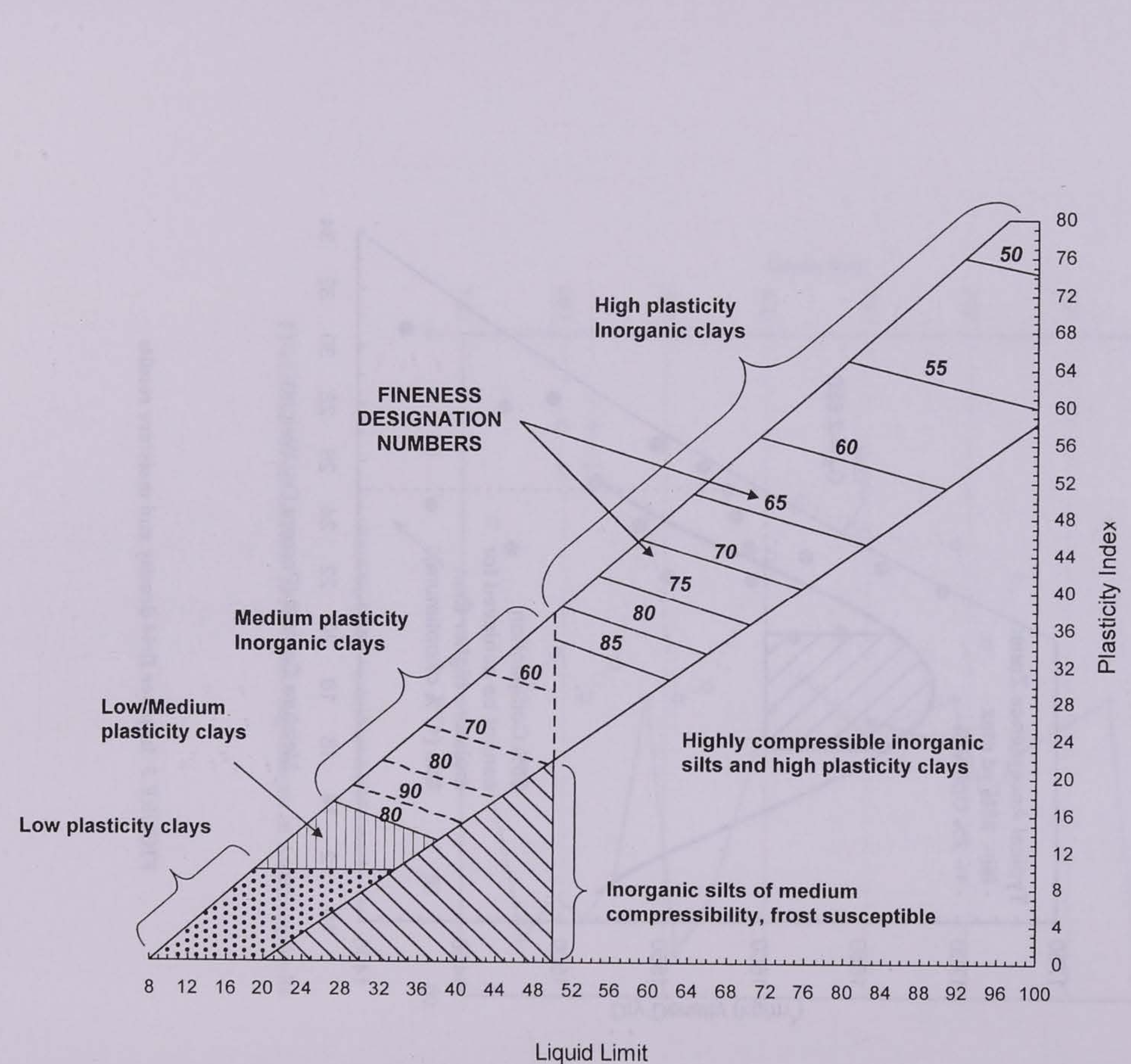


FIGURE 4 Iowa Soil Design and Construction (SDC) chart

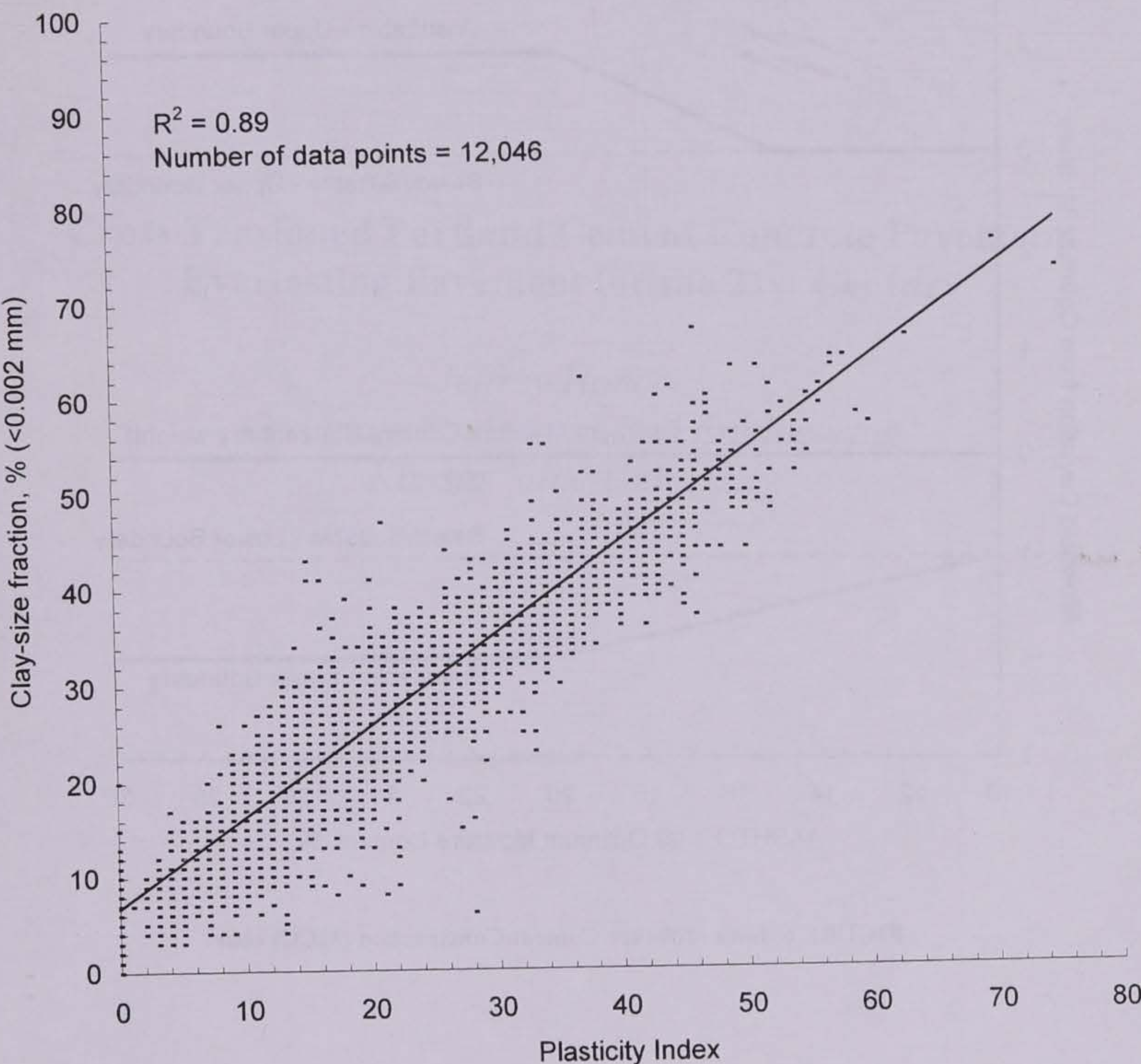


FIGURE 5 Relationship between clay content and plasticity index for Iowa soils

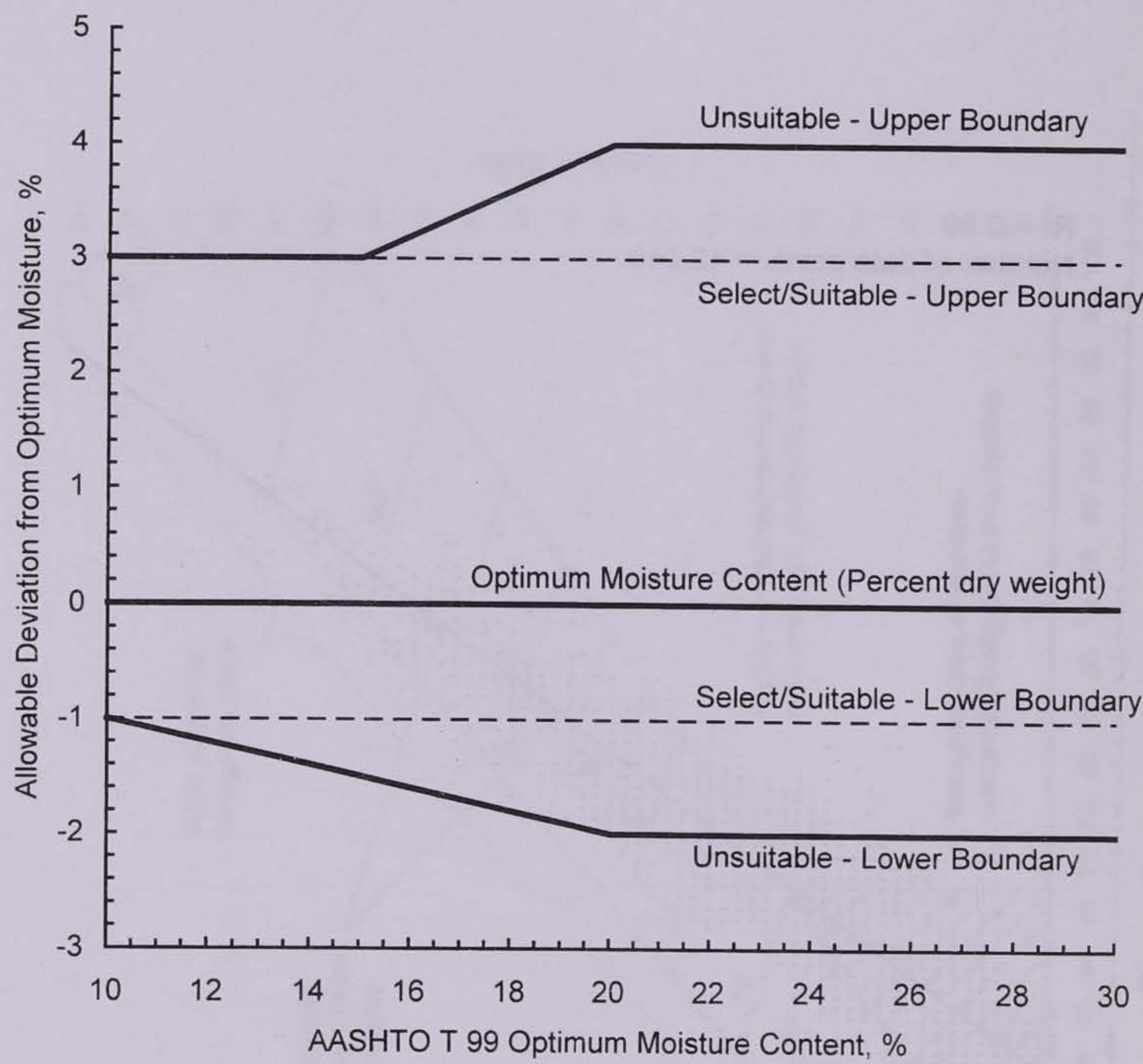


FIGURE 6 Iowa Moisture Content Construction (MCC) chart

Cross Tensioned Portland Cement Concrete Pavement, Everlasting Pavement for the 21st Century

Jeffrey Hancock
Graduate Student in Civil Engineering
Kansas State University

accomplished by **cross-tensioning** PCCP. This procedure may eliminate the need for any transverse joints and will resist the pavement's tendency to crack in any direction.

INTRODUCTION

The deterioration of Portland Cement Concrete Pavement (PCCP) is most often caused by the intrusion of water into the pavement. The water usually penetrates through transverse joints and uncontrollable cracks in concrete pavement. During the construction process and routine maintenance, the transverse joints and cracks are typically sealed with a flexible joint sealant. The sealant keeps the majority of water out, however, some water penetrate through the sealant and into the pavement. Once in the pavement, deterioration begins when the temperature drops below freezing and the water expands. Expanding water causes joints to spall and sometimes fault, cracks to expand, and an exponential increase the deterioration rate of the pavement, figure 1.

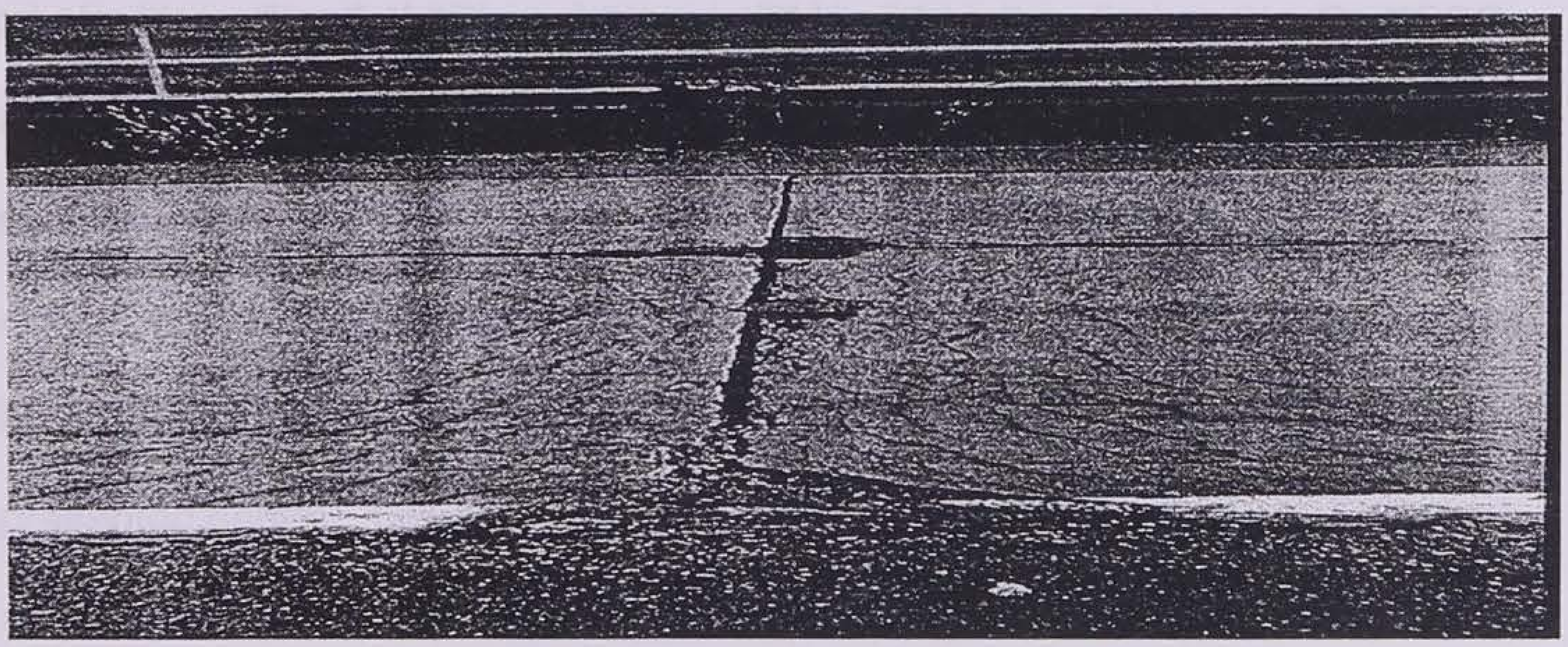


Figure 1 Transverse Joint Failure in Concrete Pavement

Eliminating transverse joints and cracks is one solution to this common problem of PCCP. Pre-stressing PCCP could eliminate joints and cracks. By applying an external force in the form of post-tensioning, theoretically all cracks and joints in the pavement can be eliminated. Therefore, at no location in the pavement can water enter and begin to deteriorate the pavement. Extended pavement life should also be an expected result of pre-stressing PCCP. Pre-stressed PCCP's are also of greater quality. The elimination of potentially all transverse joints will provide a smooth and comfortable travelling surface; lower maintenance cost, and increases the load-capacity of the pavement.

Pre-stressing is a broad term that covers many different types of applied external forces to PCCP. These types include post-stressed, pre-tensioned, and post-tensioned. Post-stressing is accomplished by applying an external force to the ends of slabs and is done without wires, strands, or cables. Pre-tensioning is done before the concrete is ever poured. Once poured strands, cables, or wires are able to apply an internal compressive force through surface friction to the PCCP. Post-tensioning, the topic of this report, is applied after the concrete has cured. The post-tensioning force is applied through un-bonded strands to the ends of the pavement. All of the for-mentioned procedures are generally referred to generically as "pre-stressing." This report discusses post-tensioning procedures.

BACKGROUND

Post-tensioning of PCCP is not a new concept. It has been done all over the world. The first construction of a post-tensioned pavement was at the Orly Airport, Paris, in 1946. The Europeans have long since advocated the use of post-tensioning in airport pavements. Europe has also taken the lead in the application of post-tensioning to highway pavements. During the 40's, 50's, and 60's the Europeans constructed over thirty post-tensioned pavements. During the same time the United States constructed only six airport pavements and zero highway pavements that were post-tensioned. Finally, in the early 70's three highway pavements and an access road at Dulles International Airport were constructed, table 1.

Since the early 70's, post-tensioning has become almost obsolete in highway design in the United States. However, as part of a number of experimental projects in 1977 the Arizona Department of Transportation (ADOT) constructed its first post-tensioned PCCP. The ADOT post-tensioned PCCP will be highlighted in this report for comparison to the method of post-tensioning presented herein. The ADOT PCCP has its good and bad qualities, all of which will be discussed in this report.

The design of post-tensioned PCCP is quite different from conventional design of PCCP. Most applications of post-tensioning occur in structures such as parking garages, buildings, and bridges. Furthermore, the majority of designers that use post-tensioning are of the structural breed not the pavement breed. Post-tensioned design involves some knowledge of the origins of post-

Date	Location	Slab Length (ft)	Thickness (in.)	Method
1953	Patuxent Naval Air Station, MD	500	7	Longitudinally with Cables
1955	San Antonio, TX	80	4	Both Directions with Cables
1955	Sharonville, OH	80	4	Both Directions with Wires
1957	Sharonville, OH	500	9	Both Directions with Steel Bars
1959	Biggs Air Force Base, TX	500	9	Both Directions with Cables
1959	Le moore Naval Air Station, CA	568	6	Both Directions with Cables
1971	Milford, DE	300	6	Longitudinally with Teflon Coated Strands
1971	Dulles International Airport, VA	400-760	6	Longitudinally with 1/2 in. dia., 7-wire strands
1973	Harrisburg, PA	600	6	Longitudinally with 1/2 in. dia., 7-wire strands
1973	Kutztown, PA	500	6	Longitudinally with 1/2 in. dia., 7-wire strands in polypropylene conduit

Table 1 Historical Post-Tensioned PCCP in the United States (Hanna, Nussbaum, Arriyavat, Tseng, Friberg, 1976)

tensioning, structures. For this reason, most pavement designers do not look at post-tensioning as an option in pavement design.

THEORY

Post-tensioned pavement design is unlike that of any other PCCP. Conventional PCCP design is based on the low modulus of rupture of concrete and does not take advantage of the high compressive strength of concrete. Post-tensioned concrete pavement increases the stress range in the flexural range of the concrete. The result has several advantages: (1) absence of cracks from the road surface, (2) reduced cost through a reduction in slab thickness, and (3) an extensive increase in load carrying capacity.

The fundamental formula for design of pre-stressed pavements is:

$$f_t + f_p \geq f_{\Delta t} + f_F + f_L$$

where,

f_t	=	allowable concrete flexural stress
	=	modulus of rupture/factor of safety
f_p	=	compressive stress in concrete due to post-tensioning
$f_{\Delta t}$	=	curling stress due to difference in temperature between top and bottom surfaces of the concrete slab
f_F	=	stresses due to subgrade friction
f_L	=	stresses due to traffic load

The allowable concrete flexural stress may be taken as high as 80 to 100 percent of the modulus of rupture of concrete. Introducing a comressive stress in the concrete pavment changes the criteron for failure from a bottom tesion crack to a top circular crack. The failure load is at least double the load that produces the first bottom crack . This allows the designer to choose a safety of factor betweeen 1 and 1.25 for f_t.

Allowable stresses in the post-tensioning strands, $f_{se,}$ after all losses,, should not exceed 80 percent of the ultimate strength, f_s' of the post tensioning steel. A typical monostrand post-tensioning strand will be a one-half inch diameter strand, made with six wires twisted around one wire. The ultimate strenght of post-tensioning strands is 270 ksi.

The curling stresses in concrete can be calculated from:

$$f_{\Delta t} = \pm \frac{\alpha E_c \Delta t}{2(1-\nu)}$$

where,

α = coefficient of thermal expansion (6X10^{-6} in/in/°F)

$\frac{\Delta t}{h}$ = temperature gradient 3°F/in.

h = slab thickness in inches

E_c = static modulus of elasticity of conrete in psi

ν = Poissons ratio for concrete = 0.15

Curling stresses in post-tensioned PCCP are caused by the warping of a slab due to creep or to differences in the temperature of moisture content in the zones adjacent to its opposite faces. Usually concrete curls when one face is warmer or cooler than the other face. The warmer face of the concrete wants to expand while the cooler face will not. This causes the slab to curl either up or down depending on which is face is warmer. If the warmer face of the concrete is on top then the slab will want to curl down on the ends, however, if the opposite is true the slab will want to curl up on the ends. These changes in stress in the concrete can have adverse effects on the stresses within the strands. For example suppose we have a slab where the stressed post-tensioning strands are in the lower half of the slab. When the lower face of the slab is cooler than the top face the slab will want to shorten on the bottom and expand at the top. The shortening at the bottom will cause some relaxation in the strands, hence the strands will loose some of their stress. Cracks may develop as a result.

Figure 2 shows the fluctuations in stress due to differing seasons on a PCCP in Liege, Belgium. The 11,000 ft. pavement was post-stressed with screw jacks. Engineers tried for four years to maintain the stress in the concrete by frequently adjusting the screw jacks. The results of their attempt shows fluctuations in prestress that range from nearly 0 to 1900 psi. By shortening the total slab length the adverse stresses caused by temperature differences would not have fluctuated so drastically.

Creep also has adverse effects on stresses. Creep is the change in length of pavement due to a constant pressure, figure 3. Inducing a post-

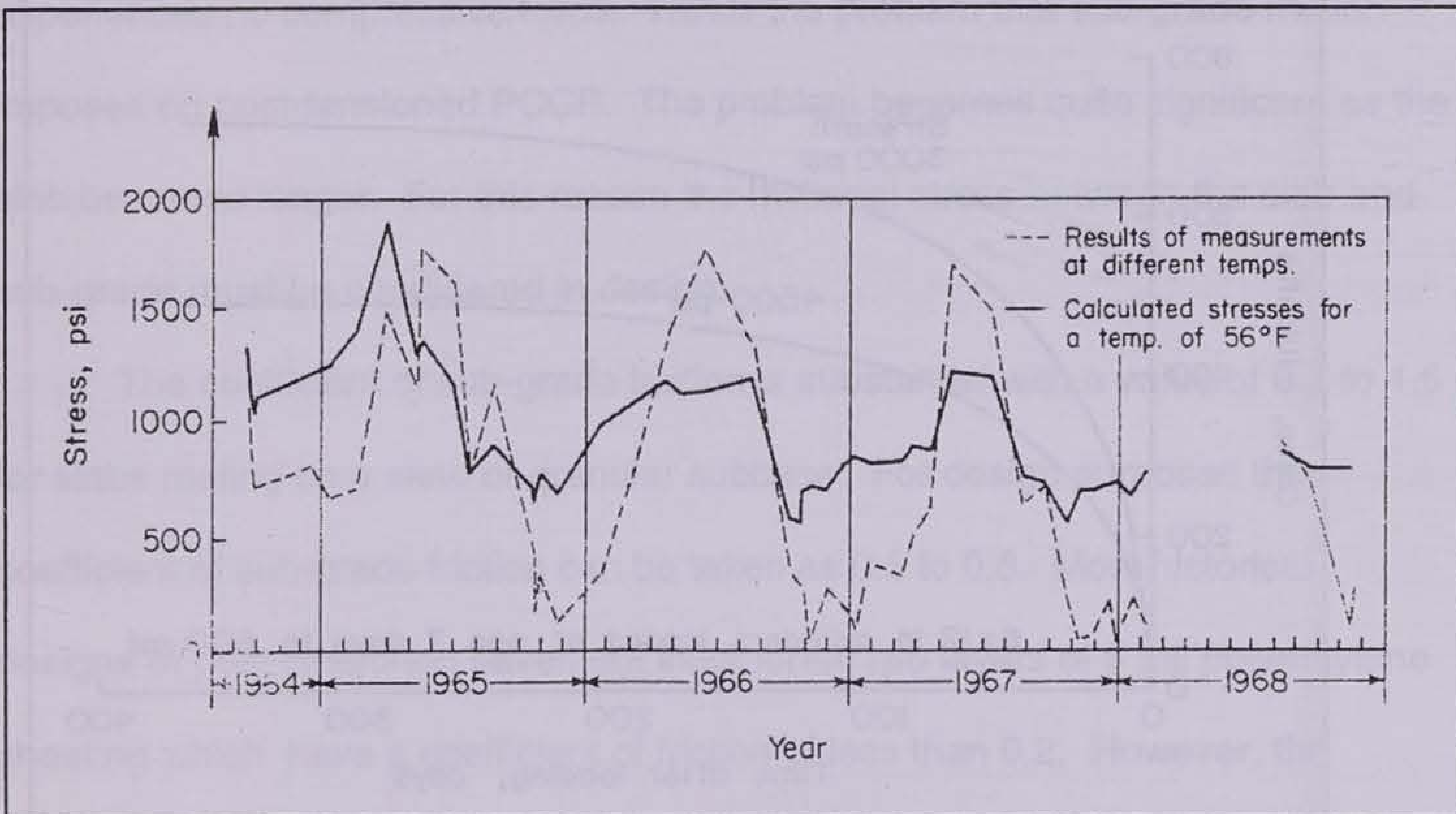


Figure 2 Fluctuations in Concrete Stress Due to Curling (Hanna, Nussbaum, Arriyavat, Tseng, Friberg, 1976)

tensioning force to the pavement subjects the concrete to a constant creep. Creep has the same effect on the stress force as curling, however, creep does not fluctuate unlike curling stress. Figure 3 shows that higher compressive strenghts in concrete produce less creep. Furthermore, for both of the cases shown the creep does level off after 300 to 400 days of load application. This allows the designer to design around the creep effects of the concrete. Most importantly the designer is aware that creep does exist and after a long period time is minute, under a constant force.

The friction between the sub-grade and PCCP also produces a stress that is most significant in the middle portion of the slab. For a slab that is less than 700 ft in length the frictional stress between the sub-grade and PCCP can be estimated as:

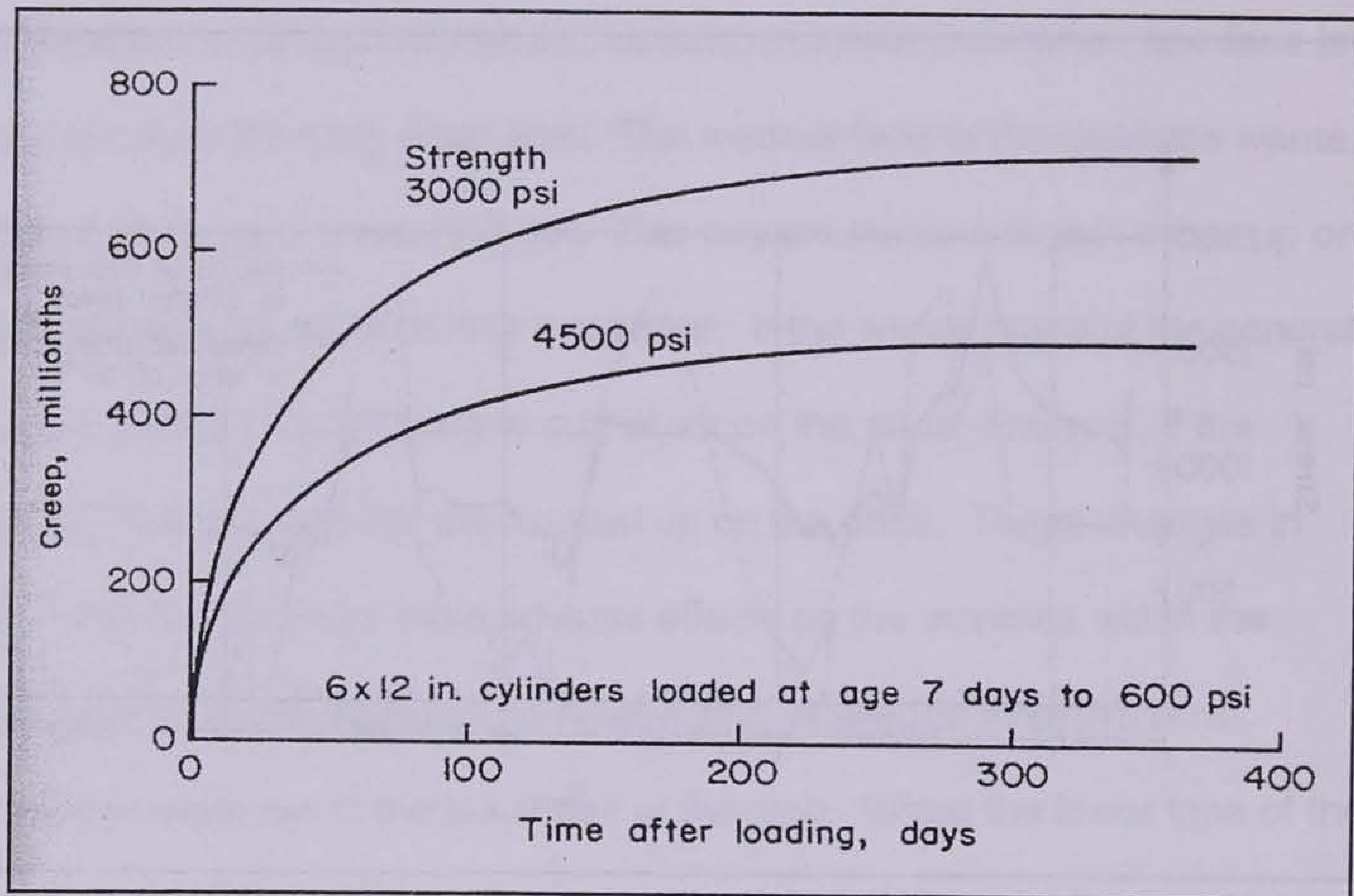


Figure 3 Creep in Concrete Due to a Constant Force (Hanna, Nussbaum, Arriyavat, Tseng, Friberg, 1976)

$$\max \quad f_F = \frac{c\gamma L}{2 \times 144}$$

where,

c	=	coefficient of sub-grade friction
L	=	total longitudinal length between ends of stressing strands
γ	=	unit weight of concrete = 145 lbs./cubic ft.

