GEOTECHNICAL AND PAVEMENT MANUAL

Part 1

MINNESOTA DEPARTMENT OF TRANSPORTATION

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Chapter One
GENERAL

1-1.0 PURPOSE

The purpose of this Geotechnical and Pavement Manual is twofold. One is to give the District Soils Engineers the required reference material to complete the design of pavement sections and uniform, stable, highway subgrades, embankments, and back slopes. The other is to provide the direction required to achieve consistency in the way geotechnical and pavement problems and solutions are approached.

1-2.0 SCOPE

This manual provides background information in geotechnical engineering and pavement design. It also includes guidance for performing geotechnical and pavement surveys, preparation of design recommendations, and submittal of the Design Recommendations Report (formerly called the Soils Letter). It is based on past Minnesota Department of Transportation (referred to herein as Mn/DOT or "the Department") experience, the experience of other groups and agencies with similar requirements, and the current state of the practice in geotechnical and pavement engineering.

The intent is that this manual will stand on its own; however, to avoid unnecessary duplication, reference is often made to other available manuals and publications of the Department, as well as other references which are readily available. Additionally, since it is impractical to be totally encompassing, and since the state of the practice is continually evolving, some subjects are simply referenced for further research and study as needed.

1-3.0 RESPONSIBILITY

Generally, the District Soils Engineer will apply the presented information under the guidance and supervision of the District Materials Engineer. Monitoring of this manual's use will be through the review of Design Recommendations Reports by the Office of Materials Research & Engineering, as they are routinely submitted during the design of projects, and the historic performance of completed highways following construction.

1-4.0 REVIEW AND REVISION

Comments and/or recommendations, which may ultimately lead to revision of this manual, should be submitted in writing to the Geotechnical Engineer or the Pavement Engineer. The comments and/or recommendations will be reviewed by the Office of Materials Research & Engineering. It is anticipated that updates will be developed and distributed by the Office of Materials Research & Engineering via memoranda as required; more complete reviews will be performed and revisions issued approximately biennially.

Chapter Two
GEOLOGY

2-0.0 IMPORTANCE OF GEOLOGY

"Geology is the study of the planet Earth - the materials of which it is made, the processes that act on these materials, the products formed, and the history of the planet and its life forms since its origin." (from Glossary of Geology, Bates & Jackson, 1987). It is evident from such a definition that geology encompasses a wide range of studies. It is an equally wide range of processes that have combined over geologic time to produce the interesting and varied surface and subsurface materials that we now find in the state. In highway engineering, we struggle to identify, classify and quantify these earth materials as we encounter them in our construction. To gain an understanding of the materials with which we work, whether they are rock, soil, or something in between, it is important to have some knowledge of their geologic origin, and how subsequent geologic processes have altered them.

Some of the basic objectives of geotechnical investigations are to define the nature, characteristics, properties, thickness, and lateral extent of bedrock and overlying soils as well as ground water conditions within a given project area. An understanding of some of the basic geologic processes and how they combined to create the varied landscapes in our state, give us a base line of information from which to begin more detailed studies of the subsurface.

2-0.01 A BRIEF OVERVIEW

The geology of Minnesota is both complex and diverse. The geologic history includes periods of mountain building and subsequent erosion, folding and faulting, a near tearing apart of the continent, extensive volcanism, and submersion by epicontinental seas with subsequent deposition of thousands of feet of sedimentary rocks. All of these events took place prior to the start of the Quaternary Period, which began about two million years ago. The bedrock skeleton that was created over eons of time is buried in all but the southeastern and northeastem portion of the state by more recent, unconsolidated Quaternary deposits.

Most of the present land surface in Minnesota was developed during the Quaternary Period, which includes the Pleistocene (Glacial) and Holocene (Recent) Epochs. In light of this, much of the emphasis of this chapter will be on Quaternary geology, including glacial soils and landforms, recent soil types, and hydrogeology. The bedrock geology will be discussed only in generalities because it is too complex and diverse to cover within the scope of this manual.

All of the topics presented in Chapter 2 fall under the sphere of geology, and questions or problems in characterizing soils or ground water conditions should ultimately be directed to the Geology Unit. The subject of bedrock geology is nearly exclusively dealt with by the Geology Unit.

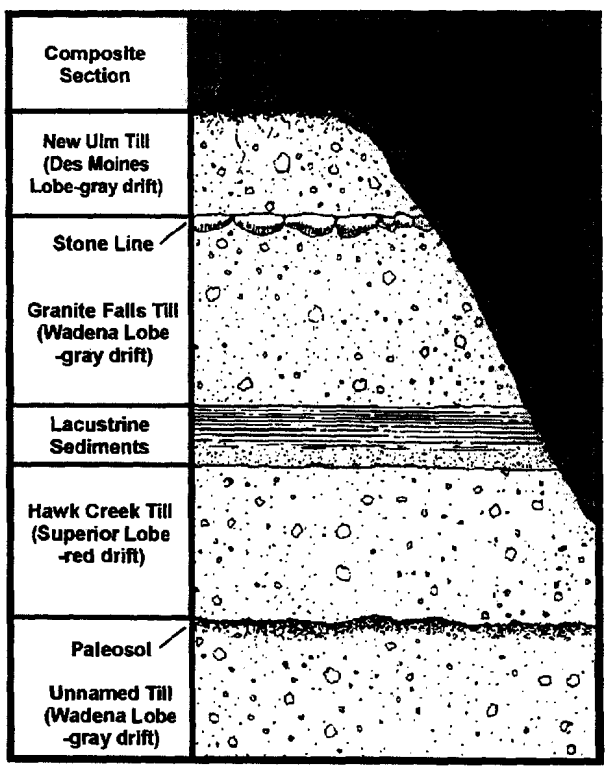
2-1.0 QUATERNARY GEOLOGY

The start of the Quaternary Period was marked by cyclic variations in the climate, resulting in successive advances and recessions of glacial ice sheets across Minnesota. This time period, which is referred to as the Pleistocene Epoch, or the "Great Ice Age," has been subdivided into four, major, glaciations, with interglacial periods. These are from youngest to oldest:

Present interglaciation
Wisconsin glaciation
Sangamon interglaciation
Illinoin glaciation
Yarmouth interglaciation
Kansan glaciation
Aftonian interglaciation
Nebraskan glaciation

Glacial intervals are recognized by various glacial sediment sequences, including glacial till and granular materials. Interglacial periods are recognized by evidence of soil development; fossils and pollen; and from sediments which indicate warmer climate, such as lake deposits, beach ridges, and peats. Figure 2-1.1 illustrates a succession of glacial advances and intervening episodes of soil formation during the Wisconsin glaciation as exposed along Hawk Creek in Renville County.

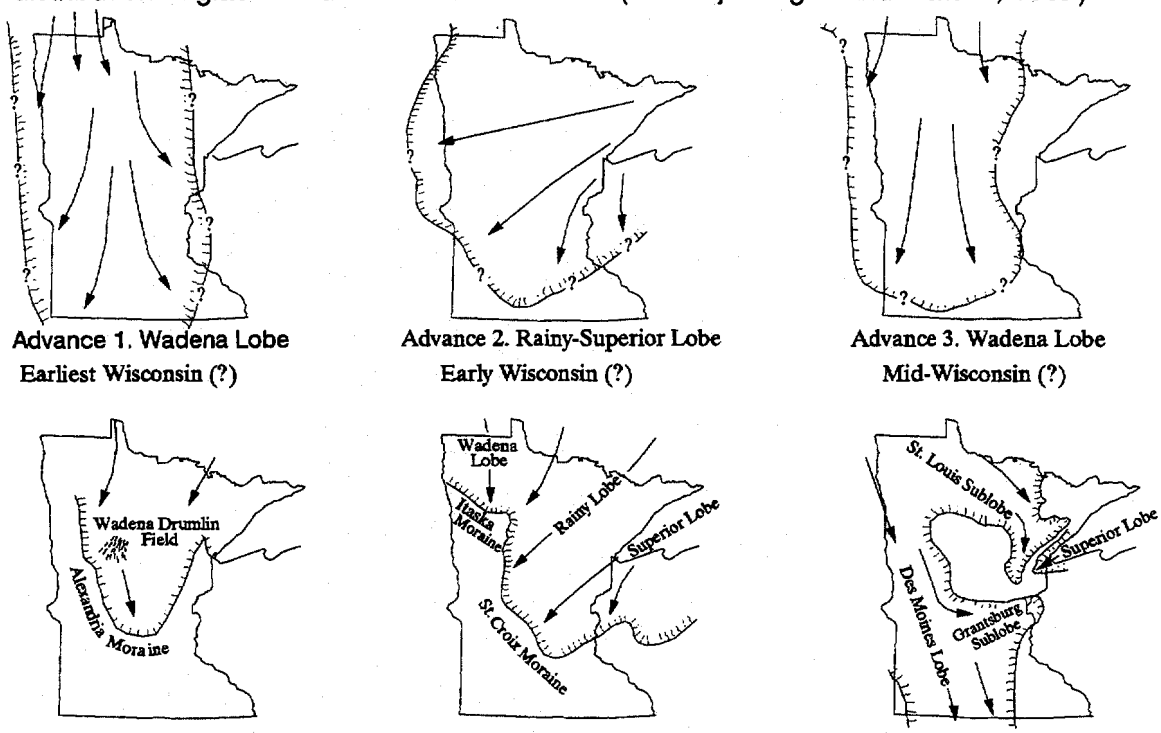
Figure 2-1.1
Composite section showing glacial deposits (after Ojakangas and Matsch, 1982)



During the Pleistocene Epoch, the movement of ice and ice-bound debris constituted a strong abrasive force, and most of Minnesota bears the scars of this force. The final glaciation, the Wisconsin, lasted from about 75,000 to 12,000 years before the present time and was responsible for the majority of the landforms seen in Minnesota today. Glacial activity during the Wisconsin glaciation has eroded or buried earlier glacial deposits from the Nebraskan, Kansan, and Illinoian glaciations, largely obscuring evidence of these earlier deposits.

Wisconsin glaciation is characterized by advances and retreats of ice lobes that protruded from the main ice sheet that covered most of Canada (Figure 2-1.2). The movement of the ice lobes across Minnesota was controlled by major bedrock topography of the area over which the ice moved. In general, the ice lobes followed lowlands that were developed on the soft sedimentary rocks that flank the exposed Precambrian igneous/metamorphic rocks of the Canadian Shield. In eastern Minnesota the Superior lowland (roughly aligned with the axis of Lake Superior) was a trough for the **Superior Lobe**, and separated by the igneous highlands of the North Shore, the **Rainy Lobe**. On the western side of the state, the major lowland is the Red River Valley, through which the **Wadena Lobe** and **Des Moines Lobe** pushed their way, principally south, across the state.