If the reaction force applied by the sub-grade friction is too much it could effect the results of the stress applied by the strands. Consider for example that a slab is pinned to the sub-grade at two different points between the ends of a post-tensioning strand. When the strand is pulled to tension a compressive force is exerted on the concrete slab. However, the region between the two pins experiences no compressive force. This is the problem that sub-grade friction imposes on post-tensioned PCCP. The problem becomes quite significant as the slab becomes longer. For this reason the frictional stress between the slab and sub-grade must be condidered in design.

The coefficient of sub-grade friction is substantial with a value of 0.2 to 1.5 for slabs resting on a sand or granular subbase. For design purposes the coefficient of sub-grade friction can be taken as 0.5 to 0.8. Most historical designs of post-tensioned pavement incorporate two layers of 6 mil polyethylene sheeting which have a coefficient of friction of less than 0.2. However, the ADOT post-tensioned PCCP project discovered that almost no difference exist in friction between the slab and sub-grade on either 1 or 2 layers of polyethylene sheeting. In either case it is recommended to still use the conservative value of 0.5 for a coefficient of friction in design. This gives the designer reasonable assurance that the value can be obtained in the field.

All designs of PCCP consider traffic load as the most detrimental to the design of the pavement. For post-tensioned PCCP traffic load consideration is no different. Traffic loads induce a tensile stress in the bottom of the pavement that is calculated by Westergaard's formula. The edge loading of PCCP is always the controlling criteria. The edge is where the stress in the slab will always be the greatest under a heavy tire load. The stresses in the pavement induced by successive traffic loads can lead to elastic deformation of the slab to the point where the maximum moment beneath the loaded area exceeds the sum of the flexural strength of the concrete and the induced stress force. At this point a crack forms a hinge under the load, repeated load applications cause a moment in the slab some distance away from the loaded area. If traffic repetitions continue tensile cracks can form in the top of the pavement. When loading is increased beyond this point the load will eventually punch through the slab. For this reason it is extremely important to consider traffic load in post-tensioned PCCP.

Westergaard's formula for edge loading of PCCP is given as:

$$f_L = \frac{0.803P}{h^2}\left[4\log\left(\frac{l}{a}\right) + 0.666\left(\frac{a}{l}\right) - 0.034\right]$$

where,

f_L	=	maximum edge stress under load
l	=	radius of relative stiffness
	=	$\left[\frac{Eh^3}{12(1-\nu^2)k}\right]^{0.25}$
E	=	modulus of elasticity of concrete, assume 4,000,000 psi
k	=	modulus of sub-grade reaction, if not known assume 100 pci
P	=	concentrated load, lbs.
a	=	contact radius
	=	$\sqrt{\frac{P}{Tire\ Pressure \times \Pi}}$

For a typical maximum load of 22, 000 lbs per axle on duals P can be estimated as 5,500 lbs and the tire pressure can be estimated as 100 psi.

Deflections also occur in the slab. Deflections in PCCP are most often caused by failure of the sub-grade. Post-tensioned pavement, however, can span the unfavorable portions the sub-grade. This is done by an increase in flexural strength of the concrete due to the post-tensioning force. Spanning the undesirable sub-grade material limits post-tensioned PCCP's ability to deflect under repeated loads. Figures 4 and 5 illustrate the deflective resistance of post-tensioned PCCP.

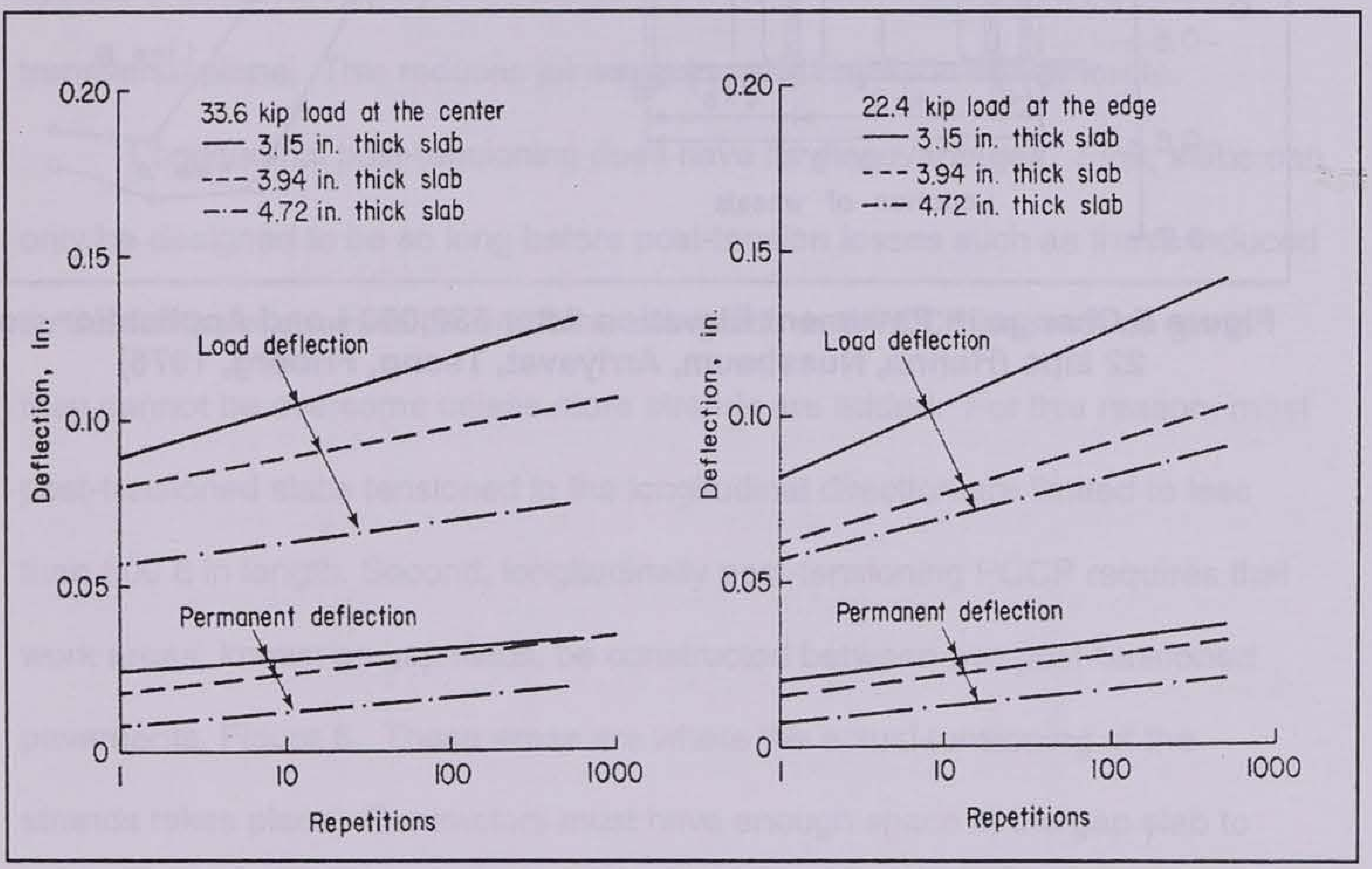


Figure 4 Center and Edge Deflections of Post-Tensioned PCCP Under Repeated Loads (Hanna, Nussbaum, Arriyavat, Tseng, Friberg, 1976)

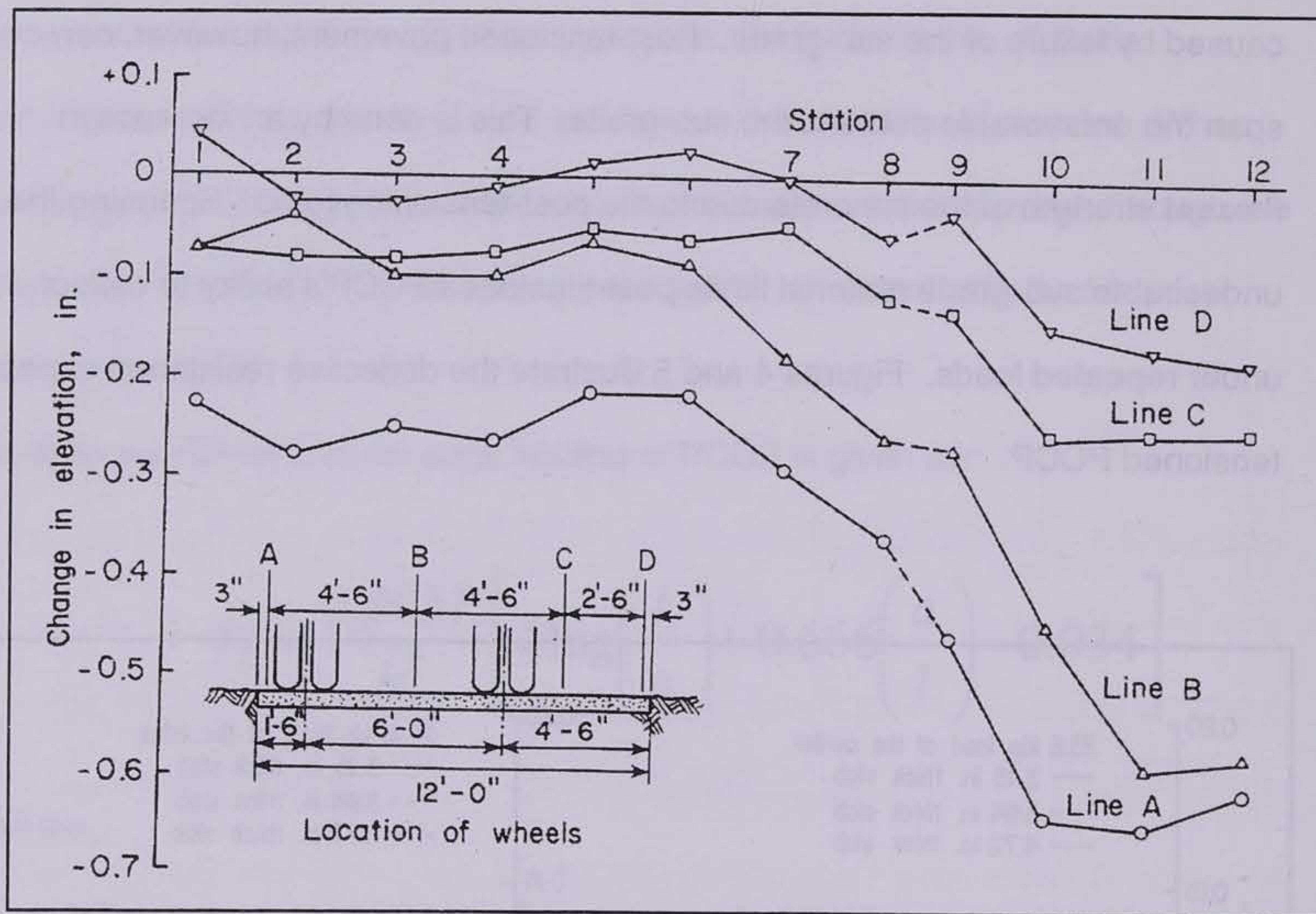


Figure 5 Change in Pavement Elevation After 580,000 Load Applications of 22 kips (Hanna, Nussbaum, Arriyavat, Tseng, Friberg, 1976)

TYPICAL DESIGN

In most post-tensioning PCCP projects the post-tensioning has been done in the longitudinal direction of the PCCP. This does have its advantages and disadvantages. One of the advantages of this type of design is the simplified procedure of placing the strands in the pavement. Most slip-form pavers can easily be adapted to accommodate the insertion of the post tensioning strands into the pavement. ADOT used this method when it post-tensioned part of the superstition highway near Phoenix. Longitudinally post-tensioning PCCP also applies a direct force perpendicular to the weak plane of the concrete, the transverse plane. This reduces joint spacing and cracks in the concrete.

Longitudinal post-tensioning does have its disadvantages. First, slabs can only be designed to be so long before post-tension losses such as those induced by the frictional stresses between the sub-grade and PCCP become so great they cannot be overcome unless more strands are added. For this reason, most post-tensioned slabs tensioned in the longitudinal direction are limited to less than 500 ft in length. Second, longitudinally post-tensioning PCCP requires that work areas, known as gap slabs, be constructed between two post-tensioned pavements, Figure 6. These areas are where the actual tensioning of the strands takes place. Contractors must have enough space in the gap slab to apply the tensioning force without bumping into the slab behind them. Gap slabs range in length from 6 to 10 feet. Amazingly, the 6 to 10 feet of gap slab are what cause the greatest discomfort to drivers. The post-tensioned sections of

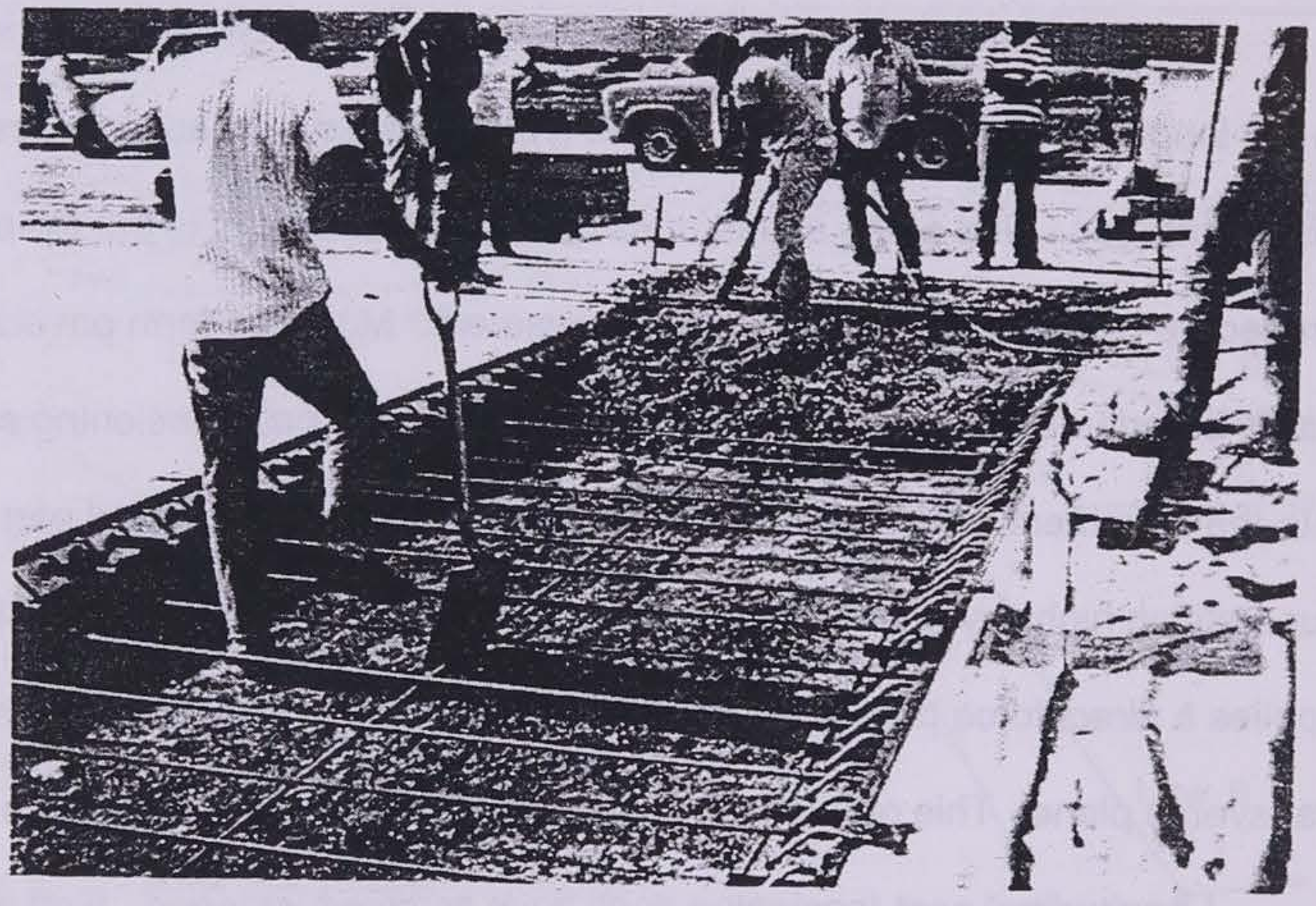


Figure 6 Gap Slab in Construction (ADOT)

PCCP are very smooth; however, the gap slabs constructed are very rough and deteriorate very quickly. The elimination of gap slabs could increase the riding comfort on post-tensioned PCCP. Eliminating gap slabs also eliminates an area where water infiltration is likely to occur. Many different joints have been developed to control the inflow of water at the transverse joint on either side of the gap slab. Most are either too difficult to install or too complicated to construct. A total elimination of the entrance of water into the transverse joints at the gap slabs has yet to be accomplished.

PROPOSED DESIGN

Cross-tensioning PCCP may give the designer the option to construct non-jointed, un-cracked concrete. The procedure for designing such a pavement is done in much the same as that of a PCCP post-tensioned in the longitudinal direction. The only difference is the interpretation of the frictional losses between the PCCP and sub-grade. In longitudinal post-tensioning design, the designer considers the pavement as having two ends that result in transverse joints. The length of the pavement between the two joints is the length of pavement that is subjected to the frictional force between the PCCP and sub-grade. In cross tensioning the designer post-tensions the pavement in both diagonal directions, figure 7. By cross tensioning, the PCCP cannot expand in any direction. This allows for the design of a non-jointed pavement.

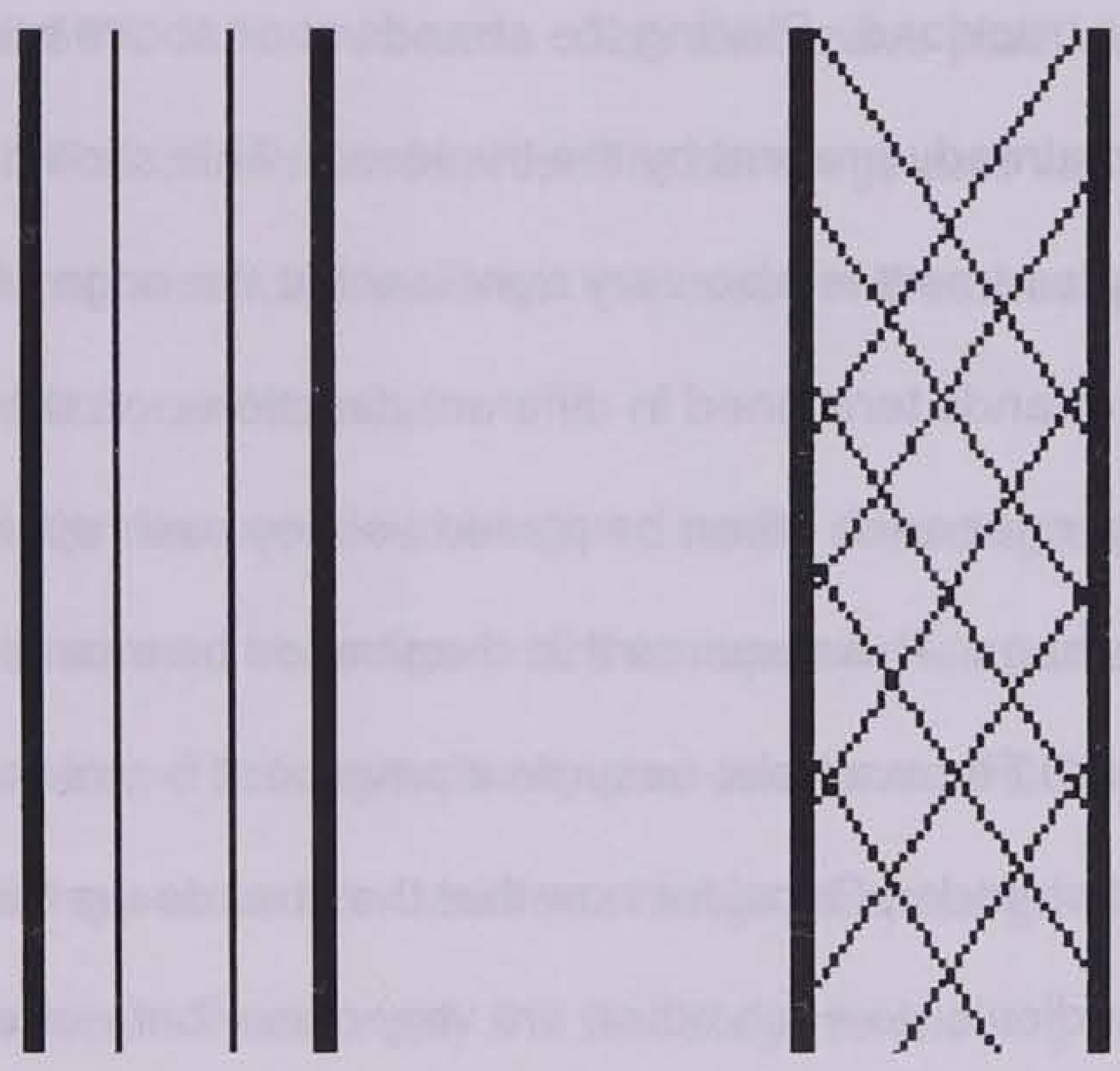


Figure 7 Longitudinal and Cross Tensioning

The angle at which the cross tensioning should be applied should be between 30 and 45 degrees to the longitudinal direction of the pavement. This allows the majority of the stress force in the strands to act against the weak plane of the concrete, the transverse plane. The 30 to 45-degree specifications also give the concrete less capability of cracking in the longitudinal direction, as did the ADOT PCCP. As a note it should be remembered that the lower the angle of post-tensioning, the less steel strands required.

Strand location in the slab is most important. Historically the strands of post-tensioning steel have been place 0.5 inches below the mid-depth of the slab. At this depth, the strands are able to carry very well the loads induced on them by heavy trucks. Lowering the strands to 0.5 inches below mid-depth causes the slab to have an induced negative moment. When a truck drives over the pavement, the induced negative moment is cancelled by the moment supplied by the truckload. Placing the strands at or above mid-depth will only add to the load already present by the truckload. This should never be done.

Strand placement is also very significant at the edge of the pavement. It is important that strands tensioned in different directions, on the same side of the pavement, very near each other, be placed so they each apply a compressive force between them. This requires that the strands be crossed very close to the pavement edge. For example, assume a pavement is stressed with 45-degree post-tensioned strands. Consider now that the strands are laid out such that two strands, perpendicular to each other, are very close, but **not crossing** at the pavement edge. This will cause a tensile stress in the concrete between the two strand ends. If the tensile stress is too great, the concrete may begin to develop a transverse crack. Now consider the same two strands as **crossing** very near the pavement edge. The only stress that will exist between the two strands in this case is a compressive stress. It has long been known that concrete cannot support extensive tensile forces. In fact, the ACI code tells the designer that the tensile strength of concrete should be neglected in all axial calculations. As a note the tensile strength of concrete is highly variable and is usually only considered to be 10 to 15 percent of the compressive strength.

The thickness of concrete used in post-tensioned PCCP is less than conventional design. Pavements as thin as 3.5 inches have been developed for use with longitudinally post-tensioned pavements. However, problems associated with coverage and uniform paving make this an unfavorable choice on most highway pavements. It is recommended by ACI that a pavement thickness of at least 65% of the thickness of an alternative plain concrete pavement be used. This allows for appropriate coverage and variations in construction. For most cases, a pavement thickness of 6 inches is sufficient for coverage and construction tolerances.

Special consideration needs to be given to the stressing operation. The strands cannot be tensioned to design until the concrete has gained sufficient strength. ADOT tensioned its longitudinally post-tensioned PCCP in three stages. Since cracking occurs from volume change, thermal gradient, and sub-grade restraint, it is imperative to apply the first stage tensioning to the cables at the earliest practicable time, usually within 24 hours. Some small cracks may form before initial jacking; however, they should close upon application of the jacking force. The second jacking should be done within 24 hours of the previous. Lastly, the final jacking should be done when the concrete strength has reached at least 3000 psi, independent of time. It is imperative to not jack the strands beyond the strength of the concrete. Jacking forces should be determined considering the size and thickness of bearing plate end anchors and minimum concrete strength necessary to withstand the applied force. It may be necessary to increase the end anchor bearing plate size to distribute the high bearing stresses at the slab edges.

Most suppliers of post-tensioning hardware have end connectors, such as the one shown in figure 8 that are easily adaptable to post-tensioning on angle. In addition, no special additions need to be made to the jacks that apply the post-tensioning force figure 8.

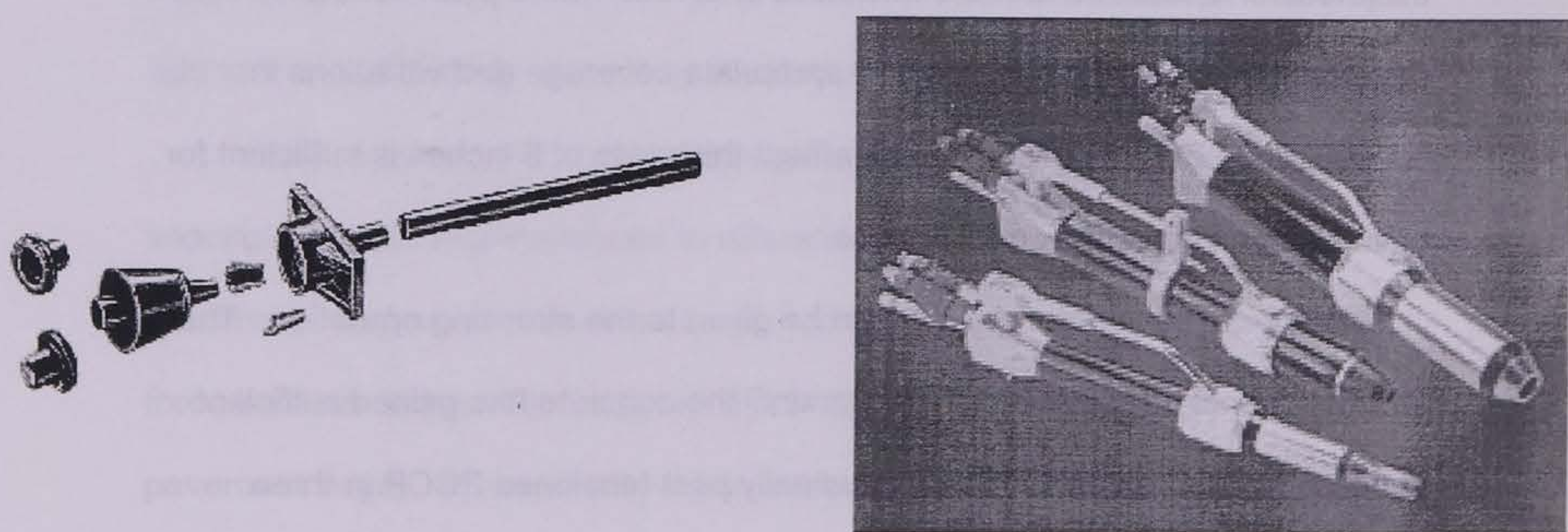


Figure 8 Post-Tensioning End Chuck Assembly and Jacks (VSL)

CROSS-TENSIONED DESIGN EXAMPLE

Applying the fundamental equation for the design of post-tensioned pavements to cross-tensioned pavements is done in much the same way as those pavements tensioned in the longitudinal direction. For this example, we will design a 6-inch concrete pavement that is 24 ft wide. It is assumed in the example that the designer will be using the maximum angle of post-tensioning, 30 degrees.

Solving for the fundamental factors, we get the following stresses:

$$f_{\Delta t} \quad = \quad 255 \text{ psi}$$

$$f_t \quad = \quad 80\% \text{ of MOR} = 440 \text{ psi}$$

With $P = 5500$ lbs., $k = 100$ pci, $l = 29.3$ in., tire pressure = 100 psi:

$$f_L \quad = \quad 422 \text{ psi}$$

With $c = 0.5$:

$$f_F \quad = \quad 12.08 \text{ psi}$$

Now,

$$f_p \quad = \quad 255+12+422+\text{-}440 = 249 \text{ psi} \approx 250 \text{ psi}$$

The value of f_p is the minimum amount of compressive stress required in the concrete in any direction to overcome the effects of warping, friction, and traffic loads. Therefore, the stress force required per ft. width of pavement is:

250 psi x 6 in. depth x 12 in./ft. = 18,000 lbs./ft

Across the entire pavement, the total force required is:

18,000 lbs./ft x 24 ft = 432,000 lbs.

Since the compressive force is going to be applied at an angle of 30 degrees, we need to break the compressive force required into components.

For a 30 degree skew:

432,000 lbs./COS (30) = 498831 lbs.

In addition, the force being applied is being applied from the two different sides of the pavement. Therefore, we can split the force required into two pieces. This will allow the strands from both sides of the pavement to contribute to the total force required in the concrete.

Splitting the force yields:

498,831 lbs./2 = 249,416 lbs.

The strands applying the force will be tensioned to the fullest capacity, however losses do occur due to relaxation of the strands. ACI recommends that the effective stress in the strand not exceed 80% of the ultimate stress. Effective stress is the stress in the strand after all losses have occurred. The effective stress is computed as:

80%x (270 ksi ultimate strength) = 215 ksi

Using 0.5 inch diameter strands with an area equal to 0.153 sq. in. the effective strength of each strand can be computed as:

0.153 sq. in. x 215 ksi = 32.895 kips = 32895 lbs.