Figure 2-1.2
Summary of ice activity during the Wisconsin glaciation as reconstructed from the distribution of glacial sediments and landforms (after Ojakangas and Matsch, 1982)



Advance 1. Wadena Lobe
Earliest Wisconsin (?)

Advance 2. Rainy-Superior Lobe
Early Wisconsin (?)

Advance 3. Wadena Lobe
Mid-Wisconsin (?)

Advance 4. Wadena-Rainy Lobe
Mid-Wisconsin (?)

Advance 5. Rainy-Superior Lobe
Late Wisconsin

Advance 6. Des Moines Lobe
Late Wisconsin

The Superior Lobe expanded out of the Lake Superior basin bringing with it red sandy till. In northeastern Minnesota the Rainy Lobe deposited drift of variable color and composition, depending on the rock types it traversed. In Western Minnesota, the Des Moines Lobe followed the Red River Valley into Iowa; it sent sublobes eastward across northern Minnesota (St. Louis Sublobe) and southern Minnesota (Grantsburg Sublobe), bringing calcareous silty till with fragments of Cretaceous shale. Down the center of the state rode the Wadena Lobe, which brought gray to buff sandy calcareous till and carbonate rock fragments from the terrain of Manitoba.

2-1.01 PLEISTOCENE SURFACE FEATURES

Most of the surface features created during the Pleistocene Epoch are the result of either direct contact with glacial ice or from the action of glacial meltwater. **Glacial erosional features** were created by the gouging and/or scraping of the underlying bedrock or older glacial deposits, or by glacial meltwater. These topographic erosional features include **striations, whalebacks, roche moutonnees**, **flutes,** and **tunnel valleys**.

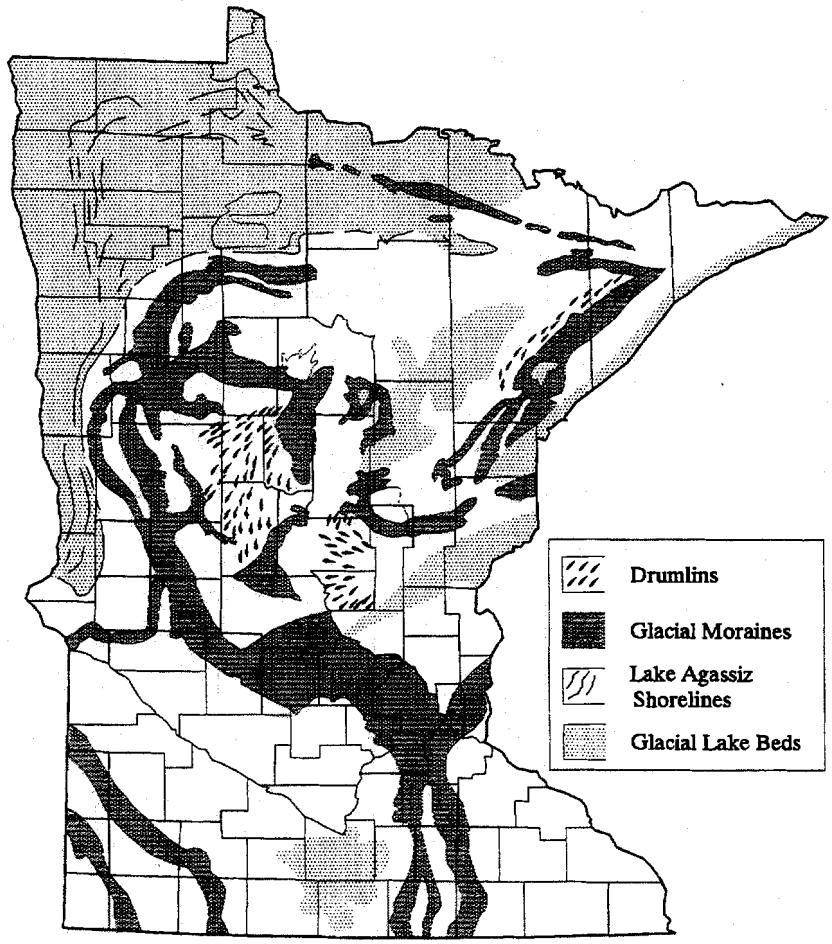
Glacial depositional features are more common and are the result of material dumped by the ice as it rode over the land surface or left behind as the ice melted or retreated. All material deposited by glacial action, whether it be directly from the ice or indirectly from meltwater, is classified as **Drift**.

Drift deposited directly by glacial ice with no sorting action, include **erratics** and **till**. Materials that are deposited by waters associated with the glacial ice include; **glaciofluvial**, which are deposited by a stream or river originating from glacial meltwater, and **glaciolacustrine**, which are sediments deposited in lakes bordering and/or supplied by the glacier. Deposits in/from meltwater exhibit some degree of sorting and are often stratified. Also associated with glacial and post-glacial activity are wind-blown or aeolian deposits.

Topographic depositional features include **moraines, drumlins, kames, eskers, lake plains**, **outwash plains** and **till plains**. The terms "outwash" and "till" have been used in the literature to describe both topographic landforms and soil types. However, they both possess very unique and easily identifiable soil types, and as such, it has been useful to apply them as an adjective to the basic soil type description. To avoid confusion, the term "plain" will be added to the description when a topographic landform is being described in this manual.