The minimum number of strands need to cross a transverse plane in the pavement can be computed as:

$$\frac{249415\, compressive\ lbs.\, req.}{32895\, lbs.\ per\ strand} = 7.58 \approx 8\ strands\ from\ each\ side$$

The spacing of the strands at the transverse section should be will vary. Using geometry and a minimum of 16 strands per transverse section, it can be determined that a spacing of 5 ft between strands tensioned in the same direction will work along the pavement edge.

It is important to remember the crossing of the strands near the pavement edge as discussed earlier. For the purposes of this design it is assumed that a cross overlap of 2 feet will work. This is an assumed value and more research should be done to determine this nominal distance. The spacing of the strands may need to be adjusted to accommodate any change to the cross overlap. Anyhow, 2 feet should give the constructor enough working space to apply the post-tensioning force, while at the same time keeping the concrete in compression.

CONCLUSIONS AND RECOMMEDATIONS

Further studies and actual field test of cross-tensioned PCCP should be done before any attempt is made to apply the theory to actual PCCP. The most extensive research should be done in the areas where the strands cross each other near the pavement edge. At this location, little is known about the behavior of the concrete. Studies should also be done on the behavior of the concrete at any location where strands cross each other. Furthermore, researchers should study the effects of applying a skew force to the strands. This will cause some side force on the strand near the pavement edge. Additionally, the construction of cross-tensioned pavements will need to be reviewed. Some new and innovative construction techniques may need to be developed for the installation of the crossed strands.

Post-tensioned PCCP of the cross-tensioned variety may be a suitable solution to the problems associated with conventional PCCP. Post-tensioned PCCP can reduce pavement thickness, reduce pavement deflection, allow for non-jointed design, and increase the load capacity of PCCP. As with any new design, this one will need to be proven. It is, however, possible that cross-tensioned PCCP could be everlasting pavement for the 21st Century.

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>From root Wed Nov 3 11:55:37 1999
Date: Wed, 03 Nov 1999 23:57:22 -0600
From: Jeffrey Hancock <jefhan@ksu.edu>
Organization: Kansas State University
X-Mailer: Mozilla 4.6 [en] (Win95; I)
X-Accept-Language: en
To: prochnow@iastate.edu
Subject: CTRE Paper Contest

Sharon,

Can I send you a final version of my paper for publication in the compendium. There have been some major changes made to the version you have. By the way I will need a computer screen projector. I have my own laptop. I will be attending the lunch.

Jeffrey

jefhan.vcf

The Need for Automated Surface Distress Survey Collection in the Iowa Statewide Pavement Management System

Kyle Evans
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Iowa State University

ABSTRACT

This research was concerned with the use of technology to improve the timeliness, objectivity, and consistency of the pavement distress surveys carried out throughout the state. Through the use of automated collection systems Iowa will save money and improve the overall highway network that makes Iowa a thriving industrial state.

At the time of this research, highway needs were calculated using a computer model called HWYNEEDS. The HWYNEEDS software, developed by the Federal Highway Administration, is a part of the Quadrennial Needs Study. This software is a planning and resource allocation tool used by the state to determine 20-year funding needs for the many highway systems, including the secondary system, in the state. The main inputs into the analysis include highway operational, safety, and surface condition data to calculate the present and future needs. The quality of condition ratings has become a major concern for many county engineers over the last ten years. Because of the inconsistency in condition ratings, many counties have experienced funding fluctuations of almost 30 percent between Needs Studies.

By furthering the use of the automated distress collection systems the state will be able to limit the amount of funding problems experienced by the many counties. This will be accomplished by providing a more consistent and reliable set of data through the use of laser sensors and computer based data processing.

INTRODUCTION

This paper dealt with the deficiencies associated with the current method of data collection by the Iowa Department of Transportation. The main focus of concern dealt with the fact that Iowa's statewide Pavement Management System has not kept pace with the growing technology leading to fluctuations in the amount of funding received by the counties.

At the time of this paper, highway needs were calculated using a computer model called HWYNEEDS. HWYNEEDS is a part of the Quadrennial Needs Study, which is a planning and resource allocation tool used by the state to determine 20-year funding needs for various highway systems including the secondary system. The pavement surface rating was previously identified as an input causing inconsistencies in the resulting highway needs. The pavement surface ratings are manually collected for each county once every ten years. The manual collection procedures are not specific and timely enough to provide consistently accurate data. Due to the fact that the State is not applying the latest technologies in collection methods, many county engineers have expressed concern over the validity of the results calculated by the current system.

The flowchart (Figure 1, page 2) gives a graphical representation of the chronological steps the Iowa Pavement Management System has taken from Iowa's first notion of highway needs to the present.

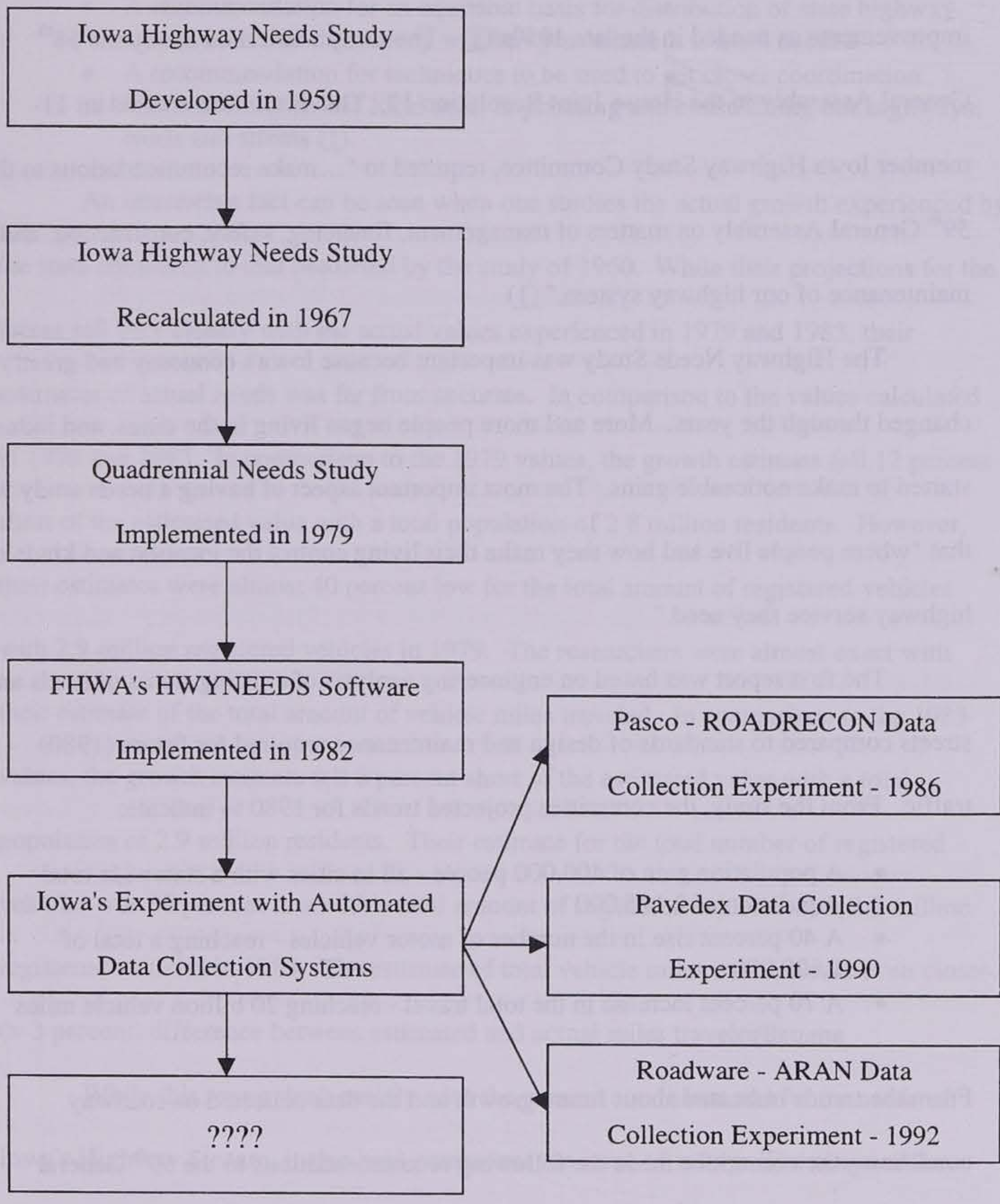

Figure 1: Iowa's Pavement Management History

IOWA HIGHWAY NEEDS STUDY

Iowa first identified the need to study the roadway conditions and make improvements as needed in the late 1950s (1). The study was called for by the 58th General Assembly in the House Joint Resolution 12. This resolution created an 11-member Iowa Highway Study Committee, required to "...make recommendations to the 59th General Assembly on matters of management, financing, safety, construction, and maintenance of our highway system." (1)

The Highway Needs Study was important because Iowa's economy had greatly changed through the years. More and more people began living in the cities, and industry started to make noticeable gains. The most important aspect of having a needs study was that "where people live and how they make their living control the location and kinds of highway service they need."

The first report was based on engineering analysis of existing status of roads and streets compared to standards of design and maintenance required for future (1980) traffic. From the study, the committee projected trends for 1980 to indicate:

- A population gain of 400,000 people - all in cities with a statewide total population of 3,125,000
- A 40 percent rise in the number of motor vehicles - reaching a total of 1,800,000
- A 70 percent increase in the total travel - reaching 20 billion vehicle miles annually

From the trends indicated about future growth and the data collected on roadway conditions, the committee made the following recommendations to the 59th General Assembly.

- A recommendation for sound legislative policies and management practices to be followed in primary highway construction and maintenance in view of the increase in federal funds for interstate highways
- A recommendation for an equitable basis for distribution of state highway revenues so that this money will be spent where it is most needed
- A recommendation for techniques to be used to get closer coordination between the state and local units in planning and constructing our highways, roads and streets (1).

An interesting fact can be seen when one studies the actual growth experienced by the state compared to that predicted by the study of 1960. While their projections for the future fall very closely with the actual values experienced in 1979 and 1983, their estimates of actual needs was far from accurate. In comparison to the values calculated in 1979 and 1983. In comparison to the 1979 values, the growth estimate fell 12 percent short of the estimated value with a total population of 2.8 million residents. However, their estimates were almost 40 percent low for the total amount of registered vehicles with 2.9 million registered vehicles in 1979. The researchers were almost exact with their estimate of the total amount of vehicle miles traveled. In comparison to the 1983 values, the growth estimate fell 8 percent short of the estimated value with a total population of 2.9 million residents. Their estimate for the total number of registered vehicles was 36 percent low with a total amount of registered vehicles with 2.8 million registered vehicles in 1983. The estimate of total vehicle miles traveled was even closer (> 3 percent) difference between estimated and actual miles traveled.

While this report dealt mainly with the present conditions and future needs of Iowa's Highway System, it also held paramount the issue of safety. The scope of the study included using all phases of engineering, whether it was design, construction, or maintenance issues, to cut the accident and death toll. In order to promote safety issues, the committee outlined a proposed rural and urban freeway system, the safest highway design, and improved design standards, on all roads and streets, encompassing the safest designs available.

The Need Study process consisted of several steps. These steps include determining functional classification, establishing design guides, collecting inventory data, performing adequacy appraisal, and estimating costs.

By determining the functional classification of the roadway, the committee was able to determine the minimum requirements necessary to keep the roadway in satisfactory condition. The purpose of behind this step was to establish design guides for each functional classification that met universal design standards for new construction.

The main focus of this paper deals with the collection method of the second step in this process. The inventory data includes a complete inventory of all roads, structures, and railroad crossings in the state. Specific information collected about each of these items included geometric data, traffic data and condition data.

The adequacy appraisal was completed to determine whether or not there was a need. For this, each asset is analyzed to identify deficient operational, safety and condition elements over the 20-year study period. For this, an asset becomes deficient when it falls below a serviceable level based on the requirements set forth by the functional classification and traffic. Also, an appropriate improvement is selected to correct the deficient element.

The final section consists of the unit costs based on surveys of all 99 counties and 133 cities. From these surveys, Cost Area Adjustment Factors are developed to relate to the varying unit costs for materials and labor across the state.

However, while this study was a breakthrough for the state in terms of devotion to maintaining safe highways and streets, it was determined that the study had already become obsolete and outdated by 1966. The main reason for the discrepancy is that the needs alone had grown by nearly 56 percent between 1960 and 1966. This increase was due to the fact that the State was not meeting the needs in a timely fashion. The 1960 study showed that in order to meet the State's needs, $4 Billion of new construction was needed. However, due to the fact that Iowa was not on pace with this estimate, the estimate increased by over $3 Billion (2).

In 1960, it was estimated using traffic demand projections that the state would only need 1400 miles of four-lane highway. New projections were completed in 1967 that showed that the state would actually need in excess of 2400 miles of four-lane highways. With these increasing traffic demands, all highway classes would have to be built to higher design standards to with stand the heavier traffic demands. Finally, the general cost index was increasing which meant that the previous estimate would be proportionately higher using the cost index from 1967.

The authors of the 1967 report made some interesting assumptions in their development of the State's highway needs. According to their report, they never conducted a field review of the roadway condition. They based their report off of the findings of the 1960 report in conjunction with a thorough analysis of geometric and sufficiency-rating records maintained by the Highway Commission (2).

The 1967 commission did state that using a needs analysis period of 20 years was not acceptable. They stated that twenty years was too long to wait for adequate roadways because the traffic continued to grow and that the social and economic costs of having in adequate roadways represented a serious loss for residents. While the committee never made a suggestion as to what length of time was adequate. It was noted that South Dakota and Nebraska were following an expenditure program that would provide for adequate highways in less than 15 years.

QUADRENNIAL NEEDS STUDY

The State of Iowa continued to struggle with the immense task of collecting roadway condition data in a timely manner. After the problems associated with the Needs Study of 1960 where made obvious with the Needs Study of 1967, the General Assembly created the Quadrennial Needs Study. The General Assembly made the necessary changes and in 1979, an addition was made to the Code of Iowa in Chapter 307A.2(3). This part of the Code of Iowa dealt with the Transportation Commission Duties and stated that "For the four-year period beginning July 1, 1979, and for each subsequent four-year period, prepare, adopt and cause to be published the results of a study of all roads and streets in the state" (4).

What this meant was that the DOT would publish a report every four years declaring the condition of the roadways in the state. In addition the study represents the cost of completing existing construction needs, as well as those needs estimated to accrue during the next 20 years, on all Iowa road and street systems. (Quadrennial Needs Study for study years 1994-2013) The designated purpose of the Need Study was to:

- Determine the long-range (20 Year) funding needs for the state, county, and city road systems
- Provide factors for determining annual road use tax allocations to counties
- Provide factors for determining annual allocation of funds to state parks and institutional roads (5).

By developing the Quadrennial Needs Study the General Assembly hoped to supply the DOT with the necessary data in a timely matter. However, due to the method with which the Pavement Condition Ratings were collected, this goal was still unattainable on a statewide level. The Quadrennial Needs Study was considered to be way ahead of its time, however, the state was still in need of a data collection system that could collect the data in a faster, more subjective method.

FEDERAL HIGHWAY ADMINISTRATION HWYNEEDS SOFTWARE

The Federal Highway Administration (FHWA) began developing a system, for use nationwide, during the 1970's. During this time, emphasis was being placed on simplifying the planning process. Many states and local agencies were trying to determine the best method by which to develop highway programs, allocate funds and comply with mandates. The work of the FHWA was to focus the diverse efforts of the many agencies to a centralized, standard goal. At the time, the Iowa system was highly regarded and therefore, the FHWA patterned their system much like that of the Iowa system. The outcome of the FHWA's efforts was an effective planning tool that would later be known as HWYNEEDS. This tool would allow for highway agencies to:

- Assess highway needs with various levels of effort
- Reduce the cost of conducting needs studies
- Provide flexibility in needs study procedures in terms of input requirements and evaluation criteria
- Facilitate the rapid updating of needs estimates
- Relate needs evaluations to other planning activities such as budget allocation, project evaluation and selection, programming, and scheduling
- Determine the sensitivity of various elements of the needs study procedures in terms of their effect on the resulting estimates (Estimating Long-Range Highway Improvements)

Iowa determined the benefits by switching to the FHWA program and in 1982 the method by which the needs were calculated was changed from the Iowa based computer program system to that of the HWYNEEDS software (6).

PAVEMENT MANAGEMENT SYSTEMS

In order to have more efficient control over the roadway conditions with the limited budgets, the nation needed to rely on a more accurate system of long-term design, maintenance and rehabilitation. Thus, the idea of pavement management was born. The term Pavement Management System was coined during the 1960's by researchers working on different projects at the University of Texas, the Texas Transportation Institute at Texas A&M University, and the Texas Highway Department (7). The researchers begin looking at pavements and designing them in more of a systems approach (8). The basic framework for a Pavement Management System has remained intact since its development in the late 1960's. The general flow chart of a Pavement Management System closely resembles that of Figure 2 (next page).

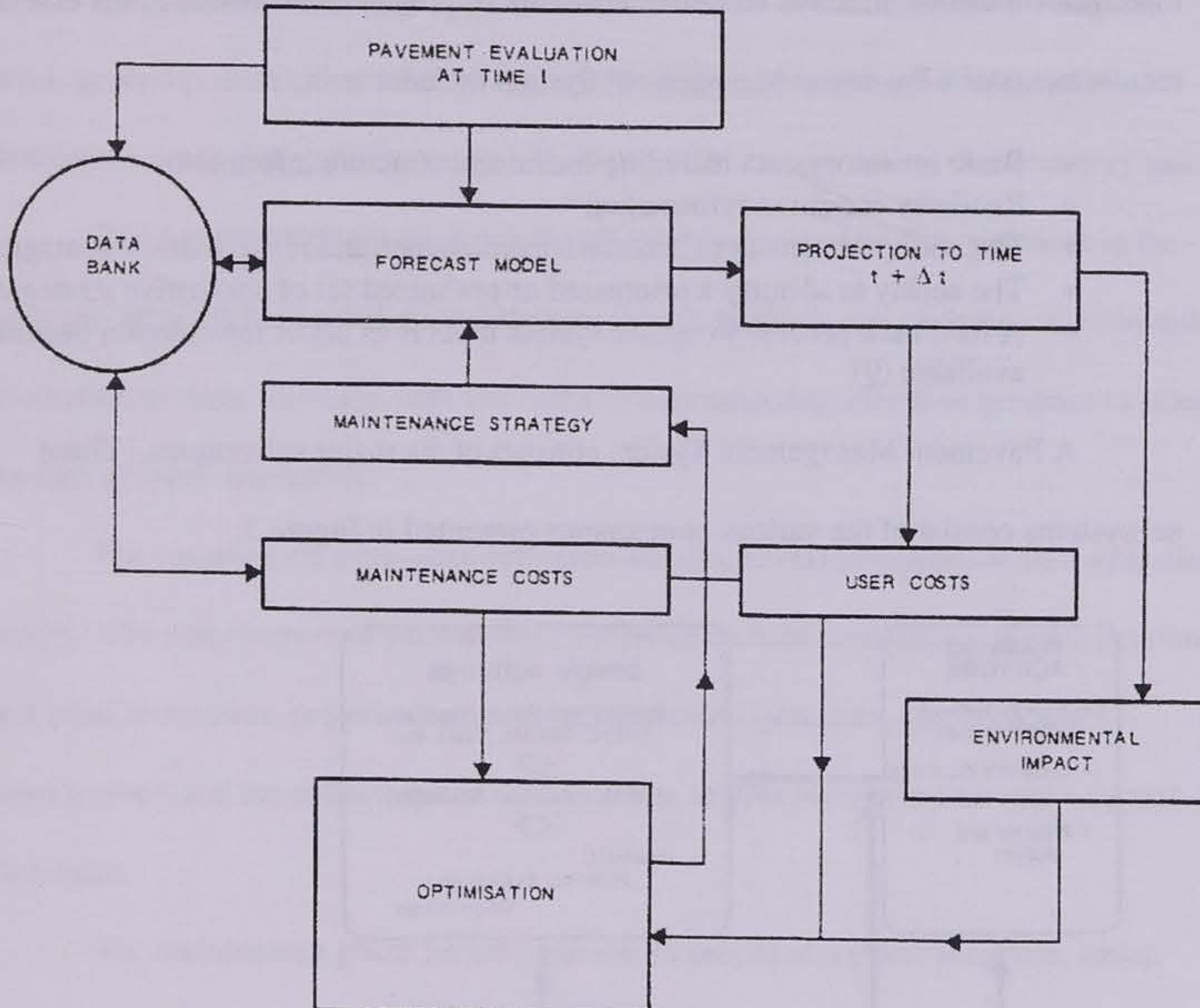

Figure 2: Pavement Management Flowchart

Figure 2 provides a summary framework for pavement management. This figure gives an overview of the interaction between the many various activities, and points out that the basic functions of the PMS are to:

- Collect inventory, condition, and cost data
- Assign strategies, identify needs, and arrange priorities
- Project future needs and build long-range programs
- Provide management information
- Support budgets (8)

In order to be an efficient system, it must be capable of being used in whole or part by various levels of decision making. This relates to the general idea that pavement management can be effective on either a network or project level. Some of the essential requirements of a Pavement Management System include:

- Basic inventory data including traffic and structure information
- Roadway condition information
- The ability to consider alternative maintenance and rehabilitation strategies
- The ability to identify a prioritized or optimized set of alternative strategies
- A feed back process to update system models as better information becomes available (9)

A Pavement Management System consists of six major subsystems. These subsystems consist of the various components presented in Figure 3.

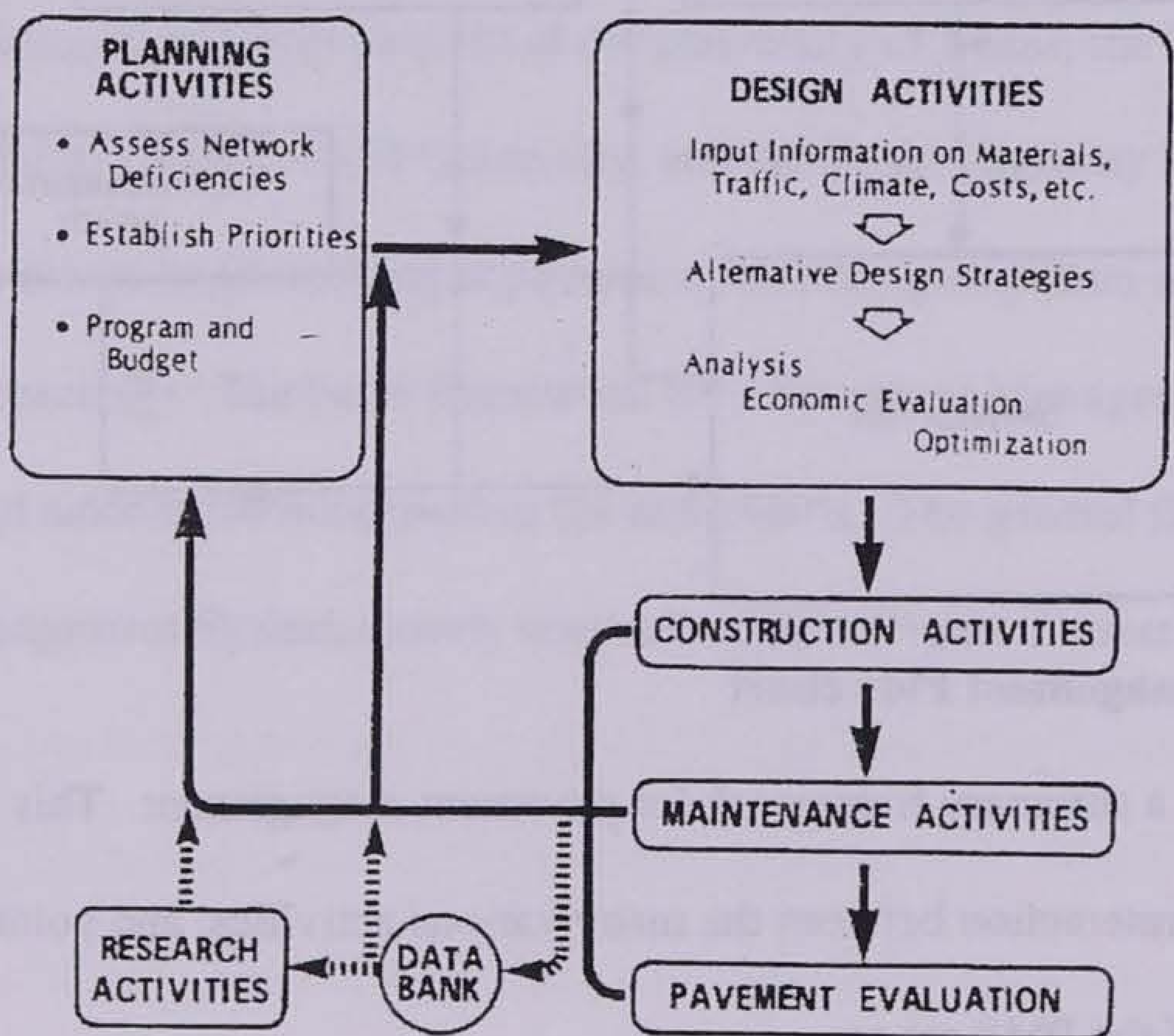

Figure 3: Six Subsystems of a PMS

Each of the six subsystems presented in Figure 3 are interrelated. Each one combines a variety of major and minor problems that are capable of being studied and solved using a general analytical system. An overall understanding of the inter-workings of the Pavement Management System can be established by outlining the function of each subsystem.

The planning subsystem the general needs of the overall system on a network level, general prioritization for reducing or eliminating unnecessary needs, and the development of a maintenance routine and budget for accomplishing the necessary tasks.

The design subsystem includes the collection of historical data pertinent to the network, all available and adequate alternate design methods, the analysis and economic evaluation of these alternate methods, and a prioritization/optimization program to select the best possible alternative.

The construction subsystem integrates the design recommendation into a physical reality. The most important parts of this subsystem include finalization of specifications and legal documents, project scheduling, construction operations, quality control management, and the collection and interpretation of data for transmittal into a central data bank.

The maintenance phase includes the development of a repair schedule, which includes the implementation of a crack sealing program, patching and any other measure that must be taken in order to maintain the condition of the pavement and extend its service life. In addition to these tasks, the maintenance phase is also responsible for collection and interpretation of data for transmittal into a central data bank.

The pavement evaluation subsystem is a part of the Pavement Management System has received enormous amounts of attention. Again the point is raised that Iowa needs to improve the technology implemented in the acquisition of pavement condition data. The user of the Pavement Management System can use this collected data to:

- Checking the adequacy with which the pavement is fulfilling its intended function
- Planning and programming future rehabilitation needs
- Improving the technology of design, construction, and maintenance

In 1985 the American Association of State Highway and Transportation Officials (AASHTO) developed the Guidelines on Pavement Management. According to these guidelines, pavement management is the effective and efficient directing of the various activities involved in providing and sustaining pavements in a condition acceptable to the traveling public at the least life cycle cost. It is also an ongoing activity that has always existed in transportation agencies and that transcends the traditional boundaries dividing these agencies into their organizational units (7).

Pavement management is important because during the last two decades, the emphasis has shifted from expansion to preservation and rehabilitation of the existing highway system. The competition for legislative funding is very intense and will probably remain so in the future. Also, many pavement decisions are based on the judgement and experience of engineers and administrators in different organizational units, as well as varying, inconsistent sources of information.

In order to continue being cost effective and beneficial, the technology with which a pavement management system is maintained must continually be updated much like the technology used in constructing roadways. The reason this is important is that many agencies are unable to document fully the overall condition of their highway networks. Frequently, they are unable to demonstrate specifically to the public and legislative bodies how funds are funds are used and what benefits could be attained with additional funding (7).

The guidelines also suggested a general approach for improving pavement management systems. It suggests a simple four-step process that consists of reviewing existing operations, policies, and organizational structure. Analyzing existing operations and compare them with those of preferred Pavement Management program ensuring that they are up-to-date, valid, and efficient. Identify any needed improvements and finally implement improvements that are necessary.

Along with the four-step improvement process, the guidelines give some suggested actions that can be taken to enhance an existing pavement management system. The most important actions mentioned are:

- Making a top management decision to install a pavement management system
- Devise a preliminary plan
- Develop a specific plan to "manage" a portion of the highway network as a test section
- Develop or improve pavement-rating procedures, reviewing technical developments in pavement survey equipment and automation
- Set up a database and collect the necessary data for the portion of the network selected
- Periodically evaluate the effectiveness of the pavement management system

The major concern over Iowa's Pavement Management System is the method in which the Pavement Condition Ratings are collected. The method in use relies on the collection of data for all of the roadways on a rotating ten-year schedule. Therefore, a roadway assessed in 1979 would not be reassessed until 1989. So that at the time of the publication of the 1986 Quadrennial Needs Study, the roadway data for some sections of the state would be almost eight years old. This was viewed by many County Engineers as unacceptable. With such a gap between survey periods, small changes made by the counties, such as simple thin overlays, would improve the rating of their pavements thus reducing their available funds. This had negative effects for certain counties where their needs were still very high, yet their funding was being reduced by over 30% in some cases.

PAVEMENT DISTRESS SURVEY SYSTEMS

Iowa has done a fairly good job of maintaining an effective pavement management system, however, they haven't done a good job of implementing new technologies. Iowa has experimented in the past with several semi- and fully automated surface condition collection systems. Some of these experiments include the Pasco, PAVEDEX, and Roadware collection systems. However, at the present time the Iowa DOT still relies on the method of manual inspection and condition rating.

Past pavement condition evaluations have been accomplished using information collected about the surface condition, structural integrity, and history of the pavement section so that a proper maintenance schedule could be implemented. The Iowa DOT determines the amount physical distresses in terms of cracking and patching by sending a crew out into the field to do a visual survey of the pavements. A Roadmeter is used to measure the degree and extent of rutting, if present, in the surface. Using these values, the Iowa DOT is able to obtain a value or Present Serviceability Index (PSI).

The PSI is a universal measure that is measured on a scale of 0 to 5 with a 5 exhibiting little to no distress and a 0 showing no positive characteristics. The PSI is generally made up of two parts. The first being the "ride" or smoothness of the roadway surface and the second being the amount of cracking, patching, and rutting. Placing a group of researchers in a vehicle and allowing their undersides to be the determining factor of the overall "ride" quality of the roadway initially developed the PSI. This test was used during the well know American Association of Highway Officials (AASHO) Road Test in Ottawa, Illinois from 1958-1961.

In an effort to improve upon the method in which distress conditions are collected, the Iowa DOT has experimented with three separate automated distress survey systems. These three systems included the Pasco - ROADRECON System, the PAVEDEX System, and the Roadware - ARAN system. These experiments have been carried out from 1988 to 1998. Each system has shown some general benefits in comparison to the method of manual distress data collection, however, due to the costs or constraints, none of the three systems have been implemented.

Pasco-ROADRECON SYSTEM

The Pasco - ROADRECON System was developed in Japan between 1970 to 1985. Each system improvement was named after the year in which it was improved. The system is semi-automated and involves the use of photographic equipment to take a continuous picture of the roadway surface. The ROADRECON system is used to survey the roadway condition at night, because it is felt that by controlling the lighting, the system can reduce erroneous data based on shadows caused by sunlight. There are several parts to the ROADRECON system that helped make it a very valuable system. The following paragraphs encompass the major components of each of the different system sections.

The ROADRECON-70 system (See Figure 4 following page) consists of a high-speed, continuous recorder using a 35-mm slit camera. The system was synchronized so that the film speed, vehicle speed, and the amount of illumination optimize the quality of the data collected. This system is limited to the collection of visual surface distresses only.

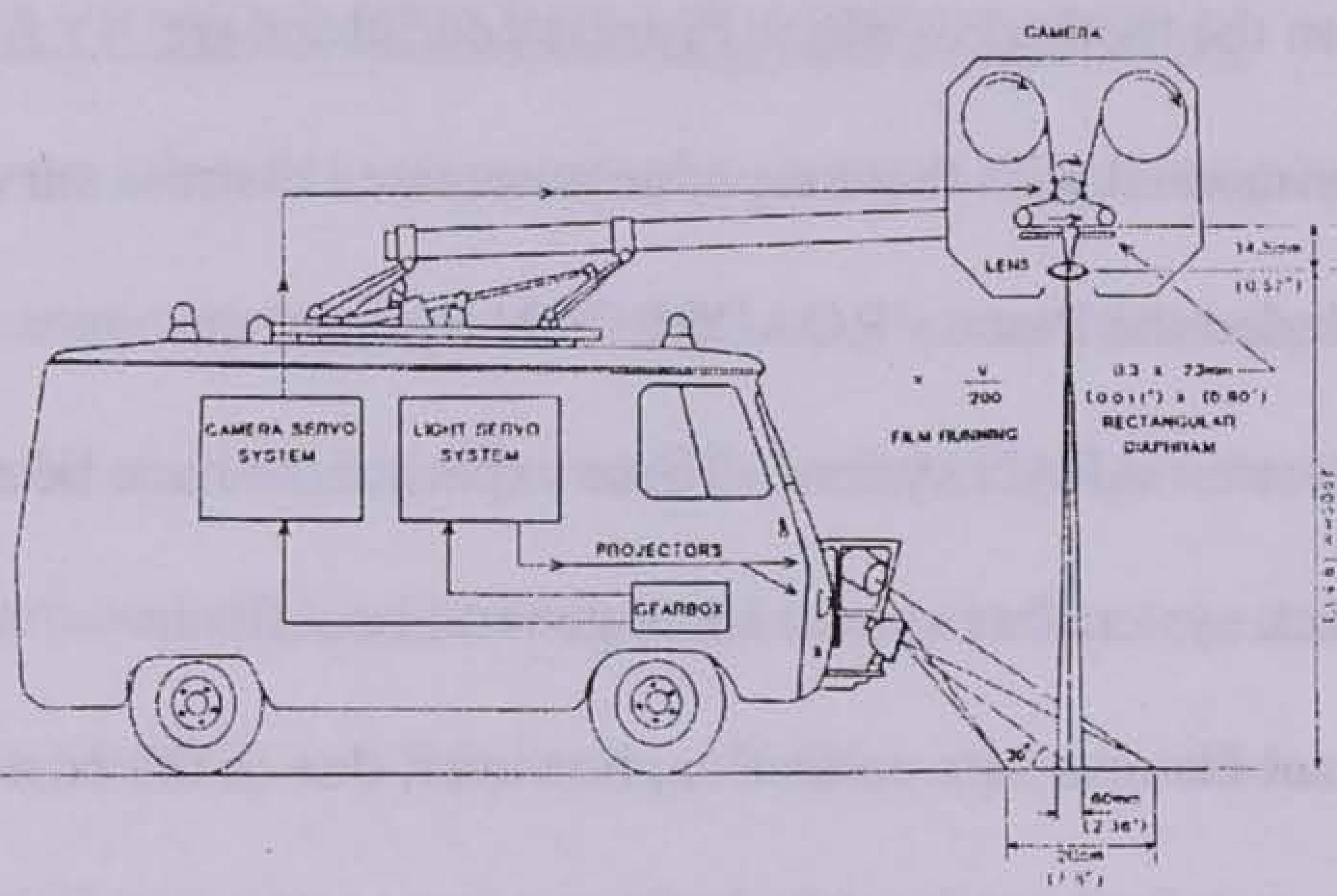

Figure 4: Pasco- ROADRECON-70 System

The ROADRECON-75 improved upon the earlier system. This was accomplished by implementing the use of a pulse-camera mounted on the rear of the vehicle (See Figure 5). With this camera set perpendicular to the roadway and a projector set to an angle of 26° 33' the system was capable of measuring the depth of any rutting that might be present in the roadway (10).

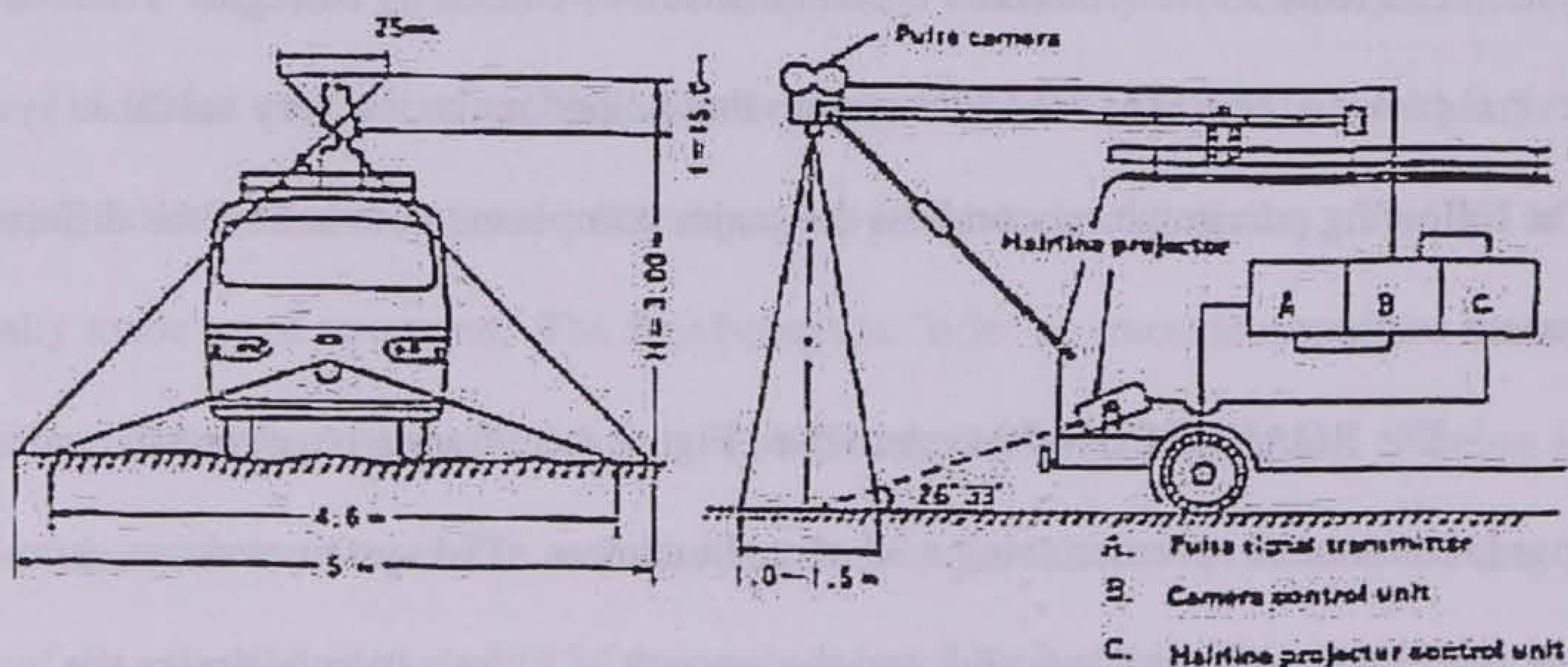

Figure 5: Pasco ROADRECON-75 System

The ROADRECON-85B System was upgraded to include the use of three laser sensors (See Figure 6 following page). These laser sensors were mounted onto the lower side of the rear bumper of the vehicle. The use of these sensors allowed the system to measure longitudinal roughness, joint faulting, and rutting at speeds ranging from 0 to 50 mph.

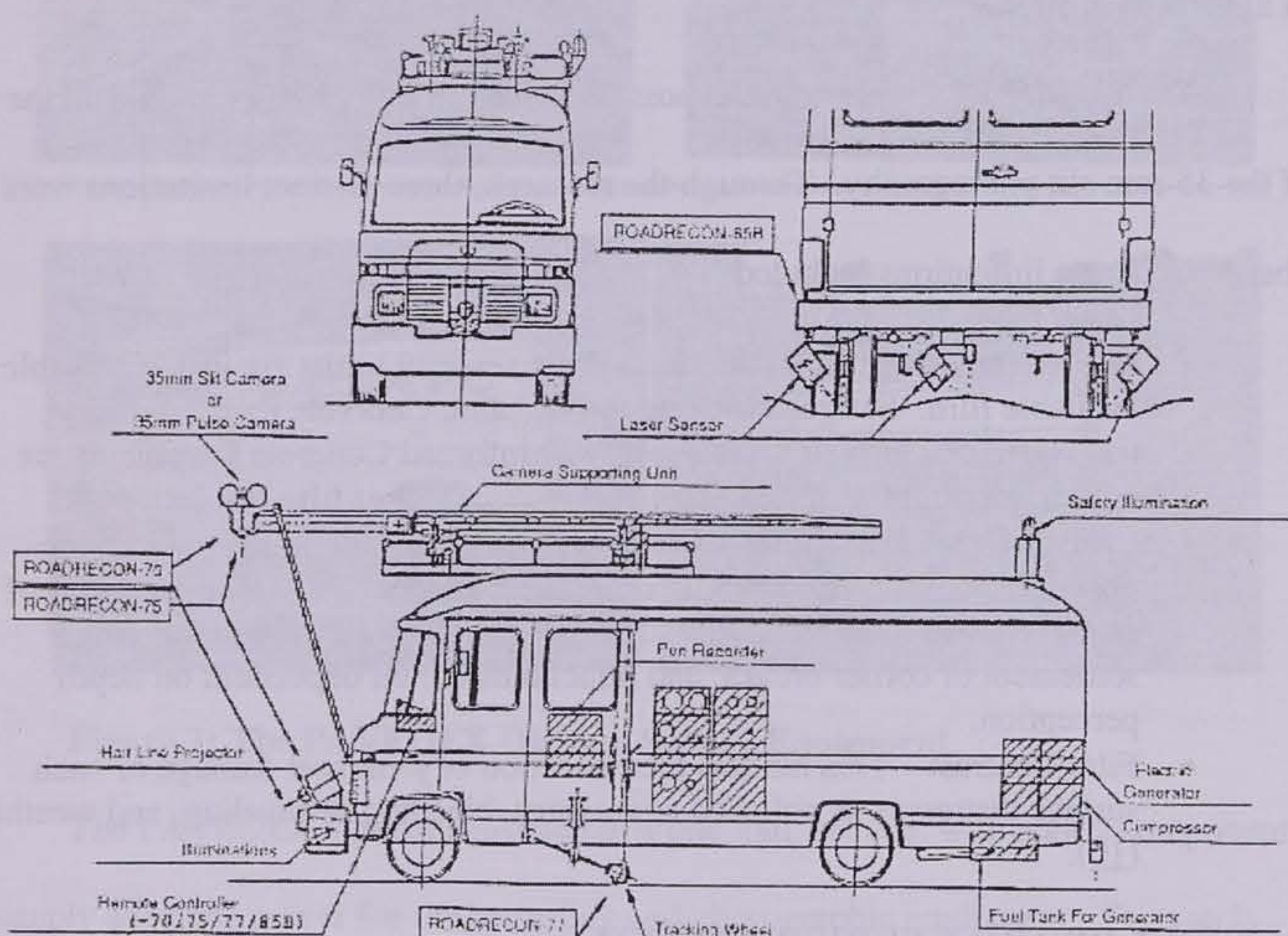

Figure 6: Pasco ROADRECON-85B System

Through testing and experimentation it was determined that the Pasco-ROADRECON vehicle could speed up the filed data collection time. In addition, the use of the vehicle would provide a permanent visual record of the actual pavement conditions viewed in the field. The main system relied on continuous 35-mm slit photography to provide the video images of the roadway. All surveying was performed at night using on board lighting in an effort to reduce the amount of visible shadows on the roadway surface. With the updates made to the ROADRECON-75 System, the vehicle was then also capable of developing rut depth surveys at speeds of 50 mph.

The ROADRECON System consisted of a semi-automated survey collection system. The film version of the survey is then processed in a central location using a digitizing process that allows a trained observer to easily record the number of distresses in a roadway section.

The ROADRECON System did possess some limitations associated with the use of the 35-mm slit photography. Through the research, three distinct limitations were observed. These limitations included:

- Dependency on Film Resolution – Low severity limits are just not visible with available film. Fatigue cracking in Asphaltic Concrete Pavements and transverse cracking in Continuously Reinforced Concrete Pavements are common examples of distresses that require higher film resolutions.
- Lack of Depth Perception – Distress types such as faulting and Lane-to-Shoulder drop-off are virtually impossible to identify. Some instances of severity levels are hard to classify. These include depth of potholes, settlement of corner breaks, and punchouts are all dependent on depth perception.
- Film Contrast – This hinders identification of joint seal damage or such surface distresses as polished aggregates, bleeding or raveling, and weathering (10).

PAVEDEX DISTRESS SURVEY SYSTEM

During the late 1980's and early parts of 1990 the Iowa DOT experimented with a semi-automated pavement condition rating collection system known as the PAVEDEX Road Survey System. The system was considered to be semi-automated, because while it did use video collection systems, the photos were analyzed by a trained observer using dual computer monitors to identify the different types of distresses and the extent of their coverage on the roadway. See Figure 7 (next page).

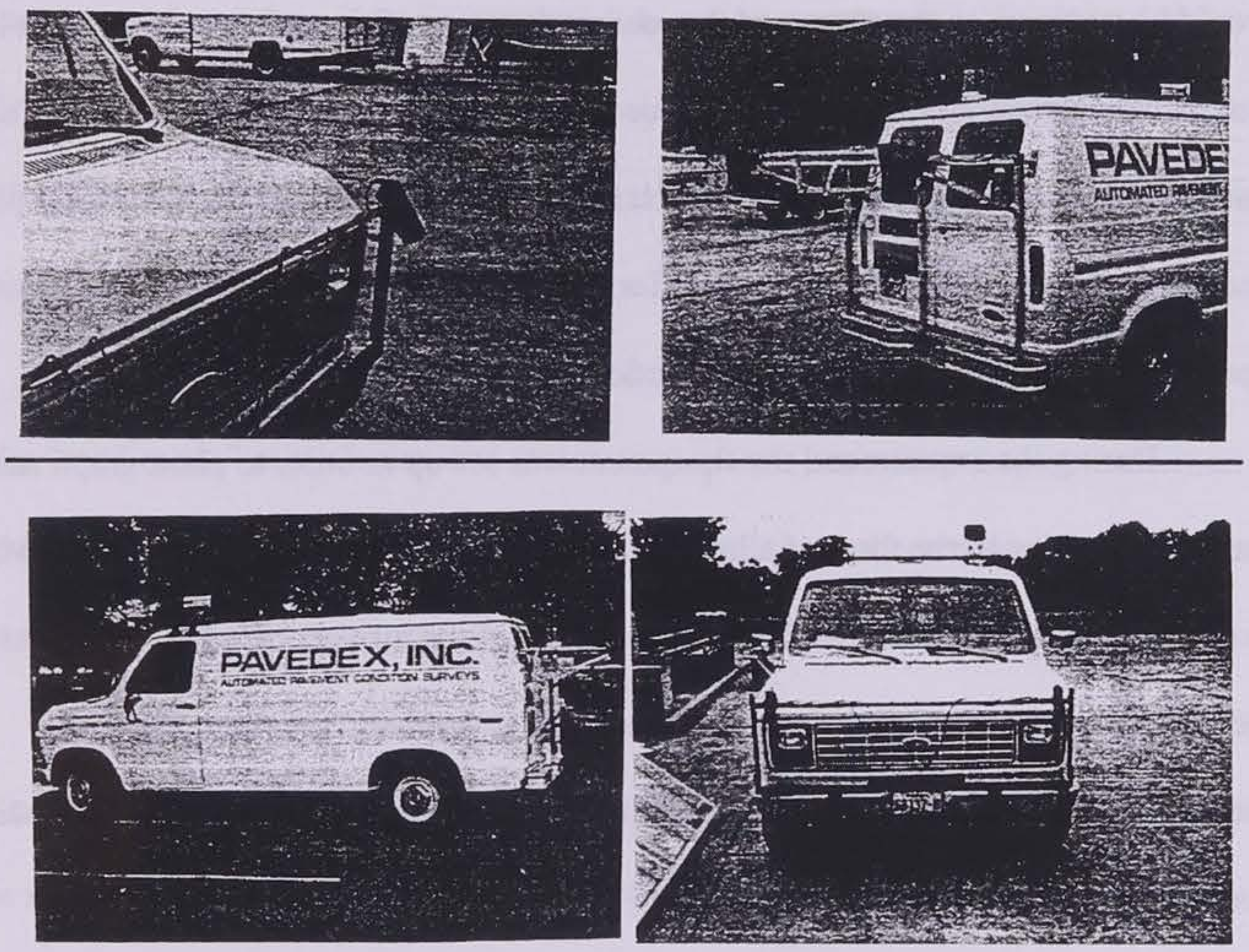

Figure 7: The PAVEDEX Distress Survey Equipment

The PAVEDEX system consisted of a one-half ton van, with added components to supply adequate power for the recording and photographic equipment. The van is equipped with five NEC Charge Coupled Device (CCD) cameras. Two cameras are mounted on the front and back of the van. The front two cameras are mounted at a 20-degree angle to the vertical, the rear cameras are positioned at a 15-degree angle to the vertical, and the fifth camera is roof mounted to provide a forward facing view. The equipment is synchronized so that any set of two cameras can be viewed simultaneously on the monitors (11).

The van is also equipped with three VCRs, one for the overhead camera and one each for the front and rear cameras. A Dictaphone was also located near the driver of the vehicle. This allowed for the driver/operator to add any additional information that would be pertinent to the survey of the roadway system. Other equipment present in the van include a generator, digitizer, distance measuring computer, TV monitor, and a switch box to allow the operator to switch from camera to camera. Each digitizer placed a header on each of the tapes that included a time and distance designator that allowed the operator the ability to view a specific section of the tape at a later time.

During the experiment, the equipment was set-up in order to photograph and measure cracking in the form of alligator, longitudinal, and transverse cracking. Manual evaluation completed with the Dictaphone increases the number of types of distresses viewed. The additional distresses include flushing, patching, raveling, and block cracking. However, the van was not equipped to measure rut depth or the longitudinal or transverse profile (11).

For their experiment, the Iowa DOT tested nine different test sections in and around the Ames area. The experiment couldn't be carried out as planned, due to the lack of rut measuring equipment, however the distress data from the two systems was compared in an empirical manner. Both systems operate on slightly different measures. The PAVEDEX system does not provide the number of cracks, but does provide more details about the different cracks. The results of the two systems over the nine test sections varied greatly.

The results had a wide variety of variations. On one of the test sections there was less than a one- percent difference between the amount of cracking measured by both systems. However, on another test section the PAVEDEX system measured some 100 feet of transverse and longitudinal cracks while the Iowa DOT system didn't find any cracks in the same section. Almost all of the discrepancies in the nine test sections could be accounted for with engineering analysis. In some cases it was just a matter of the definition of what a crack was based on its width and direction. It was established that the PAVEDEX system worked fine with rigid pavements, however, it did exhibit some problems with measuring certain distresses associated with flexible asphalt pavements.

The results of the experiment showed that the system was easy to operate, easy to make “road-ready” at a job site, and maintenance free. It was an acceptable alternative to surveying pavement surface conditions. The information collected during field surveys was adequate for use in planning, design, and maintenance of the roadway system.

Because the collection process was accomplished as a service to the Iowa DOT the cost per lane mile was more costly. The Iowa DOT estimated a cost of $10-15 per lane mile while the prices for the PAVEDEX method ranged from $19-60 per lane mile depending on whether the survey was being conducted on a state highway, Interstate lane, county lane, or a city lane. The PAVEDEX costs include the measurement of alligator, longitudinal, transverse, block cracks, patching, corner cracks, and slab cracks. The costs did not include the measurement of other noted distresses. It was noted that if the DOT owned the equipment rather than use it as a pay service, the cost per lane mile would be drastically reduced over time (11).

It was mentioned that several modifications could be made to the PAVEDEX system that would make it more adequate for use by the Iowa DOT. Some of the modifications included the ability to analyze longitudinal and transverse profile, computer analysis of the distress types, and the ability for the user to view individual sections of the surveyed section. A later system tested by the Iowa DOT does include many of the modifications suggested for the PAVEDEX system.

ROADWARE – AUTOMATIC ROAD ANALYZER

In 1994 the Iowa DOT began their largest experiment towards improving their statewide Pavement Management System. With the help of the Center for Transportation Research and Education (CTRE) at Iowa State University they began to develop a central database that uses automated distress data collected statewide to enhance the efficiency of their system.

The automated distress data is collected using an Automatic Road ANalyzer (ARAN). This platform is actually a van instrumented to collect the various distresses at highway speeds (See Figure 8).

Figure 8: ARAN Data Collection Vehicle

In order to collect very accurate data for the Iowa Pavement Management Program, the ARAN uses differential global positioning systems (DGPS) to locate each pavement distress section. Several different types of sensors and video cameras are mounted on the ARAN vehicle in order to measure the amount of distresses and rut depths present in the pavement sections (See Figure 9, following page). Lasers are used to measure the roughness of the ride. Another set of sensors measure the depth of pavement ruts and video images are used to record the various types of cracking. The different types of distresses collected by ARAN include:

- A ride quality measure referred to as the international roughness index (IRI) for both wheel paths measured in mm of roughness per meter of pavement
- Pavement rutting in both wheel paths measured in millimeters
- D-cracking measured by the number of joints with durability cracking
- Joint spalling measured as the number of spalled joints
- Transverse cracking measured as the length of the transverse cracks in meters
- Patching measured in square meters and number of patches
- Longitudinal cracking measured over the width of the roadway in length of longitudinal cracks in meters
- Block cracking measured as the area of block cracking in square meters
- Alligator cracking measured as the area of alligator cracking in square meters
- Potholes measured as the number of potholes (10).

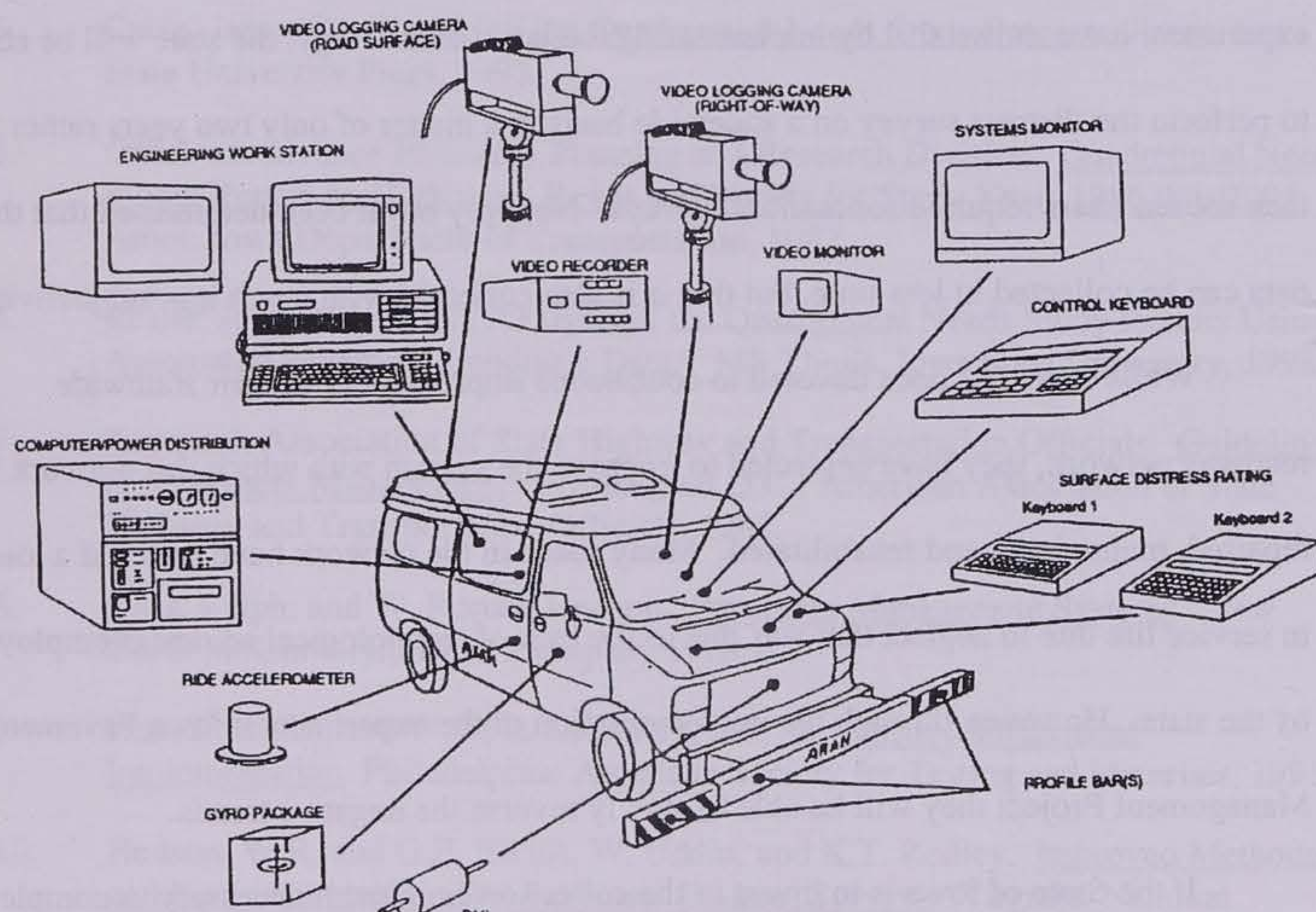

Figure 9: The components of the ARAN Collection System

The ARAN System integrates all of the latest technologies available. The system is fully automated and allows for all of the distress survey data to be compiled and stored in a computer database. Through the use of a Geographic Information System (GIS) database the data can be imported into a central processing software where it is associated with the correct pavement section through the use of the DPGS. By interfacing a GIS

System a user is able to look at a physical map of a roadway network and select certain trigger criteria for highway needs and the software will highlight the individual sections that activate these trigger levels.

CONCLUSIONS

This experimentation has been in progress for over five years due to the success with which it has been executed. Unfortunately, the state has yet to use the information collected for anything more than experimental procedures. The conclusions of this experiment have shown that by implementing the latest technology, the state will be able to perform the distress survey on a statewide basis in a matter of only two years rather than the ten years required for manual surveys. Not only has it been determined that the data can be collected in less time, but that it is also collected with much less subjectivity.

While Iowa has been devoted to continuous improvement of their statewide roadway network, they have neglected to improve the system with which this network is repaired, maintained, and rehabilitated. Many roads in the network have suffered a loss in service life due to neglect that was due to the lack of technological advances employed by the state. However, through the implementation of the experimental Iowa Pavement Management Project they will be able to slowly reverse the negative trends.

If the State of Iowa is to invest in the collection equipment necessary to complete the survey data, they will be able to save a considerable amount of money that would be required to pay for the contracted collection surveys. In addition to monetary savings, the statewide Pavement Management System will become more consistent without the extensive funding swings that are associated with the current system.

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Structural Performance of Cold Recycled Asphalt Pavements

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ABSTRACT

The structural performance of a low-volume asphalt pavement, constructed with a fly ash stabilized reclaimed asphalt pavement (RAP) base layer, was studied on several test sections on Kansas Route 27 from 1992 thru 1996. The control sections had conventional binders (asphalt emulsion with and without polymers) for the cold-in-place recycled (CIPR) base layers. All sections were tested with the Falling Weight Deflectometer (FWD) annually, and periodic distress surveys were also done. The main distress observed on this project mainly was rutting. An analysis of the deflection test results show that the CIPR pavements with fly-ash as binder may reduce the potential of rutting compared to those with conventional, asphalt emulsion binders. The shear strains developed in the thin asphalt pavements of this project might have contributed the most to the observed pavement rutting. Pavement sections with the fly-ash as CIPR binder showed very uniform distribution of shear strains within the pavement layers, and had the smallest rut depths among all sections studied. However, the shear stresses in these sections did not show any trends to explain rutting. Two different models to predict rutting were studied in this project, and the conventional rutting model relating vertical compressive strain to the allowable number of 18-kip Equivalent Single Axle Load applications to 0.5 in. rut depth, appeared to be suitable for the thin asphalt pavements with the CIPR layer.

Keywords: Low-volume Raods, Cold-In-Place Recycling, Asphalt Pavement, Fly ash, Emulsion, FWD.