Many topographic features (landforms) are associated with very narrow ranges of soil types; e.g., lake plains with clay to silty clay, eskers with sand and gravel, and moraines with sandy loam to clay loam. Most landforms visible in Minnesota today are the result of the Wisconsin glaciation - the final advance of glacial ice to blanket the state (Figure 2-1.3). Many of the landforms and associated glacial sediments, discussed in the following sections, and their major areas of occurrence in the state, can be seen in the 1982 wall map "Geologic Map of Minnesota - Quaternary Geology" by Howard C. Hobbs and Joseph E. Goebel, produced by the Minnesota Geologic Survey. (A reduced/simplified version of this map is included as Figure 2-1.9).

Figure 2-1.3
Major landforms of the last glaciation (after Ojakangas and Matsch, 1982)



2-1.01.01 GLACIAL EROSIONAL FEATURES

Bedrock outcrops that have survived the abrasive forces of glaciers display distinctive scars. **Striations** occur as parallel linear scratches on the bedrock, caused by movement over the bedrock surface of individual rock fragments embedded in the base of the ice. Quarrying or plucking, is glacial erosion on a much larger scale. Bedrock weakened by fractures is pulled away by the overriding ice and removed by the glacier. This process occurs most frequently on the lee side of the bedrock feature. Differential quarrying accompanied by abrasion of the front side of the rock body produces a feature known as a **whaleback**. Whalebacks are typically small (on the order of a few meters long), elongate bedrock outcrops where the upstream flank is gently planed and smooth and the downstream flank is steep and ragged. A **rouche moutennee** is a feature similar in origin and appearance, but on a much larger scale. These bedrock erosional features are much more common in the northern portion of the state, where erosion resistant bedrock is exposed.

Flutes appear as fields of long, parallel, gutter-like grooves developed in either weak bedrock or glacial deposits. These grooves parallel the direction of ice flow and were created when actively flowing ice scoured the underlying materials. Inland from Lake Superior, between Two Harbors and Lutsen, the bedrock landscape in dominated by the Highland Flutes.

Tunnel valleys are trenches cut in drift or bedrock by large quantities of flowing subglacial meltwater. They may be up to 200 feet deep, a half mile in width, and extend for over a hundred miles. Typically they are developed in pre-existing glacial drift, although they may also form on the surface of soft bedrock. Tunnel valleys could also be discussed under topographic depositional features, because many erosional valleys are subsequently filled by younger drift. Occasionally, when the quantity of meltwater lessens and the subglacial tunnel chokes with sediments, an esker will develop within the course of the tunnel valley. In central Minnesota, near Alexandria; and in east central Minnesota, roughly between Hinckley and Anoka, the tunnel valleys are often filled with granular material and may have the surface expression as a chain of small elongate lakes.

2-1.01.02 GLACIAL DEPOSITIONAL FEATURES

In considering glacial depositional features care must be taken to discriminate between glacial deposits (soils) and topographic landforms. The all encompassing term that describes glacial deposits is **drift**. Drift includes all the material ever handled by the ice even if it is subsequently affected by wind or water. Abundant glacial drift deposits, some more than 600 feet thick, cover the face of Minnesota (Figure 2-1.4). Drift from glacial advances older than the Wisconsin glaciation are largely obscured, either by erosion or by burial beneath younger deposits. Even the earliest deposits of the Wisconsin glaciation are difficult to identify because of subsequent advances and retreats of the ice lobes.

An important early observation on the glacial deposits of Minnesota was the recognition of two basic types of drift, described as **red drift** and **gray drift** according to their color. The red drift is typically sandy and contains rock fragments from the Precambrian bedrock (basalt, gabbro, and red sandstone) around the Lake Superior area. The gray drift is silty, calcareous and contains abundant carbonate rock fragments from the Paleozoic sedimentary rocks in the Winnepeg area. The red drift is associated with the Superior and Rainy lobes and is found exclusively in eastern parts of the state (Figure 2-1.5). The gray drift is associated with the Wadena and Des Moines lobes. The Des Moines lobe, because of its more westward origin tracking across eastern North Dakota, carried with it fragments of a hard, gray to greenish-gray shale, plucked from the Cretaceous Pierre Shale of North Dakota (Figure 2-1.2).

Figure 2-1.4 Generalized thickness of glacial drift (from Ojakangas and Matsch, 182)