INTRODUCTION

The Kansas Department of Transportation (KDOT) has been using an innovative, cost-effective method of rehabilitating low-volume asphalt pavements by cold-in-place recycling (CIPR) since 1990 (1). The process involves milling existing pavements, stabilizing the reclaimed asphalt pavement (RAP) materials with ASTM Class C fly ash, and compacting the stabilized material to the required depth, usually 4 to 6 inches. The recycled pavements are then typically overlaid with 1.5 to 2.5 inches of hot mix asphalt.

In this paper, the structural performance of a thin asphalt pavement with the RAP-fly ash layer was studied on several test sections on Kansas Route 27 against conventional binders (emulsion) for cold recycling from 1992 thru 1996. In total, eleven test sections were built, three with a cationic, medium-setting, polymerized asphalt emulsion (CMS-150P), five with a cationic, medium-setting asphalt emulsion (CMS-1) and three with 13% ASTM Class C fly ash as CIPR binder. CMS-150P is a polymer-modified asphalt emulsion manufactured by Elf asphalt. The residue from the distillation of this emulsion at 350° F had a penetration in between 150 and 300 at 77° F. All sections had the same CIPR layer thickness (4 in.). Deflection data was collected with a Dynatest 8000 FWD from 1992 to 1996. All sections were tested with FWD annually, and periodic distress surveys were also done. The objective of this paper is to discuss the results of the structural analysis of the test sections on these pavements, and to correlate the mechanistic responses within the pavement structure with the distresses observed (rutting).

PROJECT DESCRIPTION, TEST SECTION LAYOUT, AND DATA COLLECTION

The original project was located between MP 140.407 and MP 169.776 on Kansas route 27 in Sherman and Wallace counties. The pavement is a full-depth section with asphalt thickness of 7.0 in. between M.P. 140.407 to M.P. 159.976 and 10.0 in. between M.P. 159.976 and M.P. 169.776. The former section has 4.0-inch lime-treated subgrade under it. The project had a 1992 Annual Average Daily Traffic (AADT) varying from 243 to 405 on different sections. The daily 18-kip equivalent single axle (ESAL) values varied from 62 to 81. In 1996, the AADT was found to be increased between 350 and 473, but the daily ESAL values remained essentially the same on almost all sections.

The 30-year maximum average daily temperature was 70.4° F and the minimum average daily temperature was 37.5° F. The maximum extreme temperature was 108° F and the minimum observed was -15° F. The site receives an average annual precipitation of 18 in. and the depth of frost penetration is estimated to be 35 in. The soils at the project site varied from AASHTO A-4 to A-6, requiring lime treatment for the subgrade soil on part of the project during original construction.

Eleven pavement sections of varying length were constructed using 4-in CIPR base with different binders as shown in Figure 1. A 1.5-inch hot-mix asphalt (HMA) overlay was placed on the CIPR base. On each test section, a 200-ft test section was established for future deflection tests. Test sections 1, 2 and 3 are the sections with CMS-150P as CIPR binder, while test sections 4, 5, 6, 6A and 7 are with CMS-1, and sections 8, 9, and 10 are built with 13% ASTM Class C fly-ash. No construction-related problems were evident during surface condition surveys immediately after construction.

Falling weight Deflectometer (FWD) tests were done on each section before CIPR, on CIPR and after 1.5 in. hot mix overlay in 1992. Tests were done at 5 different locations at about 40 ft intervals. Deflection tests were repeated annually approximately at the same locations on each test section till 1996 for most of the sections. The target load during testing was 9,000 lbs and seven sensors, placed at uniform intervals of 12 in, were used. Pavement distress survey was repeated in 1996 when all test sections were in service for four years.

PROJECT PERFORMANCE

The 1996 distress survey showed that Sections 1, 2 and 3, which were built using CMS-150P as CIPR binder, had been overlaid by HMA in 1994 and 1996, respectively. Sections 4,5,6 and 7, with CMS-1 as CIPR binder, also were not observed to be performing well. Section 4 received a maintenance fog seal in 1996. No cracking was observed, but the section had 1/4" to 3/8" rutting on the wheel paths. Section 5 did not have any cracks, but had a 50-ft long maintenance patch in the northbound lane. Rut depth on this section ranged from 1/8" to 1/4". Section 6 did not have any cracks, but had 3/8" to 1/2" wheel path rutting. Section 7 has a few cracks with slightly more than 1/4" rutting. Sections 8, 9 and 10, built using 13% fly-ash as CIPR binder, performed much better. Section 8 had a few more cracks and a 60-ft long, half-lane wide crack. The numbers of cracks on Section 9 and 10 were similar. Although there were some more cracks at the early ages of these sections, partly because the pavement system would become somewhat rigid after adding fly-ash as a CIPR binder, these cracks did not grow over the performance period covered during this investigation. Sections 9 and 10 showed less than 1/8" rutting, and on section 8, the rut depth was 1/4".

From the results of the distress surveys on these sections, it is obvious that the use of fly-ash as a CIPR binder may reduce the rutting problem on the CIPR pavements. There appears to be two reasons for this. First, the use of fly ash increases the rigidness of RAP layer, and the stabilized RAP layer behaves somewhat like a rigid plate, thus reducing the rutting potential in that layer. The other reason is that the whole stress-strain state within the pavement structure changes as the rigidness in the RAP layer increases. This will be discussed in details later. Another study showed that fly-ash as CIPR binder reduces the moisture susceptibility of CIPR pavement (2). Therefore it may retain its properties unchanged due to climatic/environmental changes.

ANALYSIS OF FWD DATA

Significance of FWD data

The average, normalized and temperature-corrected FWD first (d_0) and seventh sensor (d_7) deflection values on each section were studied in details. Figures 2 shows these parameters on all eleven test sections in 1992. Generally, the pavement sections with fly-ash as a CIPR binder had the lowest d_0s among the three types of pavements. Figure 2 shows that the values of d_7 were in a narrow range of about 2 to 3 mils, indicating no large difference in subgrade strength among the test sections.

Correlation Between d_0 and Rut Depth

Figure 3 shows the plots of the temperature-corrected d_0 at 70° F, measured on different CIPR sections in 1992, and the rut depths measured on different sections in 1996. It appears that the rut depth measured in 1996 on each section showed the same trend as the corrected d_0 measured in 1992. Rut depth and surface deflections were found to have an obvious linear relationship in this project, and the regression equation can be expressed as:

Rut Depth (in.) = 0.0848 + 0.00467*d_0 (mils) (R^2 = 0.604)

Certainly, there are a number of other variables such as, tire pressure, load repetitions, pavement surface temperature, shear stresses or strains in pavement layers, vertical strains, which would influence the wheel path rutting. However, higher d_0 values compared to d_0s on similar pavement sections appears to indicate the propensity for rutting.

Backcalculation of Layer Moduli

A linear-elastic analysis backcalculation program, EVERCALC developed by the Washington Department of Transportation, was used to backcalculate the layer moduli of each section. All pavement sections were modeled as four-layer elastic layer systems - new HMA overlay, CIPR layer, existing HMA pavement, and infinite subgrade. It is to be noted that since all test sections in this project had the same thickness of HMA overlay, a modulus value of 435 ksi at 70°F was assumed for all test sections during backcalculation. The Root-Mean-Square (RMS) error was restricted to be less than 1%, otherwise the results were not included in further analysis. Table 1 shows the backcalculated moduli results on each section.

The CIPR layer with the CMS-1 binder had a range of moduli of 29.5 ksi to 161.4 ksi,, while the CIPR layer with the fly ash binder had a range of moduli of 317.7 ksi to 634.3 ksi. It is apparent that the moduli of CIPR with the CMS-1 binder changed with temperature as indicated by the higher range in moduli than the fly-ash CIPR layer moduli. In general, the CIPR layer with the fly ash binder had higher moduli values than the CIPR layer with the CMS-1 binder.

The moduli of existing HMA pavements were in the range of 60 ksi to 741 ksi over the years. The lower moduli values indicate the damage of this layer due to fatigue cracks, etc. In general, the existing HMA layers showed higher moduli on the sections with the CIPR layers incorporating CMS-1 binders. The moduli values were consistent over the years for most of the sections.

The subgrade moduli varied from 9 ksi to 17 ksi across the test sections. Test sections 1 thru 7 had modified subgrade. The subgrade moduli of these sections were slightly higher than the sections with compacted subgrade. The subgrade modification did not appear to affect the backcalculated subgrade moduli significantly.

ANALYSIS OF MECHANISTIC PAVEMENT RESPONSES

Determination of the state of stresses and strains within a pavement section under a load is an essential step in any pavement performance prediction methodology. Shear stresses, strains, tensile strains and vertical compressive strains were computed in each pavement test section using the backcalculated layer moduli. Each pavement section was modeled as a 4-layer system as was done during backcalculation, and the stresses and strains were computed by the layered elastic program, ELSYM5, corresponding to a single axle, dual wheel load of 18 kip with 100 psi tire pressure.

Shear Stresses and Shear Strains

Shear Stresses may cause two types of deformation in pavement structure. The first one is the volume change, and the other is the shape change, or lateral movement. The lateral movement happens mainly due to the permanent shear strain, which is part of the total shear strain in the pavement structure. Permanent shear strain, whose magnitude is controlled by the properties of pavement materials and loads, should be measured from the shear tests. However, the total shear strains could be determined by analyzing a pavement structure using a layered elastic theory. Table 2 shows the variation of shear stresses and strains over the years in the pavement cross-sections on test sections 3, 7 and 10. Figure 4 illustrates the shear stresses (averaged over the years) on these sections at three different locations within the pavement section. The average shear stresses in the middle of the AC overlay were much higher than those in other layers. On the other hand, the average shear strains, shown in Figure 5, showed a very different distribution than the shear stresses. The highest shear strain values were always in the CIPR layer. Also, the CIPR layers with different binders had different values of shear strains - the highest shear strain value of about 750 microns was observed on section 7 with the CMS-1 binder. The smallest value was found for the section with the fly-ash binder (Section 10). The values of shear strains in other layers of the pavement structure were very small compared to those in the CIPR layer. Only exception was section 10. The shear strain value at the bottom of the HMA overlay was 181 microns which was very close to the shear strain value in the CIPR layer of about 202 microns. This explains the fact that the CIPR layer with the fly-ash as binder has much better load distribution ability than those with conventional binders (CMS-1and CMS-150P). Since rutting is caused partly by the lateral movement to which the shear strains contribute the most part, the higher values of shear strains in the CIPR layer support the fact that a large part of the total rut depth would result from this layer. Since all test sections in this project just had 1.5-in. HMA overlay, this layer could not have contributed much to the total rut depth. It should be noted here that the pavement structure in this project was thin - 1.5-in. HMA overlay on top of 4-in. CIPR. If the thickness of HMA is high, the state of shear stresses and strains in the pavement structure may not show the distribution discussed here.

Tensile Strains and Vertical Strain

Table 3 shows the tensile and vertical compressive strains on sections 3, 7 and 10. At the bottom of the HMA overlay, sections with CMS-1 and CMS-150P had much higher tensile strains than those with fly-ash. At the bottom of the CIPR layer, sections with fly-ash have higher tensile strains than others indicating that these layers appeared to be more rigid than other sections making them susceptible to fatigue damage. Generally, different sections tend to have very close values of tensile strain at the bottom of existing pavement. Thus, the sections with the CMS-1 or CMS-150P as the CIPR binder could easily develop fatigue damage in the HMA overlay layer. However, the sections with fly-ash as the CIPR binder should develop fatigue cracks in the CIPR layer.

The tensile and vertical compressive strains in sections 3, 7 and 10 tend to show similar trends as the shear stresses and strains. Figure 6 shows the tensile strain distribution in sections 3, 7 and 10. From the figure, it is obvious that there is no tensile strain at the bottom of HMA overlay in section 10 indicating that the overlay would be in compression under wheel load.

Figure 7 shows the distribution of vertical compressive strains in the pavement structure for sections 3, 7 and 10. Pavement with fly-ash as the CIPR binder showed very uniform distribution of vertical compressive strain - it increases evenly with the depth of the pavement structure.

DAMAGE ANALYSIS OF IN-SERVICE PAVEMENTS

The pavement distress survey in 1996 showed that the common distress on these test sections is not the fatigue crack, but rutting on the wheel paths. On sections 4, 5 and 6 which had a 4-in CIPR base with the CMS-1 binder, no cracks were found, and the rut depths ranged from 0.25 in. to 0.5 in. Sections 9 and 10, built with fly-ash as the CIPR binder, had very little change in cracking since construction. The cracks found after construction were primarily due to the shrinkage of fly ash. These sections had less than 0.125 in rut depths. The mechanistic response analysis in the previous section showed that the tensile strain ε_r at the bottom of the HMA overlay layers were much smaller compared to the vertical compressive strains ε_z on the top of the subgrade. In usual practice, ε_r and ε_z could be used to estimate the fatigue life and permanent deformation life of asphalt pavements, respectively, by using some transfer functions.

Table 4 shows the results of life calculations for sections 3, 7 and 10 using the transfer equations developed by the Asphalt Institute. Table 4 clearly shows that the fatigue life of each section is much longer than the permanent deformation life, which could be best used to explain why no cracks were observed on the test sections.

ANALYSIS OF RUTTING

Intolerable rutting in pavement with asphalt concrete pavements is commonly an indicator of needed rehabilitation. Rutting occurs only in flexible pavements, as indicated by the permanent deformation or channelized depression on the wheel paths. Two design approaches have been followed to control rutting: the first one to limit the vertical compressive strain on the top of subgrade, and the other to limit the total rut depth to a tolerable amount, say 0.5 in. (3). To predict rutting, two types of models were be studied: (i) Conventional and (ii) VESYS.

Conventional Model

The conventional rutting model, which uses a failure criterion based on correlations with the road tests or field performance, is much easier to apply, and has been used by the Shell Petroleum Co. and the Asphalt Institute (3). The general form of the model is:

$$N_d = f_4 (\varepsilon_z)^{-f_5} \qquad \text{(i)}$$

where, ε_z is the vertical compressive strain on the top of subgrade,

Nd is the allowable number of load repetitions which are generally not expected to have a rut depth greater than 0.5 in., and

f_4 and f_5 are constants

This method is based on the contention that, if the quality of the surface and base courses is well controlled, rutting can be reduced to a tolerable amount by limiting the vertical compressive strain on the subgrade (3). Equation (i) has also been used by other agencies like the U.K. Transport and Road Research Laboratory, Belgian Road Research Center, U.S. Army Corps of Engineers and CHEVRON. The coefficient f_4 varies significantly from agency to agency, but the exponent f_5 falls within a narrow range, almost around four. Since no values of f_4 exist for the project under study, relative damage concept was used for further analysis by assuming that the exponent f_5 is equal to four. The following equation can be derived:

$$N_{d1} = \left(\frac{\varepsilon_{z_1}}{\varepsilon_{z_2}}\right)^{-4} \times N_{d2} \qquad \text{(ii)}$$

Where, ε_{z1} and ε_{z2} are the vertical compressive strains on the top of subgrade for pavement sections 1 and 2, respectively, and N_{d1} and N_{d2} are the allowable number of load repetitions corresponding to ε_{z1} and ε_{z2}, respectively.

In this study, section 6 showed ½ inch rut depth in 1996 indicating that the permanent deformation life of this pavement had been exhausted after carrying 126,290 ESALs over the last four years. The vertical compressive strain on the top of subgrade in section 6 after 1.5" HMA was overlaid in 1992 was 957 microns. Using these information and equation (ii), the lives of the other test sections were computed. Table 5 shows the permanent deformation lives of the test sections in this study. The sections with the fly ash as the CIPR binder had the lowest amount of damage among all sections. The results also show that the damage ratios fit the measured rut depths very well. It is apparent that the rutting model using the vertical strain on the top of subgrade layer as the mechanistic response is applicable to the CIPR pavements with thin overlay.

VESYS Rut Depth Prediction Model

Recently, a new mechanistic rut depth prediction model, a form of the VESYS rutting model, has been revisited by some authors (4, 5). This model is based on the principle that rutting is simply a summation of the permanent deformation in the different layers of a flexible pavement caused by successive axle load applications since construction. The main assumption of this model is that the relationship between the plastic and elastic strains is linear for all pavement layers. The general form of this model is showed in Equation (iii). For each pavement layer, two parameters (μ_i, α_i) are required to characterize the permanent deformation behavior. The rut depth is:

$$\rho_p = \sum_{j=1}^{L} h_j \frac{\mu_j}{1-\alpha_i} [\sum_{i=1}^{k} n_i (\varepsilon_{i,j})^{\frac{1}{1-\alpha_i}}]^{1-\alpha_i} \qquad \text{(iii)}$$

Where ρ_p = cumulative permanent deformation in all layer, from all load groups (rut depth)

$\varepsilon_{i,j}$ = vertical compressive strain in the middle of layer j due to a passage of an axle of group i

h_j = the thickness of layer j, and

μ_i, α_i = constants for layer i

Ali and Tayabji (4) developed the following equation using LTPP data:

$$\rho_p = 0.00011 h_{AC} [\sum_{i=1}^{k} n_i (\varepsilon_{AC})^{\frac{1}{1.111}}]^{0.9} + 23.36 h_{Base} [\sum_{i=1}^{k} n_i (\varepsilon_{Base})^{20}]^{0.05} + 0.022 h_{Subgrade} [\sum_{i=1}^{k} n_i (\varepsilon_{Subgrade})^{2.81}]^{0.356} \quad \text{(iv)}$$

Owusu-Antwi and Titus-Glover(5) also developed an equation using LTPP data:

$$\rho_p = 0.286 AGE^{0.13} \{ h_{AC} [\sum_{i=1}^{k} n_i (\varepsilon_{AC})^{\frac{1}{1-\alpha_1}}]^{1-\alpha_1} + h_{Base} [\sum_{i=1}^{k} n_i (\varepsilon_{Base})^{\frac{1}{1-\alpha_2}}]^{1-\alpha_2} + h_{Subgrade} [\sum_{i=1}^{k} n_i (\varepsilon_{Subgrade})^{\frac{1}{1-\alpha_3}}]^{1-\alpha_3} \}^{0.765} \quad \text{(v)}$$

The advantage of above models may lie in that it is particularly suitable for investigating the sensitivity of rutting to traffic load, pavement design and material properties. The disadvantage of this model includes the sensitivity of determining the values of μ_i and α_i, which are site specific, and historical traffic as well as time series distress survey data are needed to determine those constant values. Rut depth on each test section was calculated using above models and tabulated in Table 6. The table also shows the measured values. A comparison of this values led to the conclusion that the models did not yield good predicted results, partly due to the fact that (i) the assumptions of those models may not suitable for all special cases- the relationship between plastic and elastic strains is linear, (ii) regressive values of μ_i and α_i are site specific, and (iii) the models just consider the vertical compressive strains contributing to rutting, not considering the shear failure in the layers. Table 6 shows that all measured and predicted rut depths appear to have a ratio relationship, and the predicted values are more conservative than the measured values.

CONCLUSION

1. Cold-in-Place Recycled (CIPR) pavements with fly-ash as a binder reduce the potential of rutting when compared to other pavements built with the conventional binders (e.g. CMS-1 and CMS-150P).
2. Pavements with the fly-ash binder in the CIPR consistently showed the smallest first -sensor surface deflection values during Falling Weight Deflectometer testing implying the highest global strength for this type of pavements. Surface deflection values had a strong relationship with the rut depths measured on the pavement sections of this project.
3. The shear strains in the thin asphalt pavements of this project appeared to contribute much to the development of rutting, especially shear strains in the CIPR layer. Pavement sections with fly-ash as the CIPR binder showed a very uniform distribution of shear strains across the pavement layers and had the smallest rut depths among all pavement sections. On the other hand, shear stresses in those pavement sections did not show any specific trends to explain rutting.
4. Pavement damage in this project happened mostly due to rutting, not fatigue cracking. From the analysis of the state of stresses and strains in the pavement

sections, it was found that these pavements were susceptible to rutting failure, not fatigue failure. Rutting in the thin asphalt pavements with the CIPR base layer appeared to be affected to some extent by the rigidness of that layer.

5. Two different rutting prediction models were studied, and the conventional rutting model relating vertical compressive strain on the top of subgrade to the allowable number of 18-kip ESAL applications to 0.5 in. rut depth, appeared to be to be suitable for the thin asphalt pavements with the CIPR layer.

ACKNOWLEDGMENTS

The author wish to acknowledge the sincere helps by Dr. Mustaque Hossain from the Department of Civil Engineering at Kansas State University and Mr. Andrew Gisi from the Kansas Department of Transportation.

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TABLE 1 Results of Backcalculated Layer Moduli (ksi)

Pavement Structure		Test Section Number									
		1	2	3	4	5	6A	7	8	9	10
4"CIPR layer	1992	91.0	77.0	83.0	95.0	125.0	53.3	82.3	273.0	403.0	356.0
	1993	146.8	60.5	67.6	114.0	58.0	58.0	68.0	408.0	500.0	475.8
	1994	91.2	84.3	70.0	66.0	76.3	53.6	87.2	361.0	634.3	524.2
	1995	N/A	N/A	112.0	34.0	29.5	47.8	34.0	478.3	586.0	340.0
	1996	N/A	N/A	161.4	89.8	56.5	59.6	66.0	404.0	398.5	317.7
Existing AC layer	1992	134.3	77.0	324.8	75.0	132.3	68.7	184.3	60.0	124.0	111.6
	1993	113.0	87.5	361.0	302.0	208.5	71.0	266.8	62.0	62.0	81.6
	1994	132.0	394.3	308.8	50.0	209.3	178.2	228.0	64.5	108.3	119.2
	1995	N/A	N/A	322.0	160.5	415.0	290.8	741.0	60.0	96.0	75.0
	1996	N/A	N/A	354.4	210.3	218.8	70.6	602.0	61.0	69.0	93.0
Subgrade layer	1992	10.0	9.5	11.0	15.0	13.6	9.0	9.0	12.0	15.0	13.0
	1993	9.0	9.3	11.0	17.0	12.5	9.0	9.0	12.0	14.0	12.4
	1994	9.6	9.5	11.0	18.0	12.5	11.4	11.0	11.5	15.0	13.6
	1995	N/A	N/A	9.6	16.5	11.0	9.0	9.0	11.3	13.0	13.0
	1996	N/A	N/A	9.6	16.5	11.5	9.2	10.0	12.0	14.5	13.0

TABLE 2 Variation of Shear Stresses and Strains

		1992	1993	1994	1995	1996
Section 3	τ_{xz1}	-48.8	-52.2	-51.8	-44.1	-39.1
	τ_{xz2}	-10.3	-9.4	-9.7	-11.3	-12.4
	τ_{xz3}	-6.5	-6.8	-6.5	-6.1	-5.8
	γ_{xz1}	-116.7	-107.7	-110.4	-128.0	-140.1
	γ_{xz2}	-597.2	-627.7	-666.7	-488.6	-373.4
	γ_{xz3}	-68.5	-59.6	-69.8	-70.9	-65.8
	γ_{xz4}	-13.3	-12.1	-15.4	-6.5	-1.1
Section 7	τ_{xz1}	-49.7	-52.4	-48.4	-65.9	-52.1
	τ_{xz2}	-10.8	-9.6	-10.9	-6.7	-8.8
	τ_{xz3}	-5.0	-6.0	-5.5	-6.6	-7.5
	γ_{xz1}	-121.4	-10.9	-122.1	-78.5	-102.9
	γ_{xz2}	-627.2	-680.7	-595.7	-944.6	-657.0
	γ_{xz3}	-110.7	-77.9	-94.0	-22.9	-35.2
	γ_{xz4}	-10.3	-8.6	-13.9	-26.0	-4.7
Section 10	τ_{xz1}	-33.2	-31.3	-30.3	-34.1	-34.4
	τ_{xz2}	-17.8	-19.6	-19.3	-18.4	-17.7
	τ_{xz3}	-0.6	-0.1	-0.3	-0.3	-0.5
	γ_{xz1}	-178.2	-184.8	-182.1	-182.2	-178.5
	γ_{xz2}	-217.2	-168.8	-151.1	-232.5	-243.6
	γ_{xz3}	-86.8	-72.1	-65.2	-96.6	-97.6
	γ_{xz4}	-50.8	-62.1	-50.0	-61.5	-55.0

Note: τ_{xz} - shear stress (psi), γ_{xz} - shear strain ($\times 10^{-6}$ in./in.)
τ_{xz1} - in the middle of HMA overlay course , τ_{xz2} - in the middle of CIPR course ,
τ_{xz3} - in the middle of existing AC course, γ_{xz1} - at the bottom of HMA overlay course,
γ_{xz2} - on the top of CIPR course, γ_{xz3} - on the top of existing AC course,
γ_{xz4} - on the top of subgrade course

TABLE 3 Variation of Tensile Strains and Vertical Compressive Strains

		1992	1993	1994	1995	1996
Section 3	ε_{r1}	120.7	155.7	151.0	73.0	30.0
	ε_{r2}	26.0	10.8	22.6	45.0	53.0
	ε_{r3}	238.2	234.3	250.9	236.7	213.7
	ε_{z1}	-55.4	-59.0	-57.7	-46.2	-43.7
	ε_{z2}	-612.2	-743.6	-717.8	-456.1	-316.6
	ε_{z3}	-138.4	-125.2	-143.4	-143.6	-133.2
	ε_{z4}	-510.6	-509.4	-536.4	-508.7	-461.4
Section 7	ε_{r1}	126.3	153.4	116.5	307.6	153.6
	ε_{r2}	96.9	32.3	66.8	-41.6	-15.0
	ε_{r3}	326.2	288.2	274.9	195.1	185.8
	ε_{z1}	-44.8	-50.0	-53.5	-69.5	-60.2
	ε_{z2}	-612.5	-731.8	-584.3	-639.3	-770.7
	ε_{z3}	-221.1	-163.9	-181.0	-60.0	-78.5
	ε_{z4}	-665.9	-599.4	-566.5	-463.6	-427.4
Section 10	ε_{r1}	-14.4	-29.3	-28.3	-14.0	-8.6
	ε_{r2}	132.7	149.3	116.5	171.4	153.3
	ε_{r3}	205.6	219.0	185.3	234.4	222.8
	ε_{z1}	-50.8	-42.3	-54.2	-41.7	-47.0
	ε_{z2}	-148.6	-113.2	-99.8	-159.3	-168.3
	ε_{z3}	-183.6	-209.9	-163.2	-236.7	-210.3
	ε_{z4}	-382.4	-405.8	-355.8	-430.1	-420.1

Note: ε_r - tensile strain ($\times 10^{-6}$ in./in.), ε_z - vertical compressive strain ($\times 10^{-6}$ in./in.)
ε_{r1} - at the bottom of HMA overlay course , ε_{r2} - at the bottom of CIPR course ,
ε_{r3} - at the bottom of existing AC course , ε_{z1} - in the middle of HMA overlay course,
ε_{z2} - in the middle of CIPR course, ε_{z3} - in the middle of existing AC course,
ε_{z4} - on the top of subgrade

TABLE 4 N_f and N_d using Asphalt Institute Damage Model

Section No.	Fatigue life, N_f (ESALs)				Permanent deformation life, N_d (ESALs)	
	tensile strain @bottom of HMA overlay	N_f (millions)	tensile strain @bottom of CIPR	N_f (millions)	vertical strain on the top of Subgrade	N_d (millions)
3	120.7	2, 600	26.0	400, 000	510.6	0.75
7	126.3	2, 200	96.9	5, 300	665.9	0.23
10	-14.4	N/A	132.7	1, 900	382.4	2.7

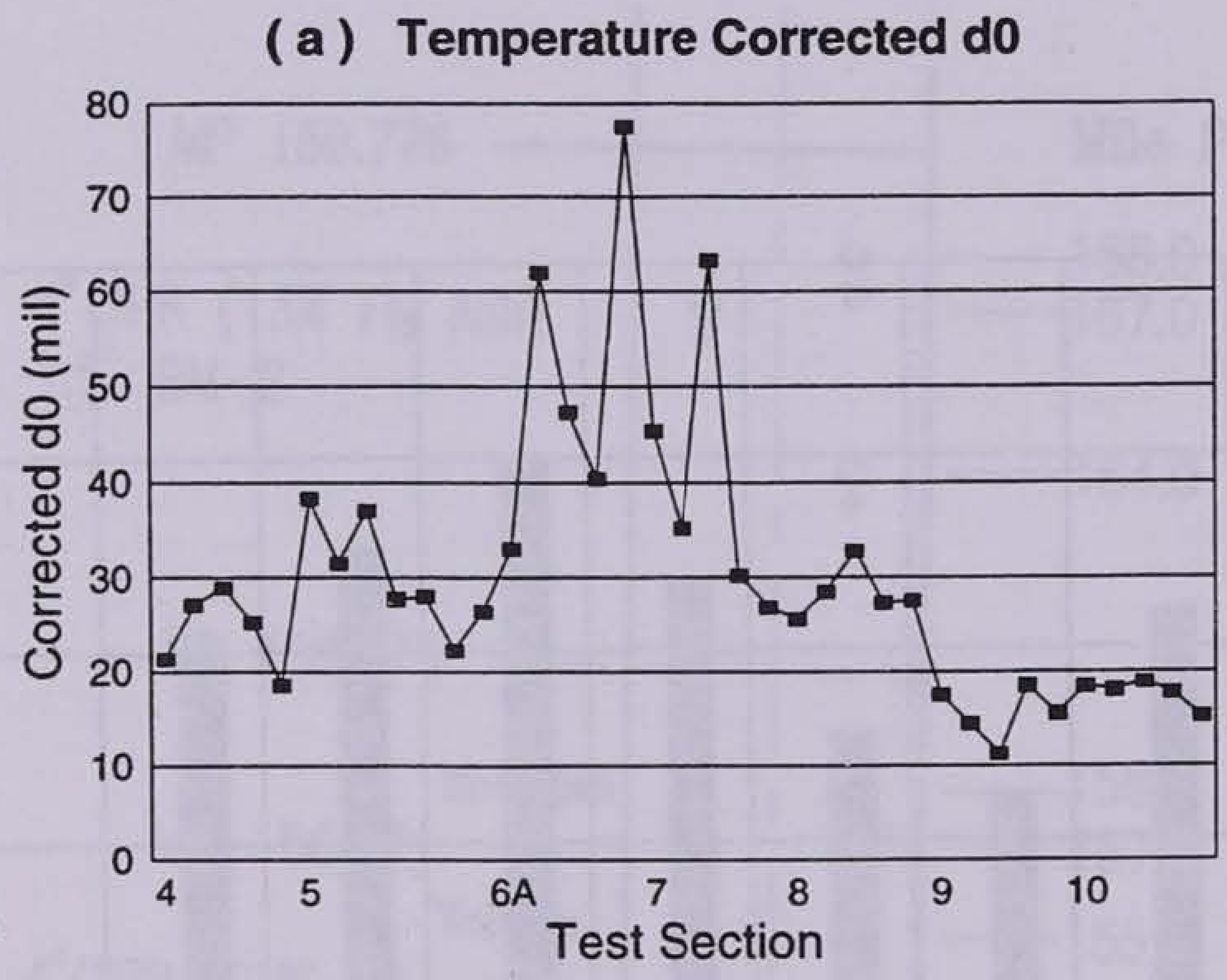
Note: $N_f = 0.0769(\varepsilon_r)^{-0.3291} E^{-0.854}$, $N_d = 1.365 (\varepsilon_z)^{-4.477}$

TABLE 5 Permanent Deformation Lives

Section No.	Actual EASLs	Vertical Compressive Strain, εz	Predicted ESALs (Nd)	Damage Ratio	Measured Rut Depth (in.)
1	124, 830	776.0	292, 126	0.43	N/A
2	131, 035	896.1	164, 356	0.80	N/A
3	128, 845	510.6	1,558, 452	0.08	N/A
4	143, 810	689.0	470, 045	0.31	0.375
5	126, 290	621.0	712, 279	0.18	0.25
6A	126, 290	957.0	126, 290	1.00	0.5
7	126, 290	665.9	538, 742	0.23	0.275
8	135, 415	489.2	1,852 , 598	0.07	0.25
9	135, 415	342.0	7,743 , 054	0.02	0.12
10	135, 415	382.4	4,953 , 877	0.03	0.12

TABLE 6 Predicted Rut Depths and Measured Values

Section	1	2	3	4	5	6A	7	8	9	10
Rut Depth using Eq.(iv) (inch)	0.128	0.154	0.122	0.149	0.155	0.20	0.14	0.093	0.069	0.077
Rut Depth using Eq.(v) (inch)	0.167	0.171	0.139	0.157	0.155	0.194	0.149	0.124	0.104	0.112
Measured rut depth (inch)	N/A	N/A	N/A	0.375	0.25	0.5	0.275	0.25	0.12	0.12



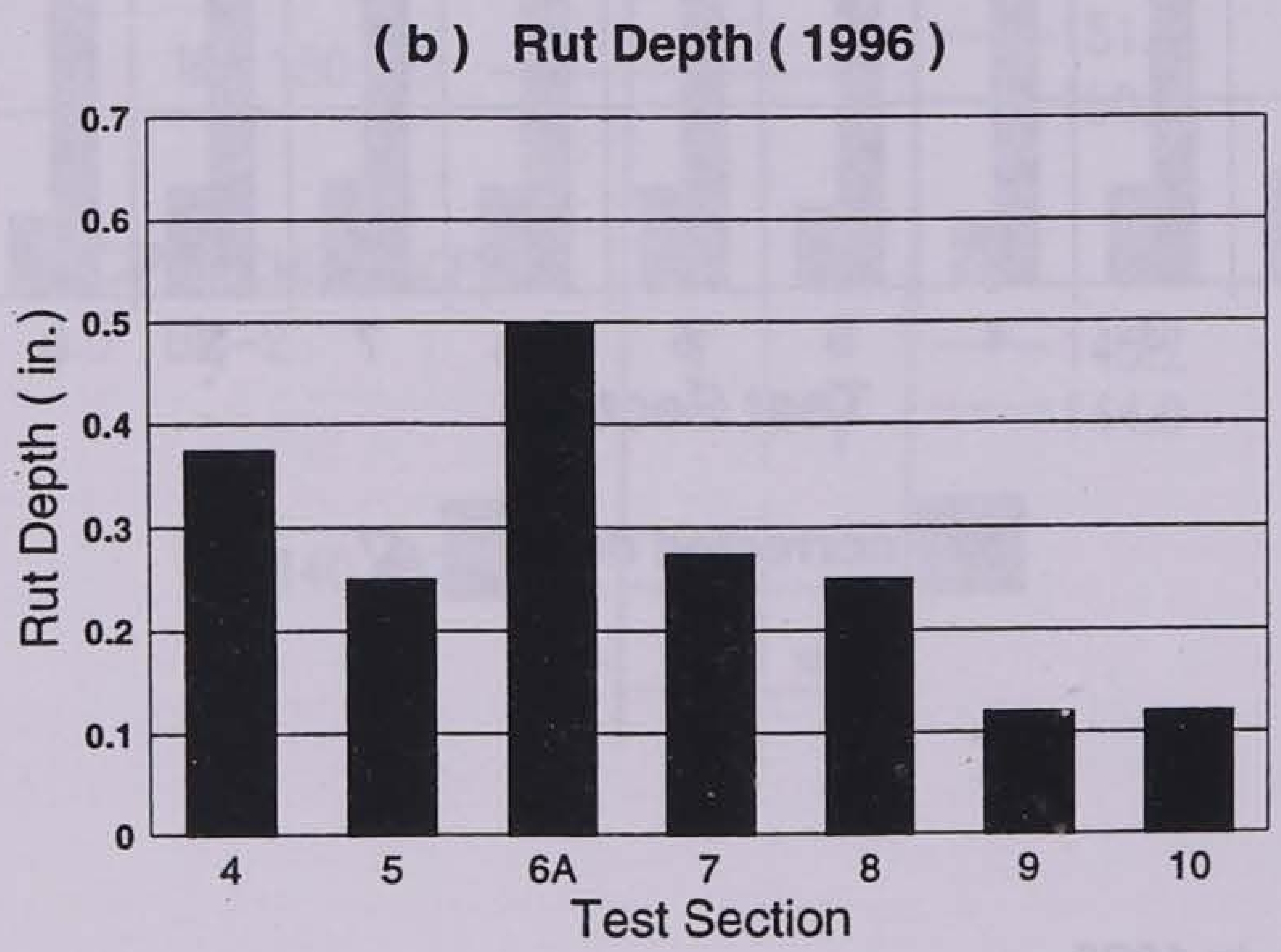


Figure 3 Rut Depth and Temperature Corrected d_o

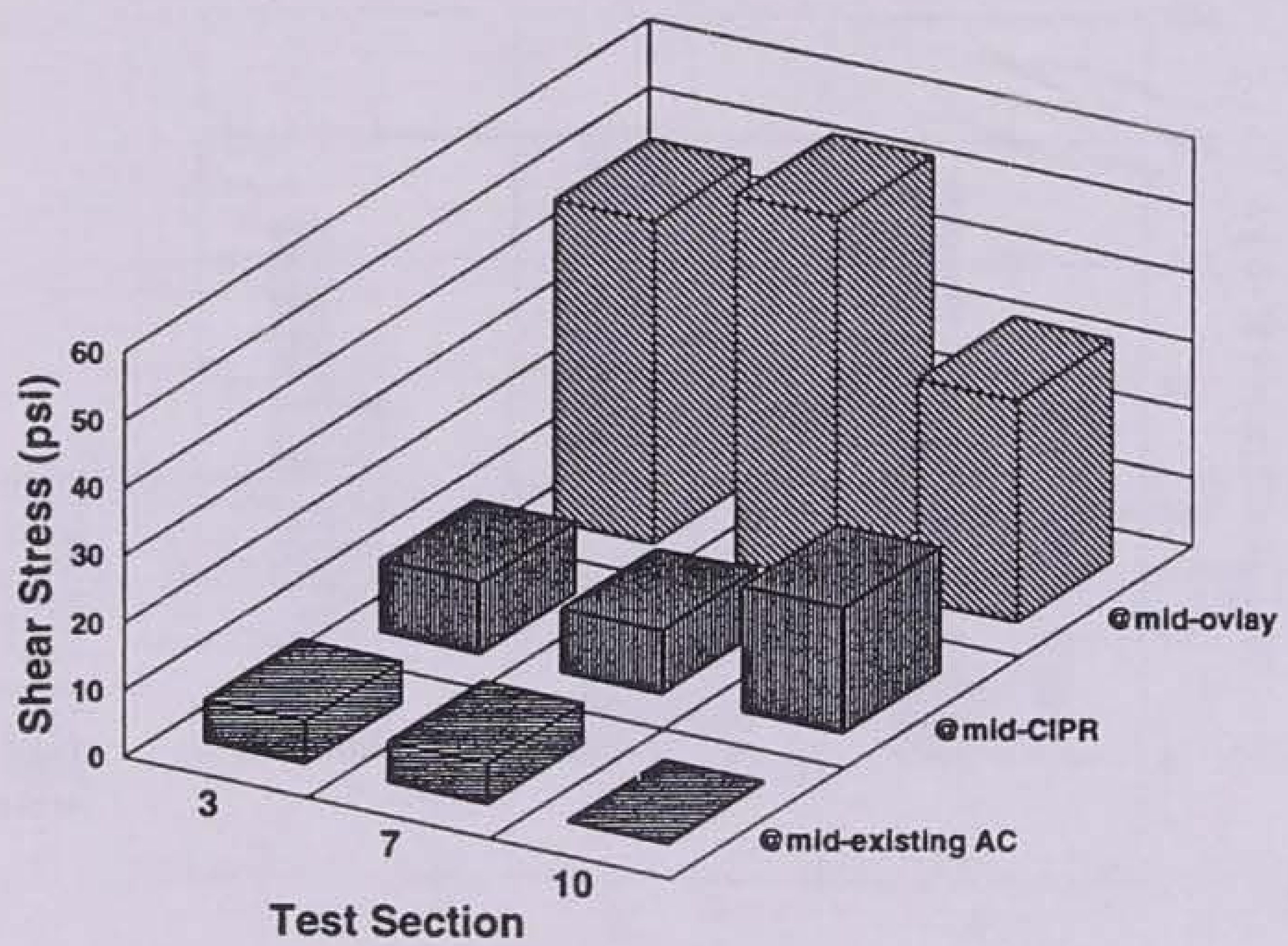


Figure 4 Distribution of Shear Stresses

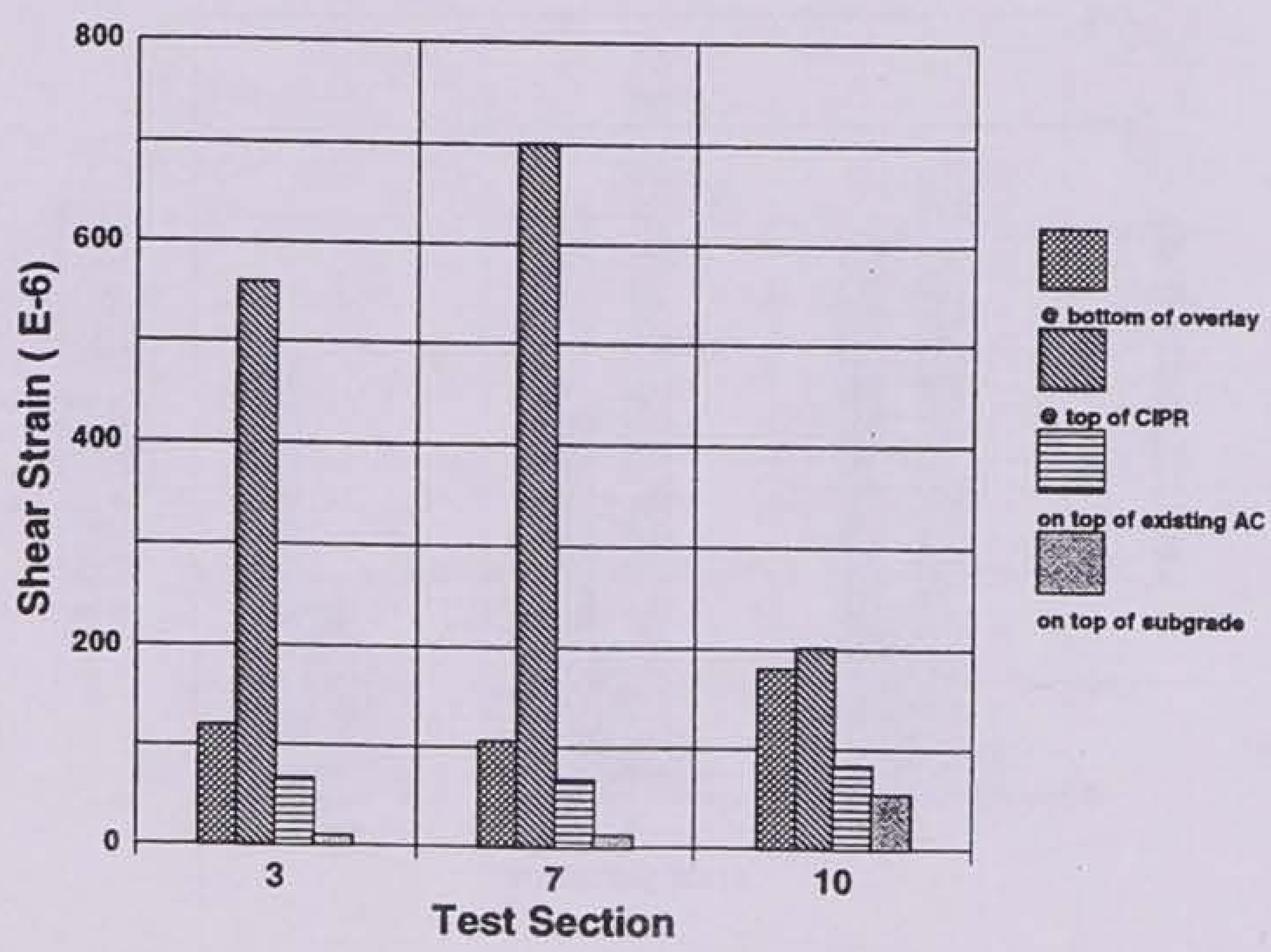


Figure 5 Distribution of Shear Strains

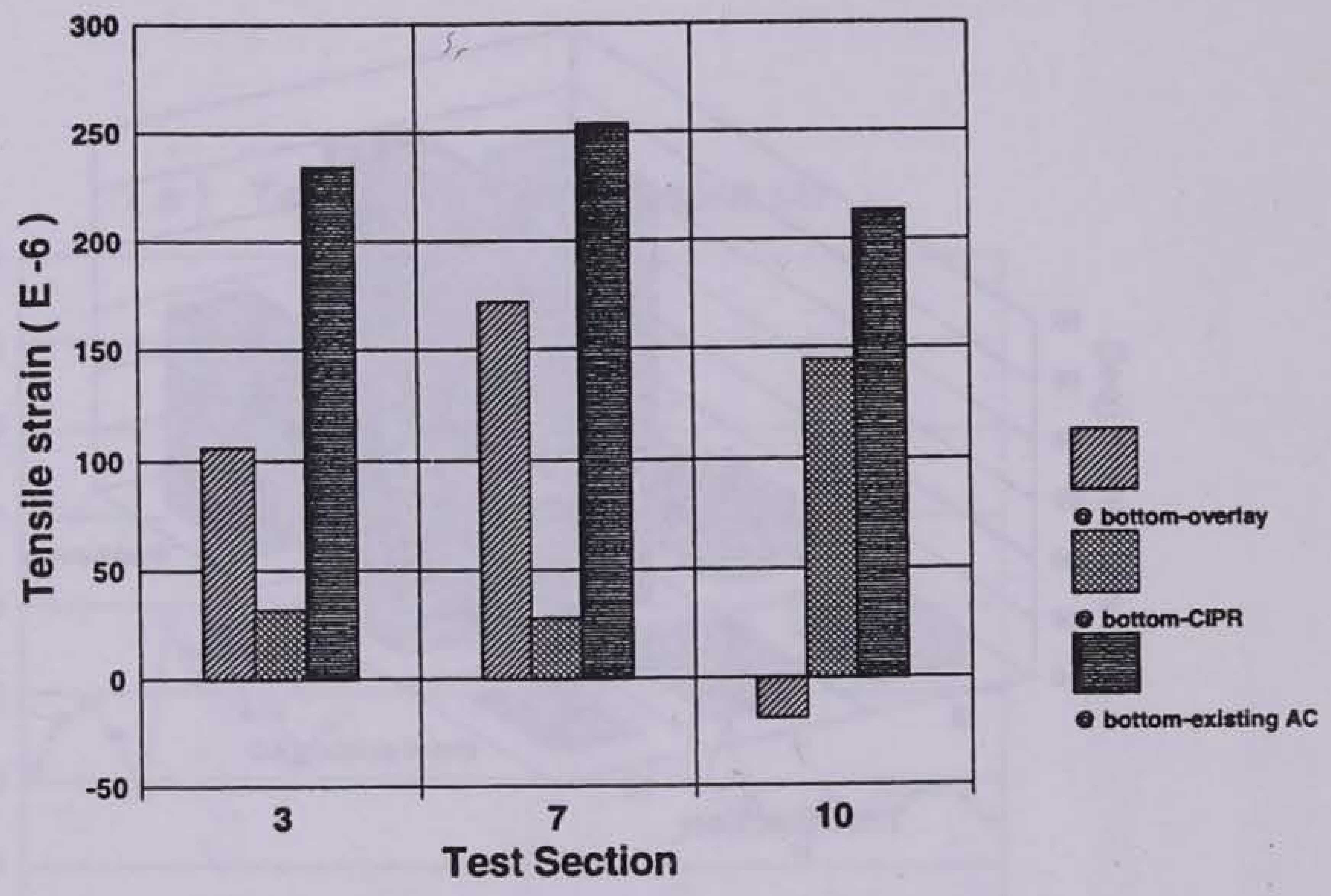


Figure 6 Distribution of Tensile Strains

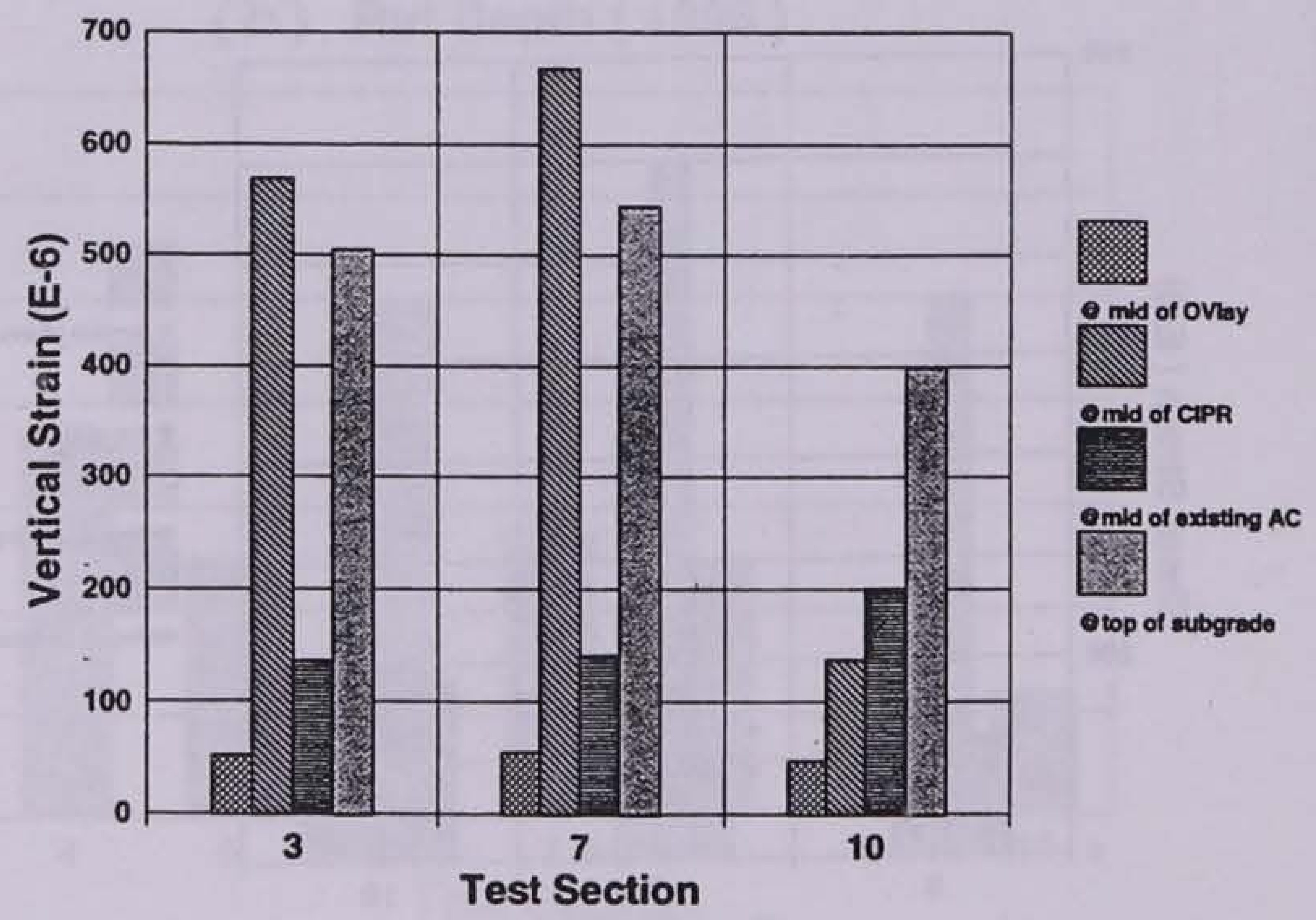


Figure 7 Distribution of Vertical Compressive Strains

Route Choice, Traffic Assignment, and Traffic Modeling for Analyzing Traffic Diversion

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ABSTRACT

In recent years, freight transportation by rail has seen a significant increase. This increase has led to greater density of freight trains on the nation's rail lines. As a result, many of the nation's at-grade crossings have seen an increase in the amount of time the crossings are blocked. In urban areas with limited access to separated grade crossings, this delay can be significant. The problem is particularly acute in smaller cities that don't have the resources to provide many separated crossings. The purpose of this study is to evaluate methods to determine and analyze traffic diversion in response to a train blocking an at-grade crossing on a roadway. This paper looks at methods to analyze traffic diversion to compare measures of effectiveness such as travel times, speeds, and delays specifically applied to the City of Ames. This paper will investigate tools to perform the three stages of traffic diversion analysis: 1) Route choice, 2) traffic assignment to these routes, and 3) modeling and network analysis. Four methods for completing this analysis are addressed in detail and summaries of the advantages and disadvantages of each are presented. Finally, the paper concludes that the INTEGRATION traffic diversion modeling package is the most appropriate tool available to analyze the city of Ames network.

1. INTRODUCTION

In recent years, freight transportation by rail has seen a significant increase. According to the Federal Rail Administration (FRA), the total freight transported by rail in 1985 was approximately 930 million ton-miles, which had grown 47.5% to 1.375 billion ton-miles by 1995. (1) This increase has led to greater density of freight trains on the nation's rail lines. As a result, many of the nation's 158,559 at-grade crossings (2) have seen an increase in the amount of time the crossings are blocked. In urban areas with limited access to separated grade crossings, this delay can be significant. The problem is particularly acute in smaller cities that don't have the resources to provide many separated crossings. The purpose of this study is to evaluate methods to determine and analyze traffic diversion in response to a train blocking an at-grade crossing on a roadway.

In the city of Ames, Iowa, population 50,000, the downtown central business district (CBD) has four at-grade crossings and only a single separated grade crossing. The traffic flow through this area is high, especially during the peak periods. The rail flow through Ames is also high as the Union Pacific (UP) Railroad's principal east west route runs through the city's downtown area. The traffic traveling north and south is greatly impeded due to the lack of separated grade crossings. The city of Ames is limited both by space and money in finding a solution that will maintain the north-south traffic flow during periods when the train is blocked. (3) It is believed that diverting traffic along carefully chosen routes and timing the traffic signals to give this traffic priority through the corridor will allow travelers to reach the one separated grade crossing in the downtown area and, thus, maintain the north-south traffic flow.

This paper will look at methods to analyze traffic diversion to compare measures of effectiveness such as travel times, speeds, and delays specifically applied to the City of Ames. The method to perform this analysis consists of three steps. The first step is to identify possible diversion routes. These routes will allow traffic to divert from the intended route, move to the separated grade crossing, and return to the intended route at a point that is past the blocked crossing. Second, the amount of traffic that will divert along those routes must be estimated. Third, the network must be analyzed using an analytical tool such as a traffic modeling or simulation tool to compare specific measures of effectiveness and determine the best plan of diversion. This paper is presented in six sections. Section 2 presents a brief overview on computer simulation and its applications. Section 3 summarizes the case study previously completed for the city of Ames, Iowa. Section 4 identifies deficiencies in the case study and additional research needed to address those deficiencies. Section 5 describes the literature review completed for this paper. Section 6 summarizes the results of the research for this paper and section 7 discusses the conclusions of this study and recommendations for further research.

2. SIMULATION BACKGROUND

Computer simulation is a powerful tool that allows experimentation and analysis on a system when actual field experiments are not feasible or practical due to money, time, or other constraints. It is used in many industries over a wide range of applications, including traffic applications. Simulation models are built to represent the operation of a system over time, either actual or planned, and allows experiments to be conducted on the system without having to implement changes in the actual system. Since the model is designed to show the operation of a system over time, it can be used to predict the performance of the system for a range of input parameters. (4)

3. BACKGROUND

The city of Ames, Iowa is located in central Iowa approximately 30 miles north of Des Moines. It occupies the northwest area of the Interstate 35 and U.S. Highway 30 intersection as shown in Figure 1. U.S. Highway 69 runs north-south through Ames as shown in Figure 2. It runs north-south along Duff Avenue, east-west along Lincoln Way and north-south again at Grand Avenue. The Union Pacific Railroad has a mainline track running through Ames, also shown in Figure 2. The rail line travels through the heart of Ames' downtown central business district (CBD). The downtown area focused on for this study is shown in Figure 3 below. As seen in the figure, there are three at-grade crossings in the CBD at Clark Street, Kellogg Avenue, and Duff Avenue. There is also one separated grade crossing located at Grand Avenue.