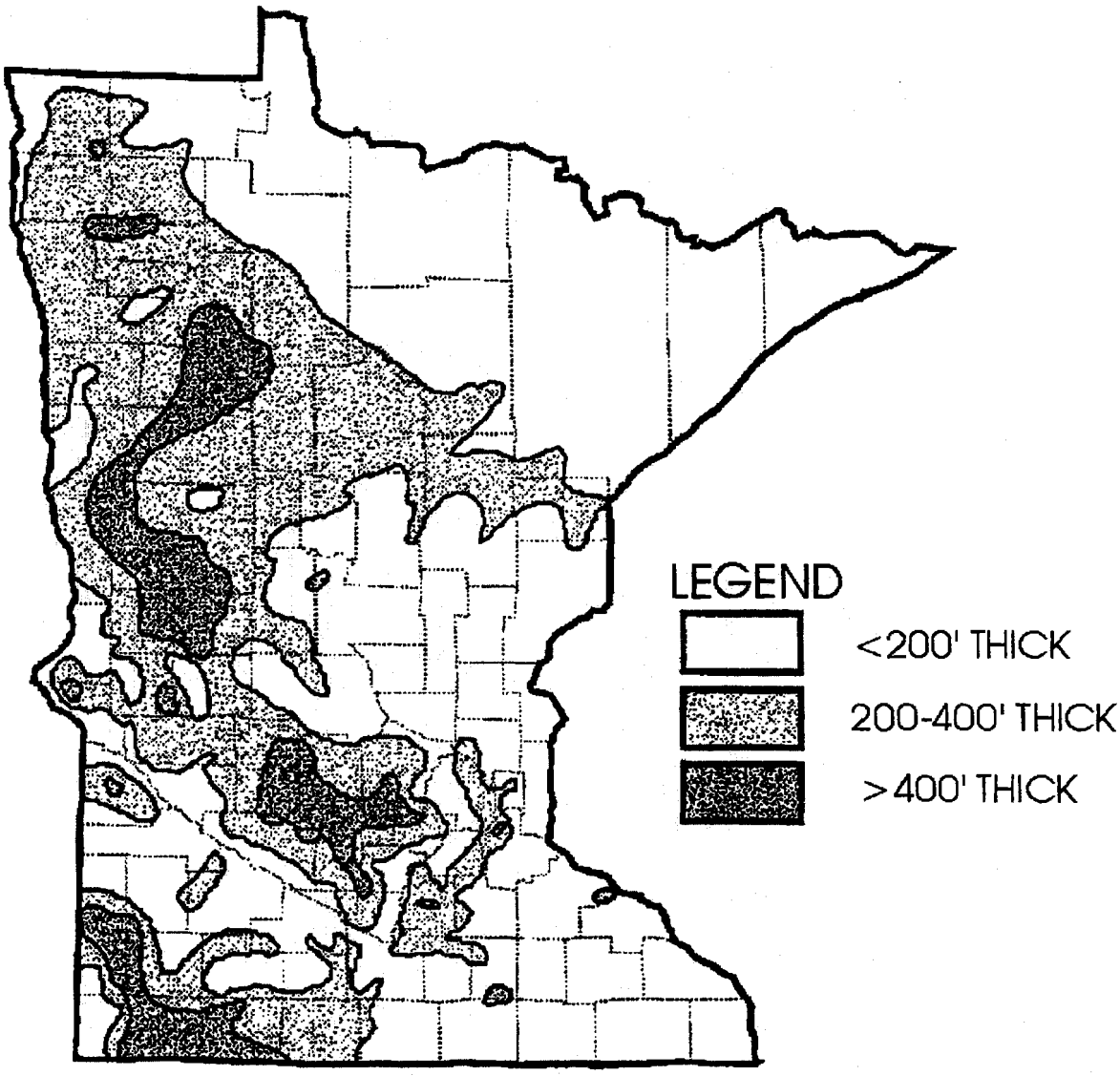
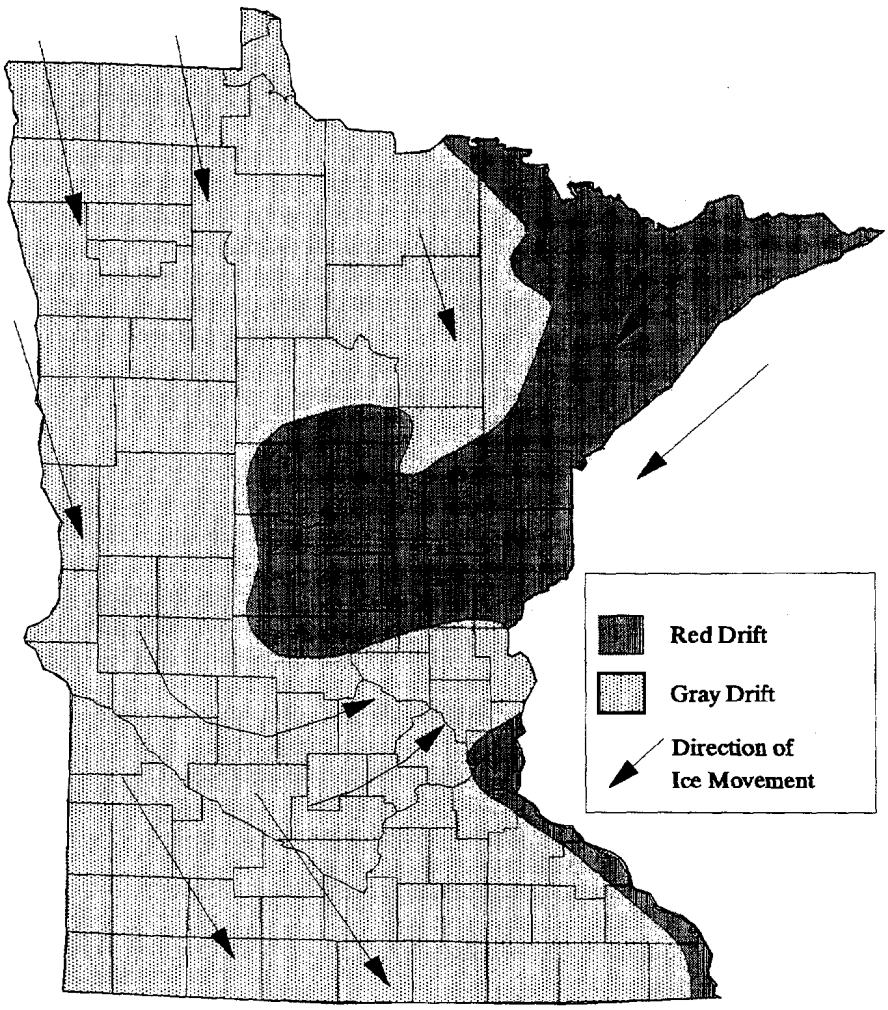


Figure 2-1.5
Generalized areas of "red drift" and "gray drift"



The distinction between these drifts is important in terms of gravel deposit1s. The red drift gravel is primarily igneous with minor amounts of sandstone making it ideal for concrete aggregate, whereas the gray drift has a very high percentage of carbonate, which is undesirable for most concrete products in highway use.

1. Unsorted Deposits.

   Till is unstratified and unsorted debris deposited directly from glacial ice without subsequent movement by wind or water. Till consists mainly of mechanically broken fragments of bedrock, as well as any soils or earlier glacial deposits that were overridden by the glacier. Till commonly displays a mixture of large rock fragments within a matrix of fine sand, silt and clay.

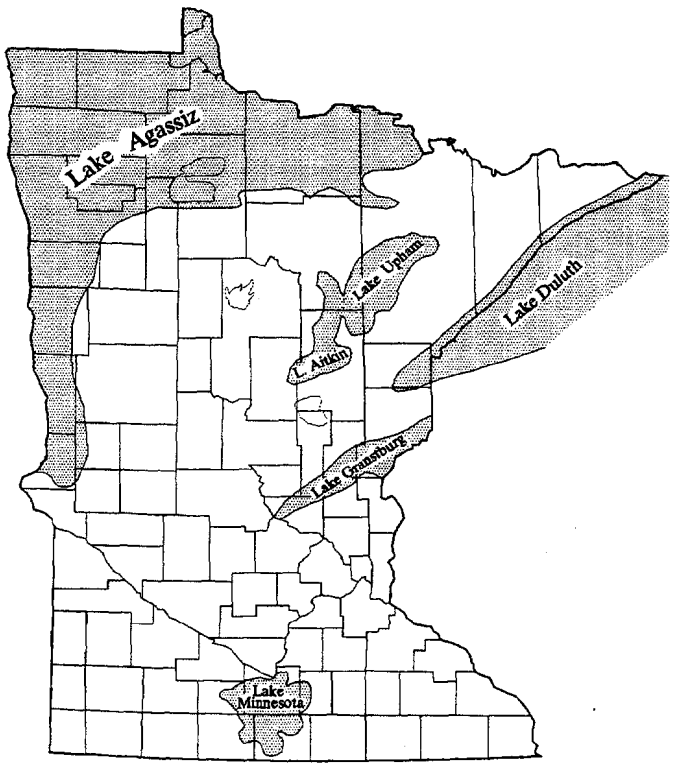
   Individual boulder-sized rock fragments deposited far from their bedrock sources are called **erratics**.

2. Sorted Deposits.

   Glaciofluvial deposits form when meltwater generated along the margins of melting glaciers picks up sediment and redeposits it as sorted beds of sand and gravel. These granular deposits are referred to as **outwash**. Outwash deposits are typically good sources of sand and gravel. The high permeability of outwash materials makes them good ground water sources. Proposed grades below the ground water table in outwash deposits will probably be difficult to dewater due to the high volume of water involved and the potential for rapid recharge.

   Sediments derived from glacial ice, deposited in fresh water lakes, are know as **glaciolacustrine**. As the glacial ice melts, meltwater runoff is dammed behind morainal deposits and other glacial features creating extensive lakes. During the Pleistocene, such glacial lakes were generally short lived; however, they had a lasting effect on the land surface. The largest and best known of these lakes was Glacial Lake Agassiz, which covered most of northwestern Minnesota and adjacent areas of North Dakota and Canada. Lake Agassiz clays are as thick as 150 feet along the northwestern border of Minnesota. Other glacial lakes formed at ice fronts elsewhere in Minnesota (Figure 2-1.6).

Figure 2-1-6
Former major glacial lakes and rivers of Minnesota (from Schwartz and Thiel, 1963)



Lake sediments are characteristically well-bedded silts and clays with occasional fine pebbles and boulders (dropstones) interspersed. These sediments are commonly varved, consisting of pairs of light/dark layers corresponding to summer/winter annual cycles. Most lake sediments pose problems in highway construction because they are often unstable in the backslopes, have poor drainage, lack strength, and are frost susceptible.

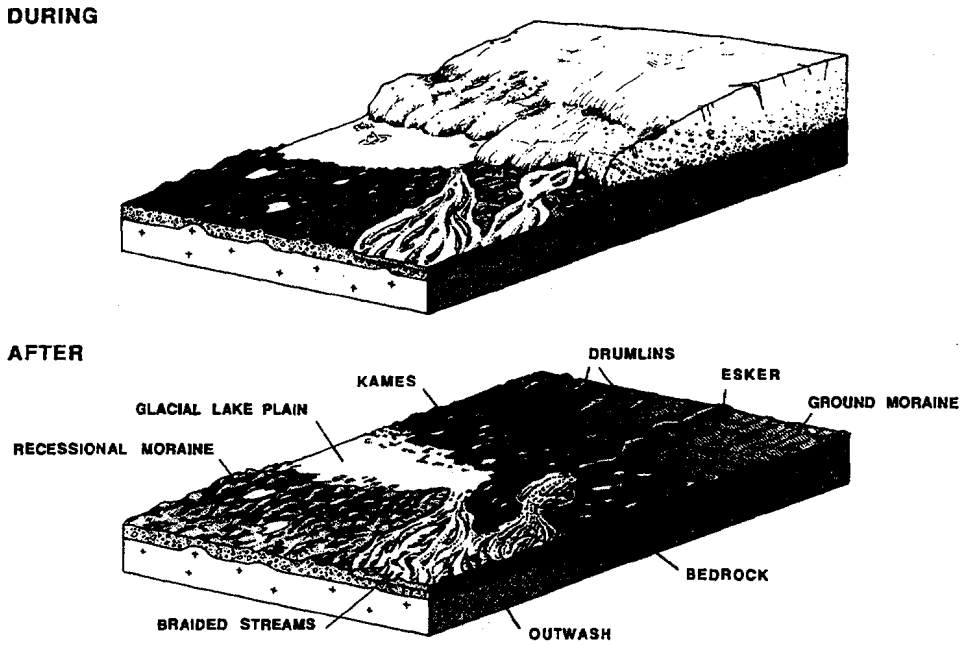
Wind-driven waves created beach ridges or strand lines at the edges of many of the glacial lakes. These old beaches remain visible in many places in the northwestern portion of the state, and mark former shorelines, often elevated above the present topography of lake sediments. Lake Agassiz beaches are visible as linear ridges, averaging 10 to 15 feet in height, 400 feet or more in width, and extending up to tens of miles. These beach deposits consist primarily of sand and/or gravel, with characteristic cross bedding. They are excellent sources of granular materials and are favored road and building sites. Highway 11 between Karlstad and Roseau is built primarily on a Campbell Beach line of Lake Agassiz. Some of the more prominent Lake Agassiz beaches (shorelines) are shown in Figure 2-1.3).