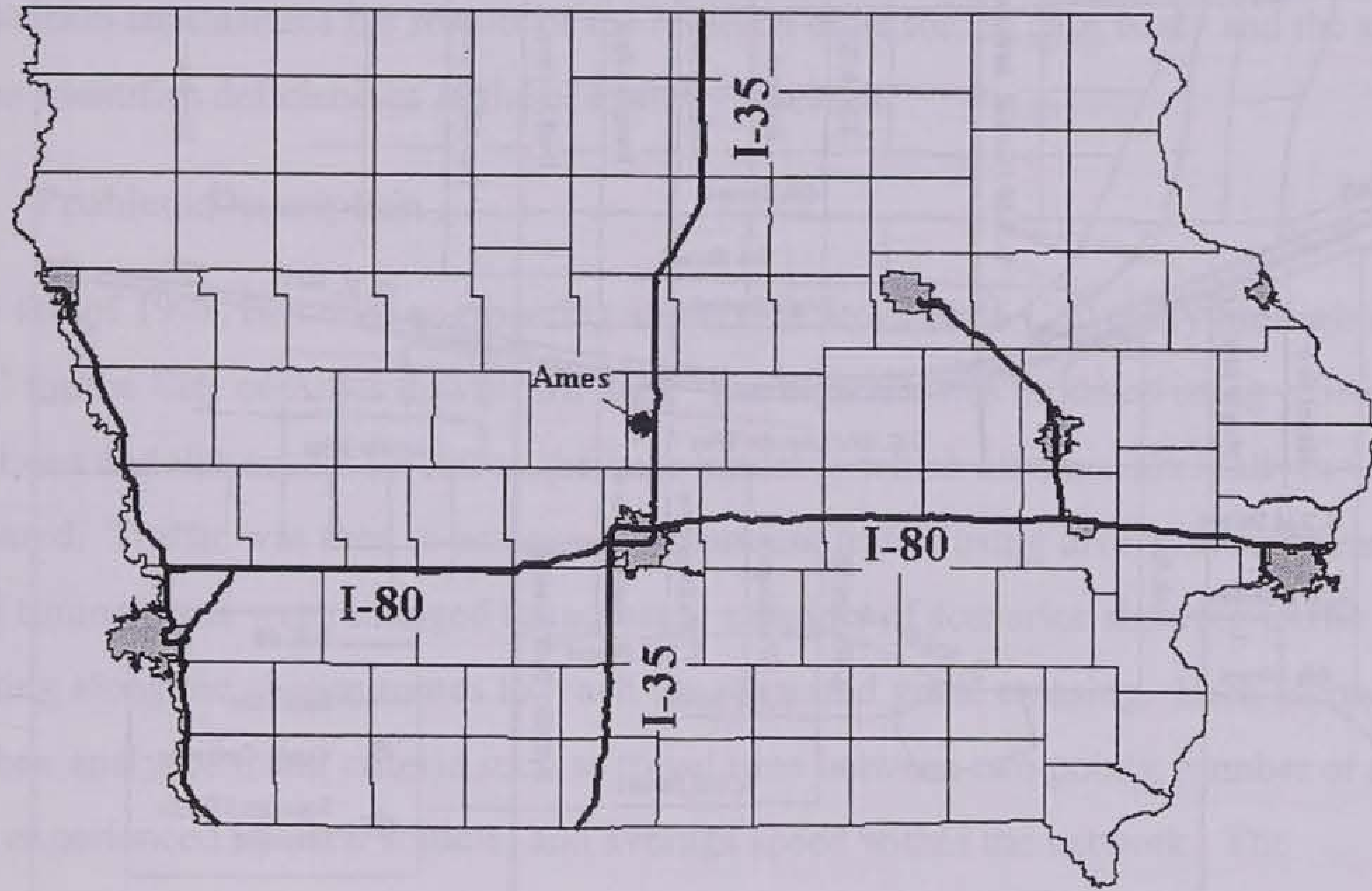

Figure 1. Location of Ames, Iowa

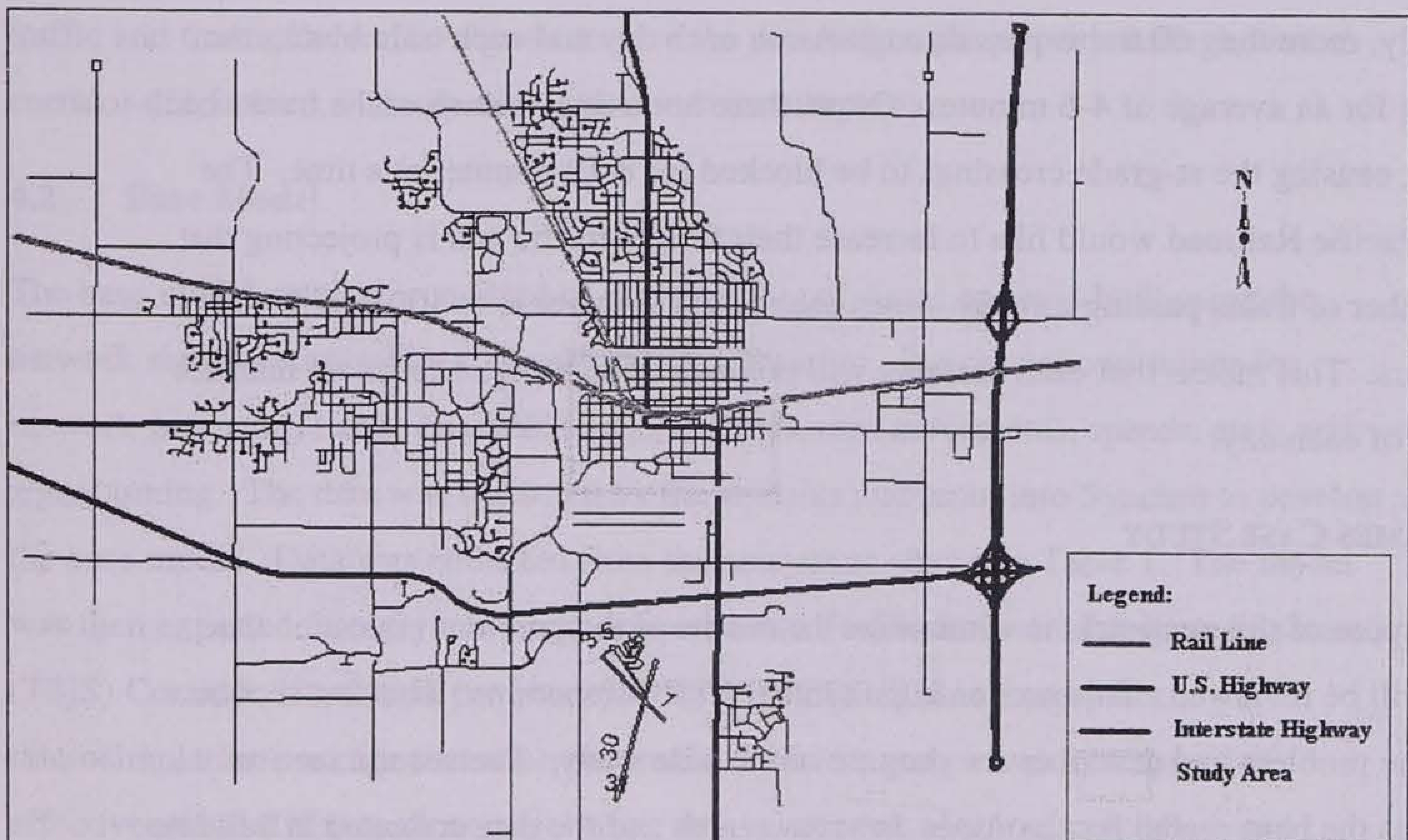

Figure 2. Ames, Iowa

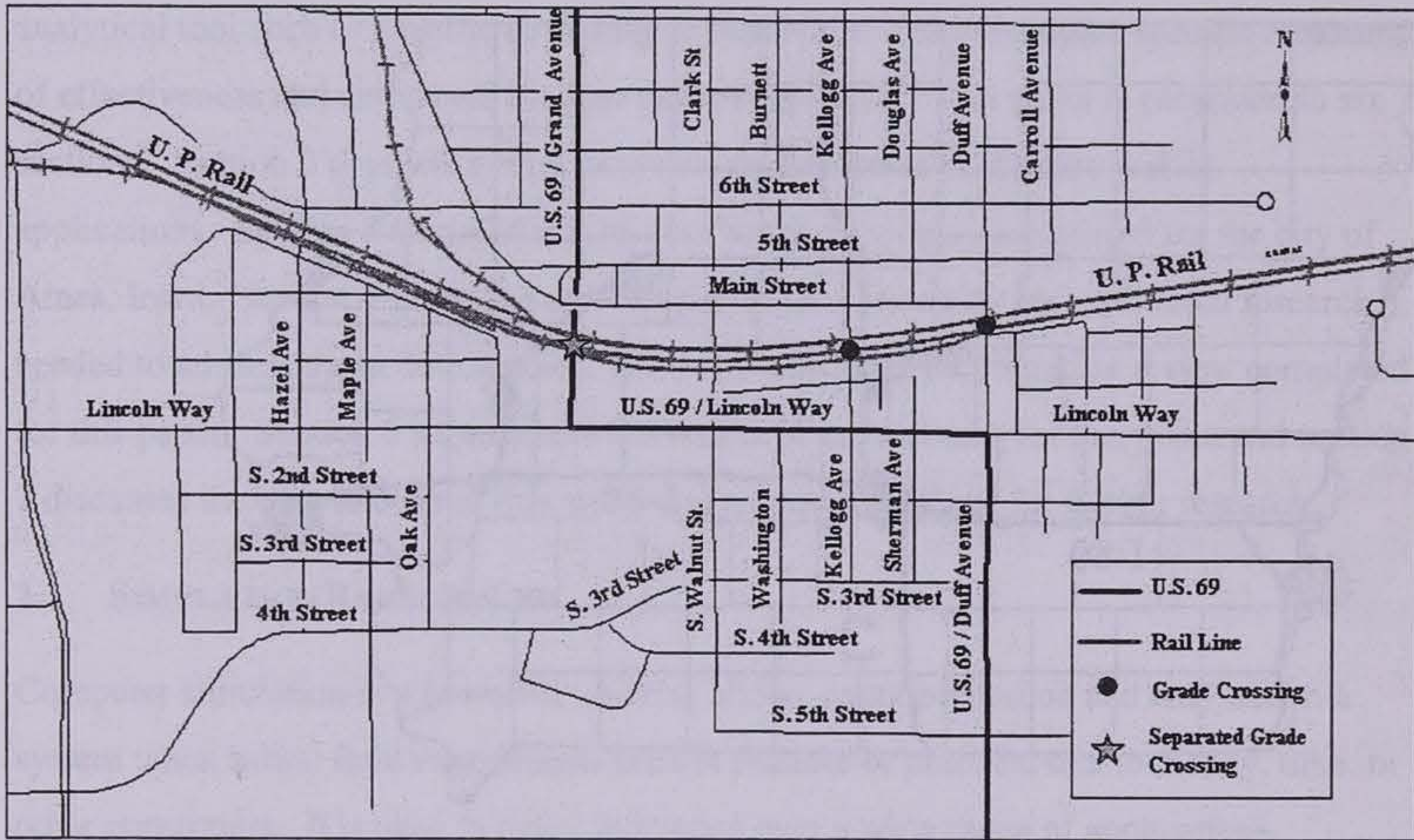

Figure 3. Study Area and Railroad Crossing Locations

Currently, more than 60 trains pass through Ames each day and each train blocks the crossing for an average of 4-6 minutes. Often, there are trains on each of the tracks back-to-back, causing the at-grade crossings to be blocked for 8-12 minutes at a time. The Union Pacific Railroad would like to increase their freight traffic and is projecting that the number of trains passing through Ames each day will increase to 100 over the next few years. This means that each crossing will potentially be blocked for more than 25 percent of each day.

4. Ames Case Study

The purpose of this section is to summarize the results of the previous research. One study will be reviewed. This section is presented in six subsections. The first section states the problem and describes the purpose of the case study. The second section describes the base model for the Ames downtown area and the data collected to build it. The third section identifies the routes chosen for traffic diversion and why they were chosen. The fourth section describes the modeling process used in the case study. The fifth section summarizes the results of the research done for the case study and the sixth section identifies deficiencies in the case study research.

4.1 Problem Description

In the fall of 1998, two civil engineering students at Iowa State University built a base model for the City of Ames downtown area. The network was modeled using existing conditions and this model served as the base model to which all other alternatives were compared. Traffic was then re-assigned to represent traffic using diversion routes and signal timing plans were changed to represent a variety of scenarios showing traffic diverting along the chosen routes to reach the separated grade crossing. Each scenario was then analyzed using criteria such as travel time between two points, number of stops, delay experienced by each vehicle, and average speed within the network. The alternative scenarios were then compared to determine whether a diversion plan with signal timing coordination would increase the north and south mobility through the downtown area while trains block the grade crossings. The results showed that diverting traffic and timing the traffic signals to give the diverted traffic priority throughout the corridor did, indeed, increase the north-south mobility. (5)

4.2 Base Model

The base model network consisted of just the CBD of Ames and was built using the network signal optimization software package Synchro. Synchro allows inputs for network geometry, traffic characteristics (volumes, turn movements, speeds, etc.), and signal timing. The data was collected by the students and input into Synchro to develop the base model. Data was collected from the sources as shown in Table 1. The model was then exported from Synchro and imported into Traffic Software Integrated System's (TSIS) Corridor Simulation program (CORSIM). CORSIM is a computer microsimulation package that analyzes traffic corridors and provides measures of effectiveness (MOEs) such as travel time, delays, number of stops, stop delay, and average speeds. These MOEs are recorded for both the link and the entire network.

The base model was run in CORSIM to provide the base MOEs for comparison with various alternatives.

Table 1. Data Sources

Item	Source
Traffic Volumes	City of Ames
Network Geometry	Field Measurements, September 1998
Intersection Control Type	Field Measurements, September 1998
Existing Signal Timing Plan	City of Ames
Mid-Block maneuvers	manual counts, September 1998

4.3 Traffic Diversion Route Descriptions

The network for the city of Ames does not provide a lot of options when trying to choose traffic diversion routes. The problem of north-south mobility not only exists while trains block the at-grade crossing, but during all other times as well. This is partly due to the poor connectivity of local streets throughout the city, especially in the downtown area. The diversion routes chosen in the previous study were chosen based on knowledge of the area and personal opinion rather than function and classification of the roads used.

Due to the existence of only one separated grade crossing and the lack of connectivity of local streets in the city of Ames, there were few paths that were candidates for diversion routes (see Figure 3). The final routes chosen for diversion were chosen based on the number of stops due to signals and stop signs necessary to traverse the path. All paths chosen have a significant number of left turns to them but, due to the congestion on Lincoln Way and the number of traffic signals, it was determined that the left turns were less impeding than the other alternate routes. The lack of routes to choose from made it difficult to choose routes that would eliminate most left turns.

Two diversion route scenarios were chosen for analysis and comparison, Alternative A and Alternative B. Each alternative has a combination of diversion paths for the northbound and the southbound traffic.

If Alternative A were used (as shown in Figure 4), the traffic would travel north along Duff Avenue and divert to the west along either South 3rd Street or South 5th Street. At Walnut Avenue, vehicles would turn north and continue on Walnut Avenue to Lincoln Way. At Lincoln Way, the vehicles would travel west to Grand Avenue and turn north.

At 6th Street, the vehicles would make a right hand turn to head East to Duff Avenue. At Duff Avenue, the traffic would be back to its original route and would continue north on Duff Avenue.

The southbound traffic in Alternative A would divert from Duff Avenue at 6th Street and continue to Grand Avenue. At grand, the vehicles would turn left and travel south to Lincoln Way where it would turn left again and head east. At Walnut Avenue, the vehicles would turn south to South 3rd Street. At South 3rd Street, the traffic would head east to Duff Avenue where it would, again, resume its original path southbound along Duff Avenue.

Alternative B is similar to Alternative A (as shown in Figure 5) except that northbound traffic diverting along South 3rd Street would not travel to Walnut Avenue. Instead, vehicles would turn north on Kellogg Avenue and continue to Lincoln Way. At Lincoln Way, the traffic would turn left to head west and continue on the same diversion path as in Alternative A.

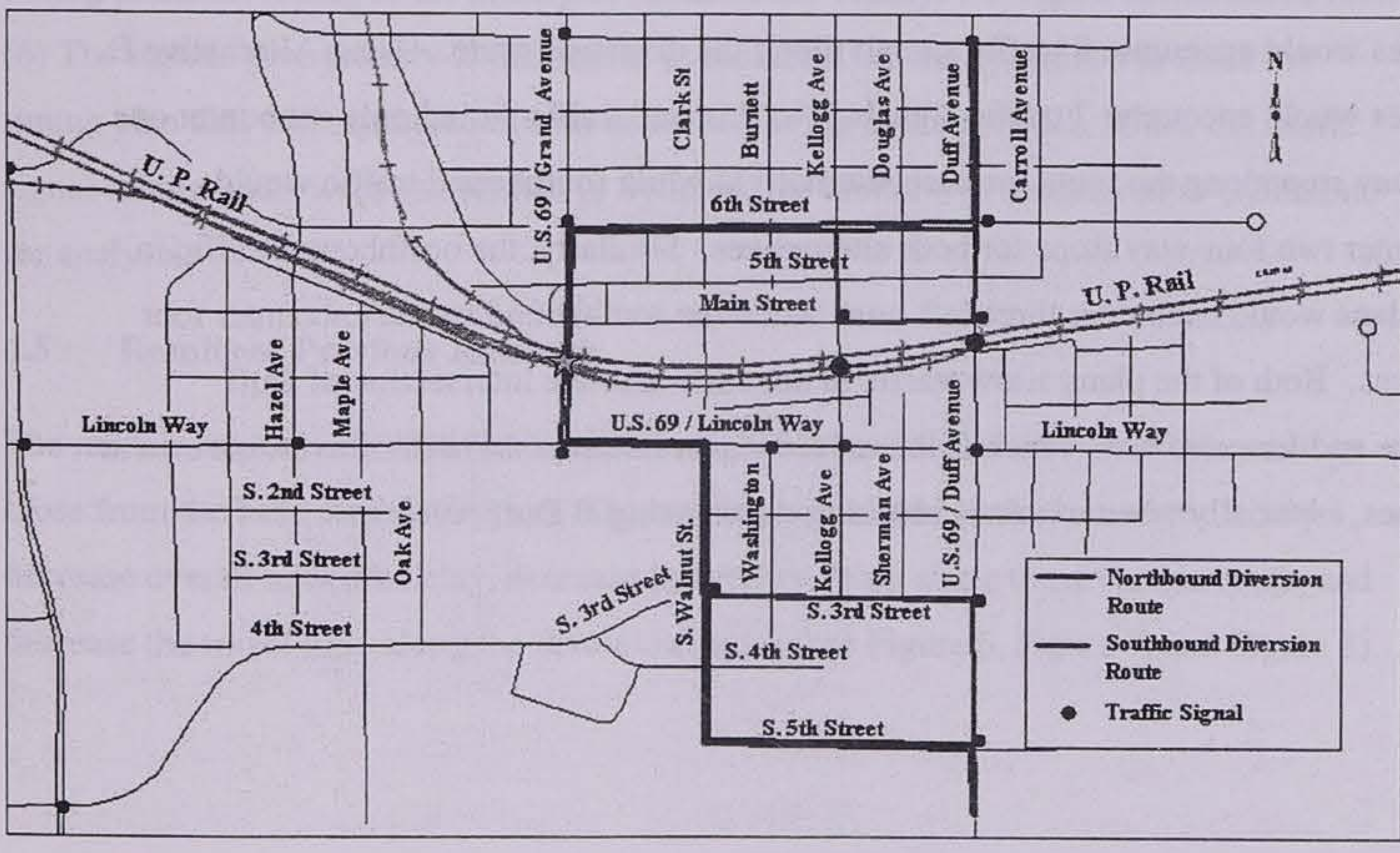

Figure 4. Alternative A

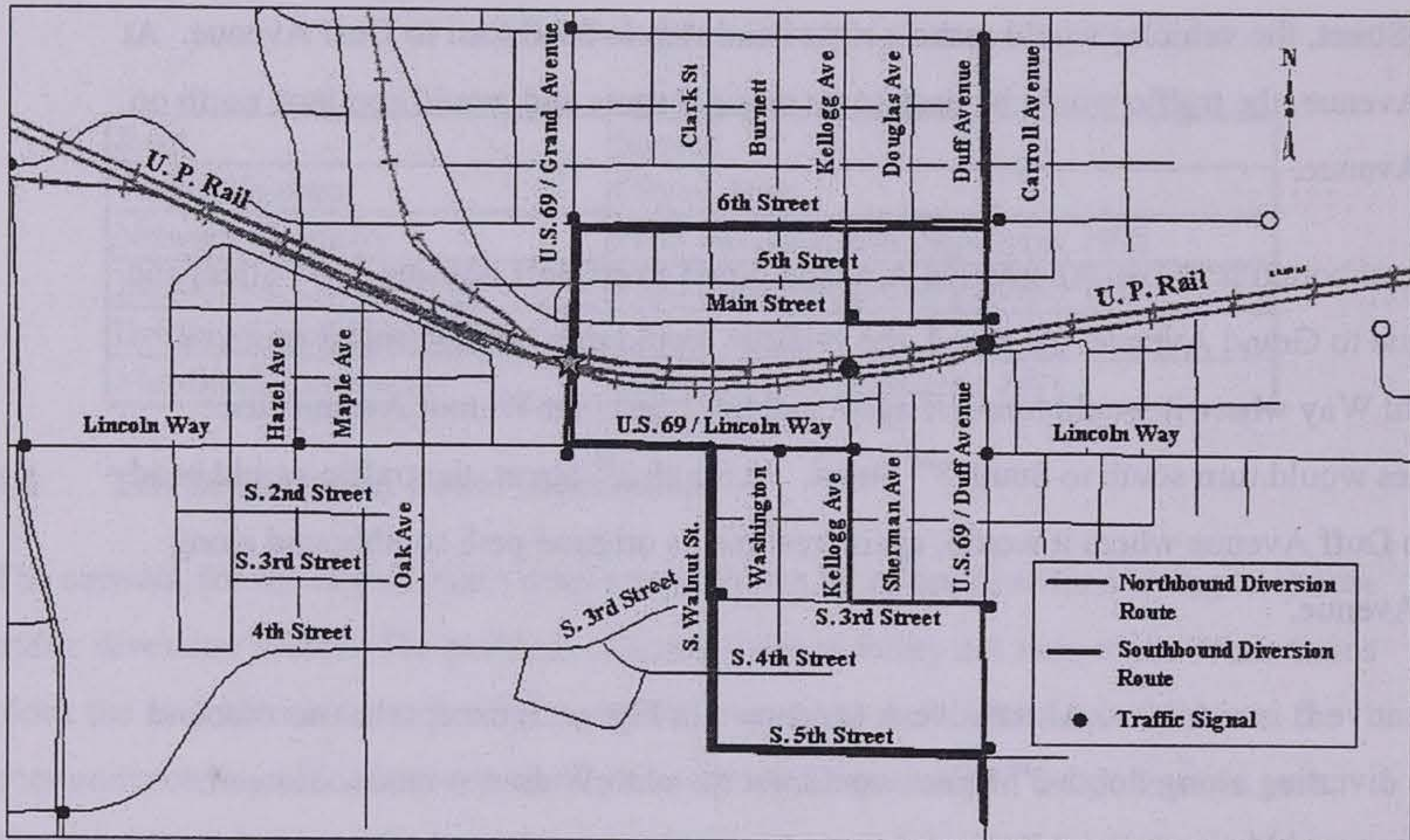

Figure 5. Alternative B

Both alternatives have similar characteristics (see Table 2). Using Alternative A, vehicles would encounter 8 traffic signals along the diversion route. Using Alternative B, vehicles would encounter 7 traffic signals. Northbound traffic would only encounter one four-way stop along the route for both alternatives while southbound traffic would encounter two four-way stops for both alternatives. Similarly, the northbound traffic in both plans would encounter three left turns while the southbound would encounter four left turns. Both of the plans allow traffic to not encounter the intersection of Duff Avenue and Lincoln Way, which is the most congest intersection in the downtown area at all times, especially when a train is blocking the crossing at Duff Avenue.

Table 2. Route Statistics

Path	# Signals	# Stop Signs 2-way	# Stop Signs 4-way	# Left Turns
Alternative A				
Northbound	8	0	1	3
Southbound	8	0	2	4
Alternative B				
Northbound	7	0	1	3
Southbound	7	0	2	4

4.4 Modeling Process

The Synchro software package has the ability to optimize and coordinate traffic signals. The input parameters allow the user to indicate which signals to coordinate and which phase at each signal to give priority to. Both Alternative A and B were modeled in Synchro and the signal timing plans were changed to give the diverted traffic priority over the other traffic on the network in the Ames CBD. Synchro optimizes its signal timing plans according to the principles found in the Transyt 7-F signal optimization tool. (6) The signals give priority to the phases as indicated but the optimization finds the timing plan that will minimize overall delay across the entire network. Once the traffic signal timing plans were established in Synchro, the model was transferred to CORSIM for analysis.

4.5 Results of Previous Research

The models for each alternative were run in CORSIM and the results were compared to those from the base model analysis. The results showed that both alternatives would decrease overall network delay, decrease in average delay along the diversion route, and decrease the travel time along the diversion routes (see Figure 6, Figure 7, and Figure 8).

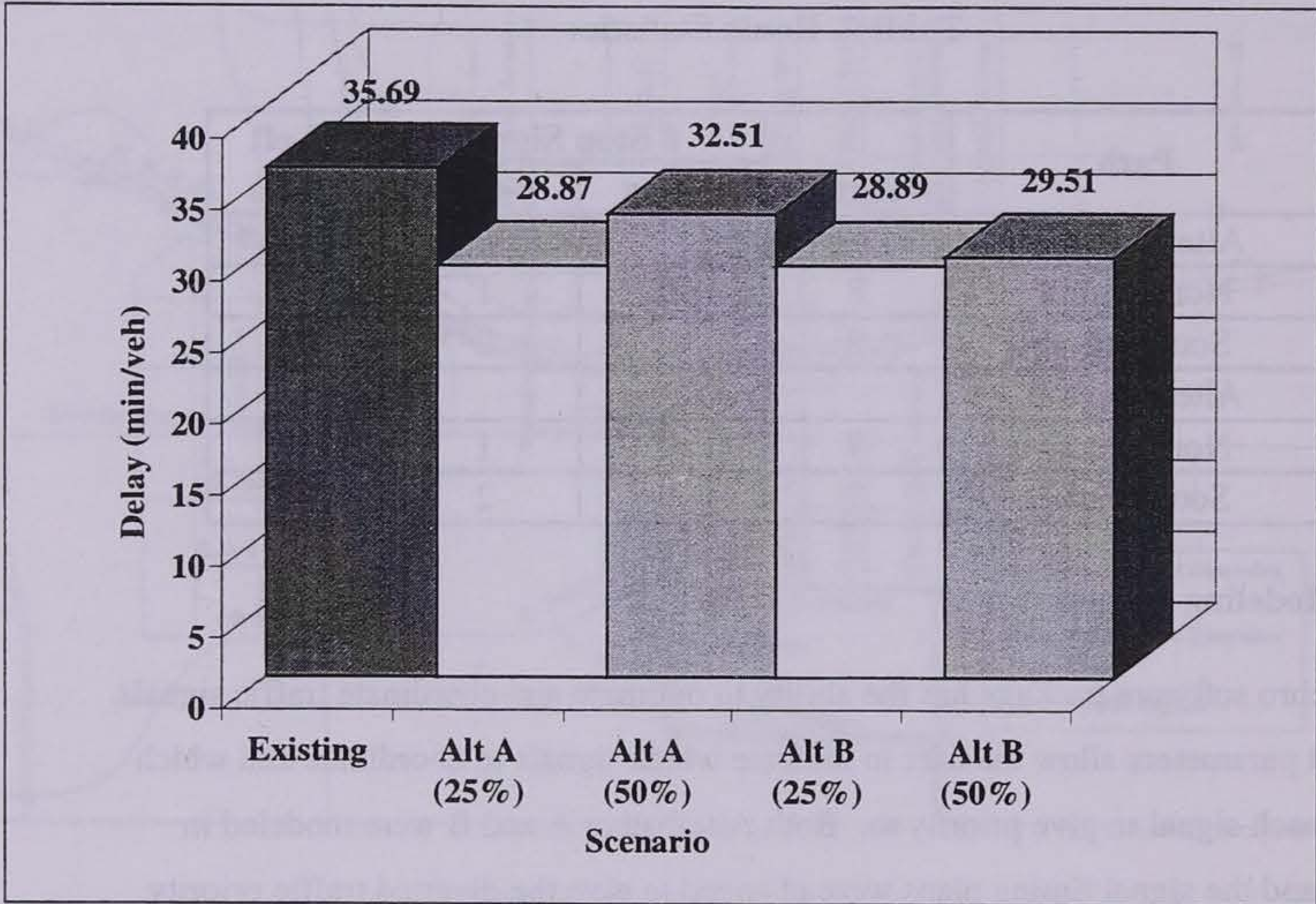

Figure 6. Overall Network Delay

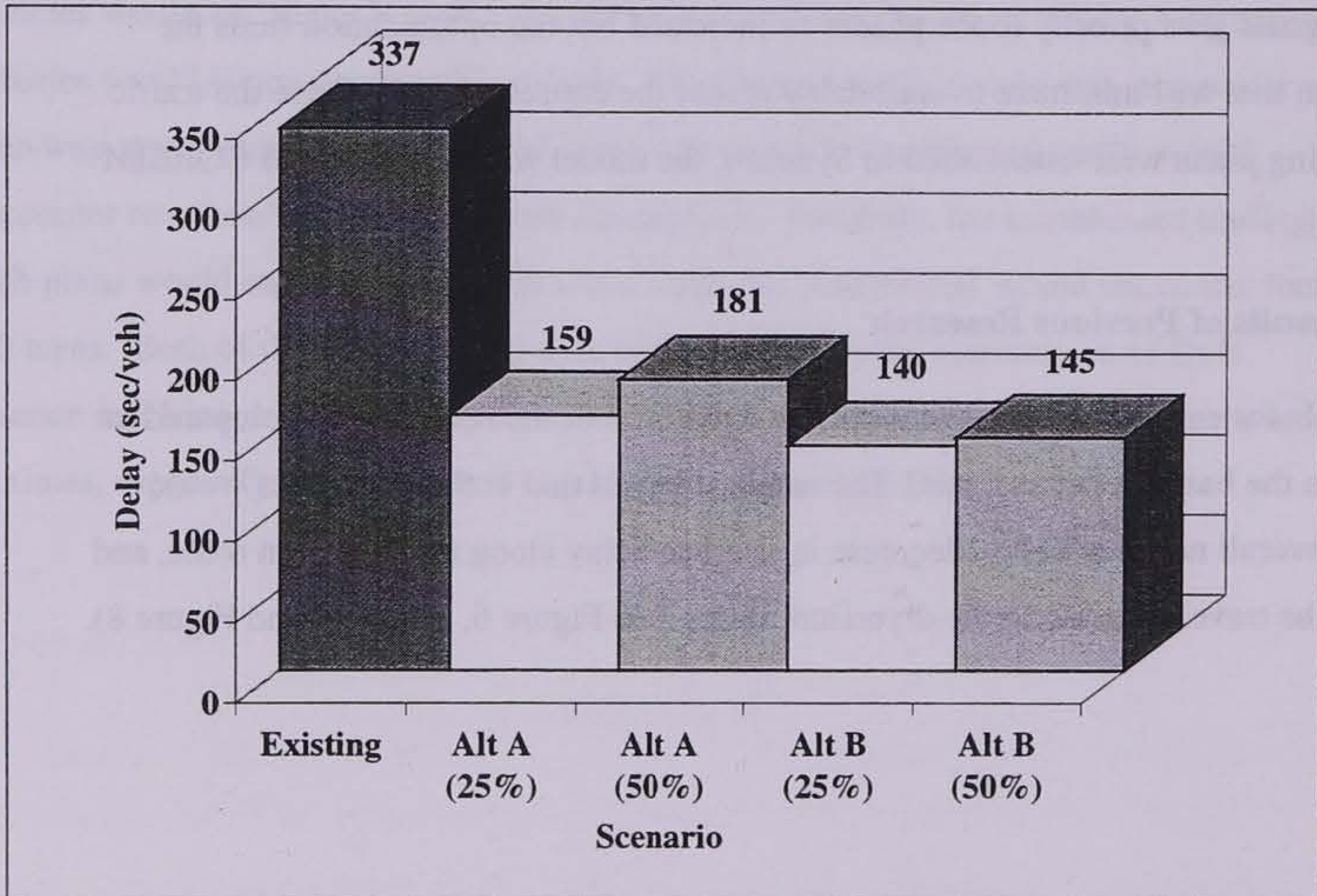

Figure 7. Average Delay along Diversion Route

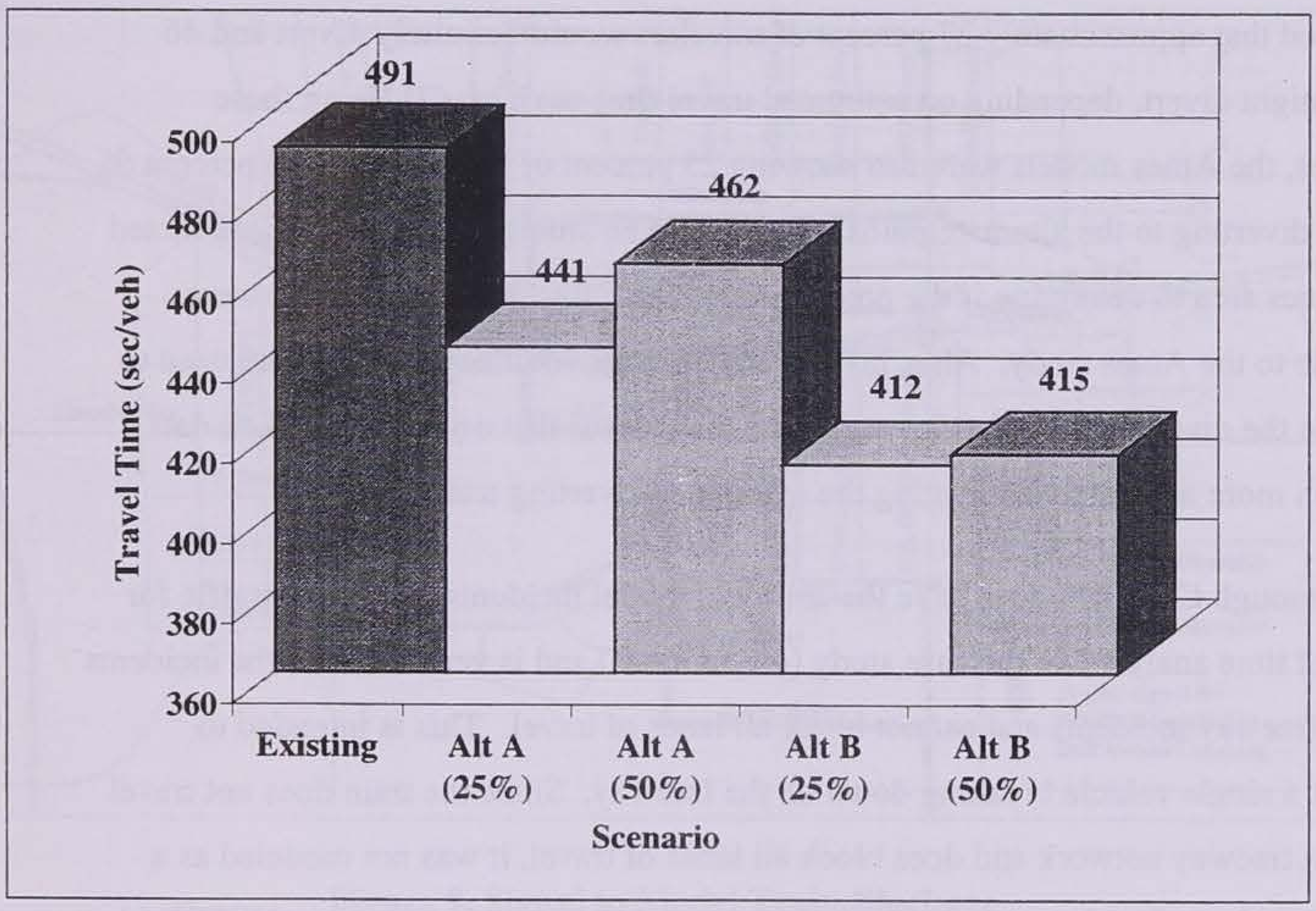

Figure 8. Travel Time

4.6 Deficiencies with Previous Research

While the simulation produced results that allowed for comparison of relative changes, there were deficiencies in the modeling process. This section will describe the deficiencies found in the case study research and identify areas for further research to address those deficiencies.

First, the diversion routes chosen for the Ames study were based on general knowledge of the area and did not take into consideration surrounding land use, roadway classification, or driver behavior and tendencies to divert from their intended path.

The second problem encountered with the previous research was the inability to calculate accurately the number of vehicles that would divert and to properly "load" the network to represent what might actually happen. A study done by the Upper Great Plains Institute was used to determine the number of vehicles that would divert due to the blockage. In this study, travelers in the Fargo-Moorhead area were surveyed to determine how many would divert if there were a blockage on the roadway that lasted 4-6 minutes. This study determined that approximately 27 percent of travelers would definitely divert and 46 percent might divert, depending on estimated travel time savings. (7) Using these guidelines, the Ames models were run showing 25 percent of travelers and 50 percent of travelers diverting to the alternate paths. There were no studies or interviews conducted in the Ames area to determine if the predictions of the Fargo-Moorhead study were applicable to the Ames study. Also, no land use or origin-destination data were used to determine the diversion. Further investigation may reveal that origin-destination data will aid in more accurately estimating the amount of diverting traffic.

Third, although CORSIM does have the ability to model incidents that block traffic for periods of time analyzed in the case study (4-6 minutes) and is very reliable, the incidents must be freeway incidents and cannot block all lanes of travel. This is intended to represent a single vehicle breaking down on the freeway. Since the train does not travel through a freeway network and does block all lanes of travel, it was not modeled as a freeway incident. Incidents can be modeled on the surface street network in NETSIM but these incidents are short-term interruptions that more closely represent parking maneuvers. Again, due to the misrepresentation of this incident, the train could not be accurately modeled using the NETSIM incident abilities. Instead, CORSIM has to be "tricked" into representing the train blockage by installing a signal at the intersection of the local street and the rail line as shown in Figure 9. (8)

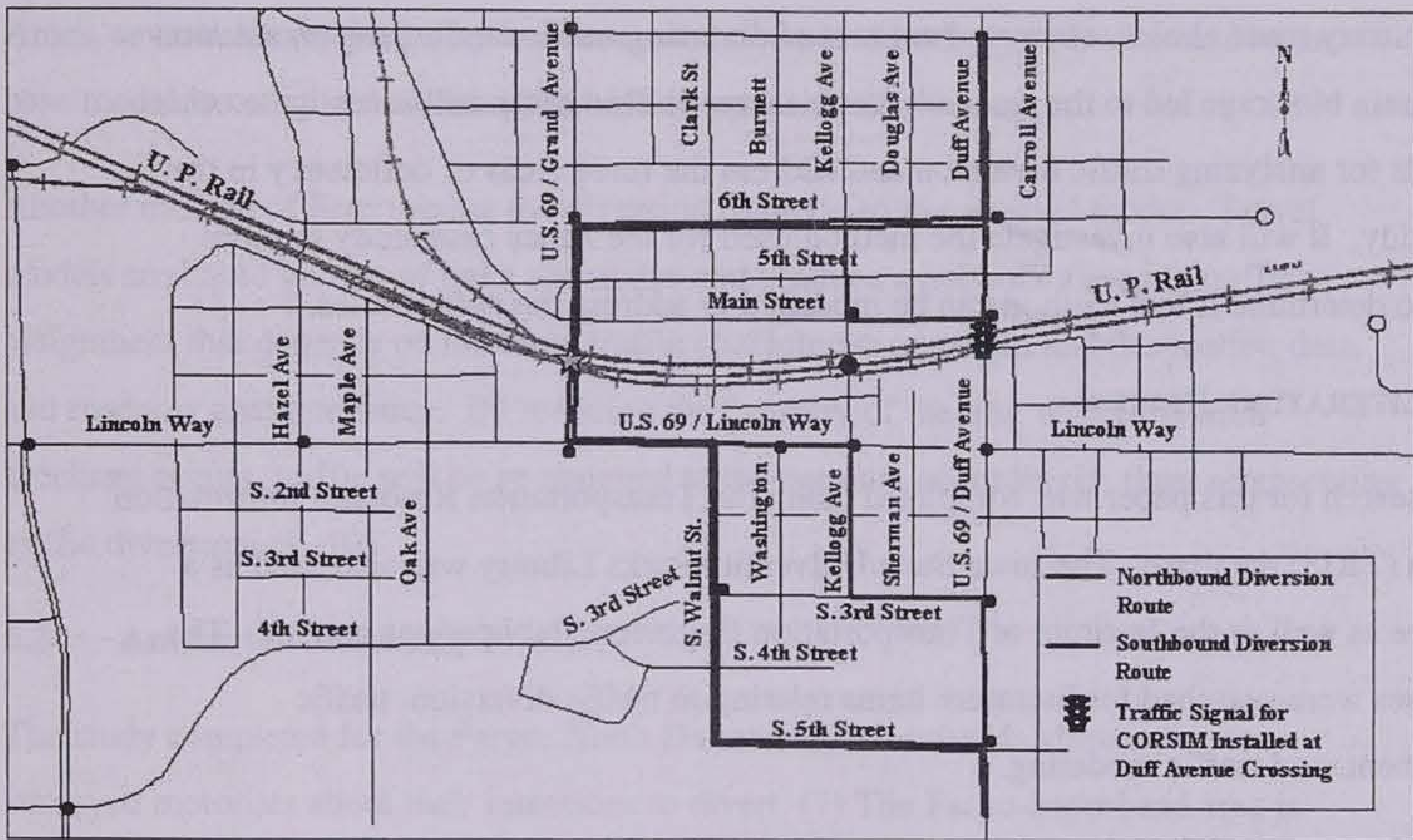

Figure 9. Signal to Model Train Blockage

The signal timing plan must have two settings. One plan provides the green phase to the traffic on the roadway to represent the intersection when there is no train present and the other setting provides the red phase to the traffic on the roadway for a length of time equivalent to the time the intersection is blocked by the train. CORSIM has a maximum cycle length of 120 seconds and a minimum green requirement on each phase of five seconds. This means during the period representing no train, the traffic on the roadway will be stopped for five seconds every two minutes. It also means that, during the period representing a train blockage, the traffic on the roadway will be given a five second green phase, thus, allowing vehicles to pass over the grade crossing. This again is not an accurate representation of the network operations. The analysis will compare the changes in MOEs from the base model to a model using the alternate routes. Since the models have the same inaccuracies, it is possible that the relative changes will not be affected by the inaccuracies. However, the objective is always to represent the system as accurately as possible. It is believed that there are other modeling tools that would more accurately represent a rail crossing.

The arbitrary route choice, estimated amount of diverting traffic, and misrepresentation of the train blockage led to the research for this paper. This study will investigate other methods for analyzing traffic diversion and address the three areas of deficiency in the case study. It will also investigate the method used for the Ames case study in more detail to determine if that method can be modified to address the deficiencies.

5. LITERATURE REVIEW

The research for this paper was completed using the Transportation Resource Information System (TRIS) database. The Iowa State University Parks Library was also used as a resource as well as the Institute of Transportation Engineers Publications catalog. The databases were searched for literature items relating to traffic diversion, traffic assignment, and traffic modeling.

This section will describe the results of the research as it pertains to the three stages in analyzing traffic diversion. The first stage is choosing diversion routes, the second stage is estimating the amount of diverting traffic, and the third stage is modeling and analyzing the network. Following these three sections, combinations of the three stages were combined to form four methods for the complete analysis of traffic diversion and the advantages and disadvantages of those methods are summarized.

5.1 Diversion Routes

Much of the literature found pertaining to traffic diversion dealt with real-time diversion techniques due to freeway incidents, which was not the focus of this study. Other literature discussed methods for encouraging diversion such as variable message signs, static signs, and other forms of intelligent transportation system (ITS) technologies. While methods for encouraging diversion will be beneficial to the project in the future, this study was aimed at developing a method to simply analyze a predicted diversion in a traffic model.

Choosing the diversion routes, while extremely important, is something that can only be done with knowledge of the area. Surrounding land use, roadway classification, and traffic characteristics must all be taken into consideration. As done in this case study for

Ames, several alternatives may be chosen and compared both to each other and to the base model in order to determine the best diversion routes to use.

Another method of determining the diversion routes is to use a travel model. Travel models are based on sets of links and nodes and produce a network showing traffic assignment that depends on land use, traffic characteristics, origin and destination data, and roadway characteristics. By reducing the capacity of the link where the train blockage occurs, traffic will be re-assigned to the network accordingly, thus, representing traffic diversion. (9, 10)

5.2 Amount of Diverting Traffic

The study completed for the Fargo, North Dakota and Moorhead, Minnesota area surveyed motorists about their intentions to divert. (7) The Fargo-Moorhead area is similar to Ames, Iowa with the Burlington Northern & Santa Fe rail line running through the heart of their CBD. (11) Fargo-Moorhead also had just a single separated grade crossing with nine at-grade crossings in their downtown area. The survey results showed that 27 percent of motorists would definitely divert to an alternate route and 46 percent of motorists said they might divert, depending on the estimated travel time savings.

A study done in Scotland surveyed motorists about their opinions on variable message signs (VMS). (12) The results of this study showed that 82 percent of motorists believed that VMS messages were accurate and 96 percent believed they were reliable. When asked about whether they would divert, however, only 13 percent said they would divert their path for predicted delays while over 90 percent would divert for a message such as "Bridge closed at Highway 10".

Trip origin and destination (O-D) data for the study network is also important and useful information. These data will show where traffic is attempting to travel to and how much traffic might be interested in diverting their route. Hobeika et al developed a model for traffic diversion to alleviate congestion in 1993. (13) The model contained an O-D trip table generator that used a linear programming technique to estimate O-D patterns from emerging link volumes. O-D data can also be obtained using survey methods, interviews, and other techniques to obtain real data from motorists who use the network, although some of these methods can be tedious and costly. There are also other packages to aid in estimating O-D patterns such as the Tranplan software by the Urban Analysis Group.

5.3 Simulation Modeling and Analysis

There are several simulation tools available for traffic network analysis. The four tools that applied to modeling traffic diversion due to incidents included CORSIM, Synchro, Highway Capacity Software, and INTEGRATION.

5.3.1 CORSIM

One of the most common tools is CORSIM, which was used in the city of Ames case study. CORSIM was developed under sponsorship of the Federal Highway Administration (FHWA) and is a microscopic traffic simulation tool that encompasses both the Network Simulation (NETSIM) and the Freeway Simulation (FRESIM) tools from Traffic Software Integrated Systems. CORSIM models traffic on networks of freeways and surface streets, treating each vehicle as a distinct object and moving them through space over time and recording measures of effectiveness in intervals of one second. Widely accepted driver behavior models determine the vehicle movements. CORSIM then produces predicted MOEs for the network such as travel time, average delay, average stopped delay, queue length, and other MOEs. (8)

5.3.2 Synchro

The signal optimization tool, Synchro, also performs its own network analysis. Its measures of effectiveness are similar to that of CORSIM. Synchro provides arterial speeds, number of stops, and level of service (LOS) based on delay experienced by vehicles. (6)

5.3.3 Highway Capacity Software

The Highway Capacity Software (HCS) is a great analysis tool for corridors. The HCS analyzes the entities in the network and is a widely accepted analysis tool developed by the National Research Council.

5.3.4 INTEGRATION

The last analysis tool investigated is the INTEGRATION package developed by the Transportation Systems Research Group at Queen's University in Kingston, Ontario, Canada. (14) It was developed to provide a single model to represent isolated functions of several models in an integrated environment. INTEGRATION is a microscopic simulation model although the macroscopic flow theory leads some to believe it is mesoscopic.

INTEGRATION was designed to evaluate integrated freeway and traffic signal network controls. (15) The model allows dynamic traffic assignment and traffic diversion. It incorporates microscopic lane- and car-following behavior as well as gap acceptance behavior at intersections. The platoon dispersion is modeled microscopically but is similar to TRANSYT's macroscopic process.

5.4 Network Analysis Options

After reviewing the literature and taking into consideration the importance of choosing an efficient route and determining percent of traffic diverting, several options for methods to model and analyze the diversion were determined from the above options. These included the following:

- Highway Capacity Software
- CORSIM with a signal optimization program such as Synchro (as used in the case study)
- CORSIM in conjunction with an O-D model
- The INTEGRATION simulation software

5.4.1 Highway Capacity Software

The Highway Capacity Software (HCS) is a great analysis tool but does not fit the needs of this project. The HCS only analyzes the entities in the network and does not model anything. (16) There is no way to change the network for analysis using the HCS, meaning that the traffic diversion would need to be calculated by hand.

5.4.2 CORSIM with Signal Optimization Program

This is the method that was used in the case study for the city of Ames. Since CORSIM does not optimize signal timing plans, an additional signal optimization program must be used in conjunction with the simulation model. Synchro was the signal optimization tool chosen for the case study. Its user-friendly graphical interface allowed the network to be built in Synchro, the signals optimized, and the network with signal timings transferred to CORSIM for analysis.

While CORSIM is a widely accepted traffic simulation and analysis tool, Synchro has the ability to optimize traffic signals in a network. The user interface for Synchro is also more user-friendly than CORSIM, allowing for easy input of traffic volumes, intersection control, turn movements, and geometry. Signal timing plans may also be input as applicable. The signal timing can than be optimized for each signal individually and then coordinated throughout the network. This optimization is based on the principles used in the Transyt 7F optimization package. (6)

Although Synchro does perform its own analysis and provides MOEs, CORSIM's history has proven it to be highly accurate and is widely accepted among traffic simulation experts. Once the network is built in Synchro, the data may be transferred to CORSIM for the analysis, as was done in the case study for Ames.

5.4.3 CORSIM with Travel Model

Another option is to use CORSIM for analysis and an urban transportation modeling system such as QRS II or Tranplan to make the traffic network assignment. More specifically, the traffic diversion patterns would be determined by the urban transportation modeling package, eliminating the need to "trick" CORSIM into modeling the train blockage. Both of these software packages base the networks on links, nodes, and traffic analysis zones. (9, 10) Traffic assignment is done by determining the productions and attractions in each zone using the four-step modeling process briefly described as follows: (17)

1. Trip Generation - In the trip generation step, the network is divided into zones called traffic analysis zones (TAZs) and the number of trips that originate and terminate in each TAZ are determined. This is typically done by using explanatory variables about the traffic analysis zone such as income level, household size, and automobile ownership to predict the number of trips. A trip consists of two "trip ends" with the origin considered a trip production and the destination considered an attraction. For example, trips are produced at a home and attracted to either a workplace or shopping area. The total number of productions and attractions in each TAZ are determined.

2. Trip Distribution - In the trip distribution step, the trips produced in one zone are allocated to another attraction zone. This is often done using a model that distributes trips among zones based on each zone's attractiveness. In a model, adjustments can be made for the impedance of travel between zones.

3. Mode Split - In the mode split step, the travel mode is determined for each trip. Many times, the proportion of trips made by modes other than automobiles is so small that this step is left out. In areas with larger portions of the trips made by other forms of transportation such as public transit, this step is important.

4. Traffic Assignment - This step is the final step in the modeling process. Here, the distributed, split trips are assigned to the network. Various algorithms are used to assign traffic to the network with the most common being the shortest path algorithm.

Both of these packages are able to model urban networks. The main difference between them is that QRS II is for a smaller network or parts of large networks while Tranplan has the ability to model large networks. The results from either of these two packages will be a network of links and nodes with volumes assigned on each link. It would be possible with either package to block links and re-assign the traffic to represent a network with incidents along the blocked links.

In research completed at Iowa State University, by Michael Anderson, the problem of transferring data from Tranplan to CORSIM was addressed. Anderson developed a method in which traffic assignment is done using Tranplan, transferred to a geographic information system (GIS), and exported to CORSIM for analysis. In this case, MapInfo was the GIS tool used. The GIS tool allows the network attributes to be updated. The centroids from Tranplan are renumbered to fit the CORSIM node numbering system. The MapBasic program in MapInfo then assimilates turning movement volumes for the intersections selected for the subnetwork. The network is then exported to CORSIM for analysis and calculation of the MOEs. Due to the lack of interface between the three software packages used, FORTRAN programs were written and used as the interfaces for the research by Anderson. (10)

Anderson modeled incidents in his research by using the method described above. In Tranplan, the links that were to be blocked had their capacity reduced to zero so that, when assigning traffic, these links were not able to have traffic on them. This produced trip tables that showed how traffic would be assigned to the rest of the network when specified links were not available. (9)

The networks compared networks during peak hours with and without incidents at specific points. The objective of the research done by Anderson is similar to that of the Ames case study. The research produced a method to allow interfacing between three tools that have been deemed necessary in traffic diversion modeling. The method used brings together the benefits of regional models and microscopic models in what is termed "pseudo-dynamic" modeling. (10)

5.4.4 INTEGRATION

The last option is to do the traffic assignment and analysis using INTEGRATION. With INTEGRATION, the traffic demand can be specified as a time series histogram of origin and destination (O-D) rates for each O-D pair in the network. Departure rates are defined in the data input files and those, in turn, generate the individual vehicles in the network. The process for the calculation of departure lists is shown in Figure 10.

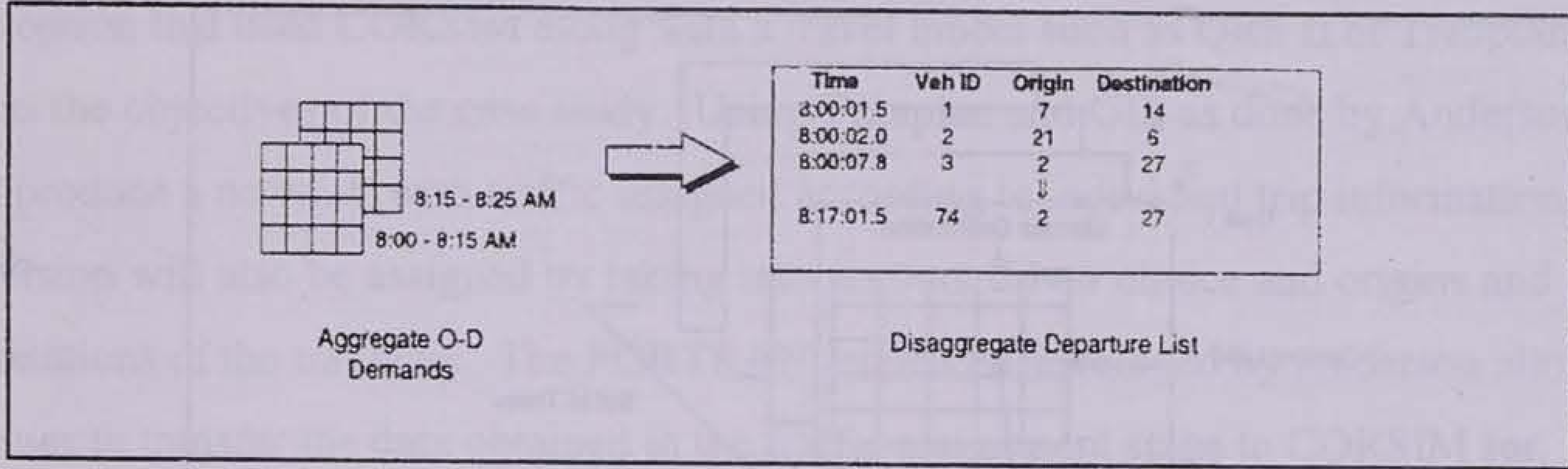

Figure 10. Departure Assignment from O-D data

Incidents in INTEGRATION are modeled by reducing the capacity of lanes by 0 to 99 percent. Incidents can start at any specified time, have any duration, and be of any severity. The incident will reduce the saturation flow, maximum speed, or availability of the lane in the given link. INTEGRATION also takes into account the fact that drivers do not change their routes the instant an incident occurs. There is a time lapse where drivers identify and perceive a situation and then make a decision on the information they have. On the other end, INTEGRATION also sustains diversion patterns after the incident is cleared to represent residual queues that remain on the network and cause on-going delays.

The measures of effectiveness (MOEs) output by INTEGRATION include link travel time as a weighted sum of the speeds that vehicles experienced while on the link. The travel time is measured as the difference between the time of entry onto the link and the time of exit off of the link. Drops in speeds are also considered a partial stop and recorded as such. The sum of the partial stops is used along with the number of full stops to calculate the total number of stops.

As stated above, the ability to accurately predict the paths of diverting vehicles and modeling that is important to produce usable results. INTEGRATION takes into account that drivers' paths are not always predetermined. Drivers often choose their path based on information obtained during the trip rather than before their trip. The route is determined link by link for each vehicle's trip by using a routing look-up table format based on the O-D of that vehicle as shown in Figure 11. (14)

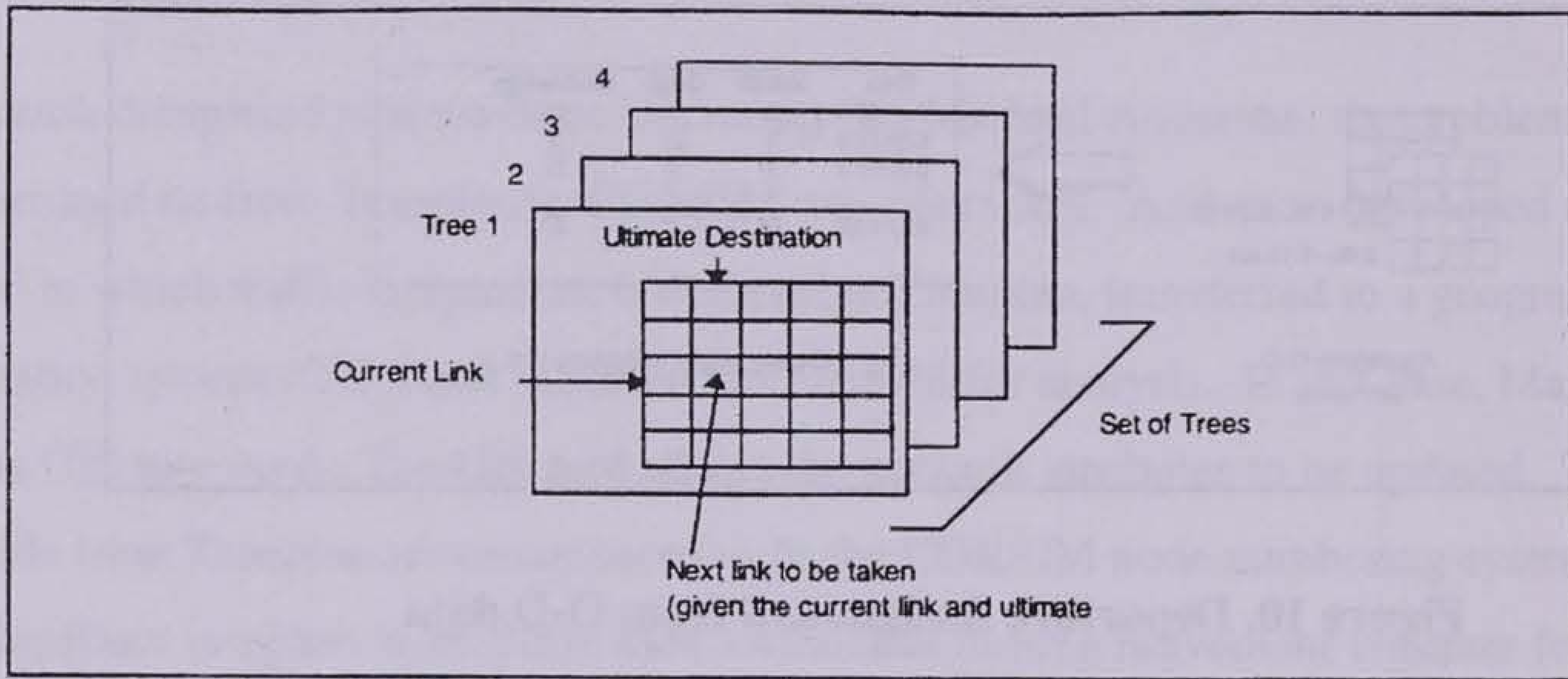

Figure 11. O-D Look-up Tables

6. CONCLUSIONS

All of the options to model and analyze traffic diversion had practical application to the city of Ames case study. They each provided means of measuring performance through individual measures of effectiveness. Not all of the methods, however, provided simple means of determining the amount and direction of diverted traffic.

The Highway Capacity Software performs excellent and reliable analysis but does not provide any tool for determining the amount of diverting traffic. The Highway Capacity Software is also not a modeling tool, therefore, the train blockage could not be modeled. The HCS is simply an analysis tool and its only use may be to provide additional analysis for comparison of outputs from other methods used.

The CORSIM and Synchro combination was previously used and was investigated as a solution to determine whether there was a way to use the two packages together to model the traffic diversion more accurately. Lack of knowledge when the original model for the city of Ames was built led to the belief that there may be capabilities in both packages that were not discovered during that time. The research done for this study has shown that it is possible to model the train blockage but because both software packages have to be "tricked" into representing the train blockage by installing a signal at the intersection, this method is not the most accurate method to use. This option also does not provide a quantitative method for estimating the amount and paths of diverting traffic.

The option that used CORSIM along with a travel model such as QRS II or Tranplan meets the objectives of the case study. Using Tranplan and GIS as done by Anderson will produce a network with traffic assigned according to individual trip information. (10) Diversion will also be assigned by taking into account driver choice and origins and destinations of the travelers. The FORTRAN interfaces developed by Anderson allow the user to transfer the data obtained in the traffic assignment stage to CORSIM for system analysis. This method has the advantage of detailed output from the microsimulation part of the process and the advantage of traffic assignment on a regional basis from the Tranplan-GIS environment.

INTEGRATION also appears to be a good candidate for the Ames case study. This is a unique tool that encompasses all the aspects that are needed for building this particular traffic diversion model. It performs traffic assignment using a series of O-D tables based on input by the user. It also models incidents in real-time, unlike the Tranplan-GIS-CORSIM method. INTEGRATION also accounts for the fact that drivers do not always determine their routes a priori, sometimes because information on incidents may not be available to them at that time. INTEGRATION encompasses a lot of the car-following and platoon dispersion theory that is used in other traffic models. The other advantage that INTEGRATION has is that it is all one package and no interface between software packages is necessary. Having only one software package to use also greatly simplifies the input and analysis of the system.

Due to its simplicity and all-encompassing style, INTEGRATION appears to be the most appropriate model to use to accurately model traffic diversion. The continuous nature of the system allows the incident to be modeled at any time for any duration and any severity. The level of detail available in INTEGRATION, especially in determining the portion of the roadway to be blocked, make this simulation package highly desirable. The Tranplan-GIS-CORSIM method also had a high level of detail but the three different software packages necessary and the development of the interfaces between them makes this option complicated. The probability of error occurring between the programs is also a disadvantage of the system.

7. Further Recommendations

INTEGRATION appears to be the method of choice for modeling and analyzing traffic diversion patterns, however, it is a fairly new tool developed in just the last decade. The CORSIM output is so widely accepted that it produces hesitation when choosing a model like INTEGRATION to perform an analysis instead of CORSIM. It is recommended that more research be done on INTEGRATION and its applications.

Further investigation into how INTEGRATION is used, how reliable its results are, and how accepted it is in industry will solidify the choice to use INTEGRATION over CORSIM for analysis. It would be beneficial to look for comparison studies of INTEGRATION to other accepted simulation models for various applications, not just traffic diversion.

It is anticipated that further investigation into INTEGRATION, its applications, and its accuracy will show that INTEGRATION is comparable to other widely accepted models. If the further research shows otherwise, outputs from INTEGRATION may be adjusted to better represent the real network.

The concept of having the traffic assignment, route choice, diversion patterns, and signal timing in one package is ideal and carries a lot of potential for use in a variety of scenarios.

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