3. Topographic Features

Distinctive depositional landforms are constructed of sediments deposited by glacial ice and/or its meltwater (Figure 2-1.7). Soil and 1rock debris entrained within the glacial ice can be deposited during an advance or retreat of the glacier. Till dropped during the advance of a glacier or deposited by a retreating glacier as a thin blanket marked by low hills and swales is called **ground moraine** or a **till plain**.

The margins of glacial ice are especially active sites for the accumulation of debris, because there, melting is most intense. As a result this melting of the glacier and continued conveyance of the debris toward the margin, considerable volumes of sediments build up to form **terminal moraines**. These long, relatively high ridges of till mark the farthest advance of the ice sheet. **Recessional moraines** form at locations where the glacier pauses temporarily, but for a significant period of time, during its final retreat. The term **end moraine** is more general and includes both terminal and recessional moraines. In southwestern Minnesota, the prominent Bemis and Altamont end moraines stand above the otherwise flat prairie landscape. The outermost Bemis Moraine forms a major drainage divide and the wider (maximum of 12 miles), Altamont Moraine is dotted with numerous small lakes. Similar moraine topography can be found throughout most of the state, but of particular interest is the Alexandria Moraine complex. This complex (so designated because of its size and complicated origin) contains the thickest drift in the state. The lake dotted landscape has rugged relief and is heavily wooded, giving it great value as recreational land.

Figure 2-1.7
Block diagram illustrating common landforms associated with glaciers (from Ojakangas and Matsch, 1982)



When ice moves over an earlier sheet of glacial deposits, such as ground moraine, it often remolds the older material into elongate hills known as **drumlins**. The upstream end is blunt and steeper than the tail, with steeply sloping flanks. Drumlins are typically composed of till and have their long axis parallel to the direction of ice movement at that location. Drumlins typically occur in swarms, often referred to as "fields". Many drumlin fields are present in Minnesota, especially in the central region (Figure 2-1.3). Drumlins have similar soil associations and construction characteristics to the adjacent ground moraine.

When large blocks of ice are entombed in glacial drift (left by a receding glacier), they eventually melt and the overlying material collapses leaving large conical depressions known as **kettles**. Kettles with steeper banks and irregular outlines suggest that the parent ice block was subject to shallow or incomplete burial. These small depressions usually become filled with water and are referred to as **kettle lakes.**

Kames are formed when a cavity within or on stagnant ice fill with soil and rock debris. When the glacier melts, this collection of material is left behind as a mound or irregularly shaped hill of crudely stratified sand and gravel. Kames often occur in groups with kettles resulting in a hummocky topography with variable soil and water conditions.

Eskers originate from stream flow through tunnels within or below the glacial ice. As the melting glacial ice sheets thinned and hydrostatic pressure decreased, water velocity in the tunnel was no longer sufficient to transport all of the sediment supplied to the stream. Coarser sands and gravels were deposited with crude bedding, and finer materials were carried beyond the front of the glacier and deposited in lower energy lakes or streams. Subsequent melting of the ice sheet exposed these irregularly stratified and sorted sand and gravel deposits, which often extend for tens of miles. Due to their coarse-grained nature and length, eskers may provide an excellent source of granular materials, but may also present ground water problems when they are encountered in construction.

Meltwater streams carried sediment out from the toe of the glacial front in large alluvial fans called **outwash plains**. Large systems of braided streams often carried material in thin sinuous ribbons distances of many miles from the ice front. The resulting, sorted, granular deposits are mixed sands and gravels with crude bedding and many scour-and-fill structures. Many large areas of central Minnesota are considered outwash plains, such as the Anoka sand plain and the areas around Wadena and Detroit Lakes. Because of their great areal extent, outwash plains can be sources of abundant water. They are also areas of high susceptibility to surface contamination because they lack overlying confining (low permeability) layers.

2-1.01.03 WIND-BLOWN DEPOSITS

Loess is a powdery, well-sorted, angular silt and fine sand which was deposited during the last glaciation. Often just underlying the loess is a lag deposit of wind-faceted stones called **ventifacts**. While most common in southwestern and southeastern Minnesota, loess deposits are found in other portions of the state, occasionally between till layers. Loess material may stand at a near vertical angle with few stability problems. However, they are subject to rapid erosion by water or wind in the absence of a vegetative cover. Loess may also be subject to settlement problems; and bearing capacity may be affected in the presence of peculating water, due to the collapse of the soil structure.

Both ancient and modern sand **dunes** can be identified in Minnesota. They are recognized by clean, well-sorted, frosted, sand-sized grains and cross bedding.

2-1.01.04 DRIFTLESS AREA

The majority of Minnesota experienced periodic overriding by coalescing glacial ice lobes. However, one sliver of southeasternmost Minnesota has historically been thought to have totally escaped modification by glacial ice. This region is known as the "**Driftless Area**", and includes the majority of Houston, Fillmore, Winona, Olmstead, Wabasha, and Goodhue counties. More recent work suggests that this area actually was overridden by ice during early glaciations; however, the drift deposits from that early event have been subsequently eroded or severely weathered, making identification difficult. Scattered patches of highly

eroded old drift and occasional erratics have been reported in the driftless area, supporting this more recent conclusion.

While not extensively modified by ice, the driftless area has experienced enhanced topographic development as a result of glacial meltwaters and modern rivers and streams. Surface materials in this area consist predominantly of loess-covered residual soils on bedrock, as well as colluvium and bedrock outcrops.

2-1.02 HOLOCENE (RECENT) SURFACE FEATURES

Such features are the result of reworking/deposition of soil materials which have taken place since the last glaciation, hence - recent. A short discussion of these soil types and their general characteristics and uses follows.

2-1.02.01 ALLUVIAL

Any material deposited by water, whether modern rivers and streams or glacial meltwaters, is known as **alluvium**. Glacial alluvial deposits have been discussed extensively in the previous section. Post-glacial rivers and streams continued the work of the glaciers, further eroding and entrenching the topography of Minnesota. The courses of the Mississippi, Minnesota, Red, and St. Croix Rivers are marked by deposits of sands and gravels, and silts and clays, which lie within the flood plains of these modern rivers. **Terrace deposits** are similar in composition but lie above the current water level, on benches on the sides of the modern alluvial valleys.

Terraces are remnants of former river flood plains, which occur as benched elevations above the present-day flood plain. They may contain sands and gravels or finer grained materials, or they may be marked by scoured surfaces on exposed bedrock. The high degree of maturity of Minnesota's major rivers means that their paths are not straight, but very sinuous. These rivers curve back and forth over their flood plains, cutting and reworking alluvial sediments in broad tight curves known as **meanders**. Occasionally, these meanders become so tight that they are breached in times of high water, cutting off a curve in the river and producing an **abandoned meander**. These abandoned sections of the river gradually become more distanced from the main river course, forming curved **oxbow lakes** which gradually fill inward with sediment and organic materials.

2-1.02.02 LACUSTRINE

Fine-grained sediments deposited in fresh water lakes, which lakes may or may not still be in existence. Wave action in lakes carries the finer grained silt and clay sized particles in suspension towards deeper water. As the water calms, these particles settle out and accumulate in the lake bed to form what is known as lacustrine soil. (As mentioned earlier, many lacustrine sediments were formed in glacial lakes.) Old lake plains are frequently evidenced by a very flat topography.

2-1.02.03 PEAT AND MUCK

Peat is formed when organic material is deposited/forms in a predominantly cool oxygen-deficient environment. These conditions lead to preservation of the plant matter, with leaf and stem materials often remaining identifiable. **Peat bogs** are dominated by sphagnum moss with stands of black spruce and tamarack, and contain highly acidic waters. **Fens** are dominated by grasses, sedges, and reeds, with waters rich in minerals and less acidic than that of bog waters. These organic sediments typically form as vegetation encroaches into and then totally fills shallow lakes.

Minnesota is one of the "richest" states in terms of peat deposits (Figure 2-1.8). Peat is found extensively in north-central Minnesota, in the counties of Roseau, Lake of the Woods, and Koochiching, as well as Aitkin and southwestern St. Louis. **Highly organic** soils contain well-decomposed organic matter with or without some plant fibers of different decomposed states; e.g., "highly organic" is mixed sands and silts, with little clay and 11 to 25 percent organic matter. **Peat** is considered to be greater than 25 percent organic matter in mixed clays and silts with minor sand. Organic materials are obviously ill-suited for road construction, due to their poor drainage, high potential for settlement, and variable organic and mineral content. Classification for peats and other organic soils is given in Table 2-1.1.

Muck is a term which has often been misused.

1) The term muck is correctly used as in **Muck Excavation**, Mn/DOT 2105.2A3. This definition is as follows:

   "Muck excavation shall consist of all saturated and unsaturated mixtures of soil and organic matter not suitable for foundtion material regardless of moisture content, that is removed from below the natural ground level of marshes, swamps and bogs over which embankments are to be constructed and the excavation of which is required to provide a stable foundation for embankments or to accelerate the subsidence of unstable material under embankment load."

   Because this use of the term is included in Mn/DOT Standard Specifications for Construction, this connotation is preferred over the more limited definition referenced in 2) below.

2) Historically, the term Muck has often mistakenly been used to identify organic material which is more highly decomposed and thus less fibrous than peat, and is thus not easily described by more definitive terminology or to describe organic deposits so completely disturbed by drilling that accurate classification is not possible. Both of these uses are not correct and should be discontinued.

Note: The person preparing the Design Recommendation Letter must determine which materials should be included in Muck (Swamp) Excavation quantities for the project. Those included soil types should be listed by the designer in the soils notes on the plan.

Figure 2-1.8
Distribution of major peat deposits in Minnesota (after Ojakangas and Matsch, 1982).

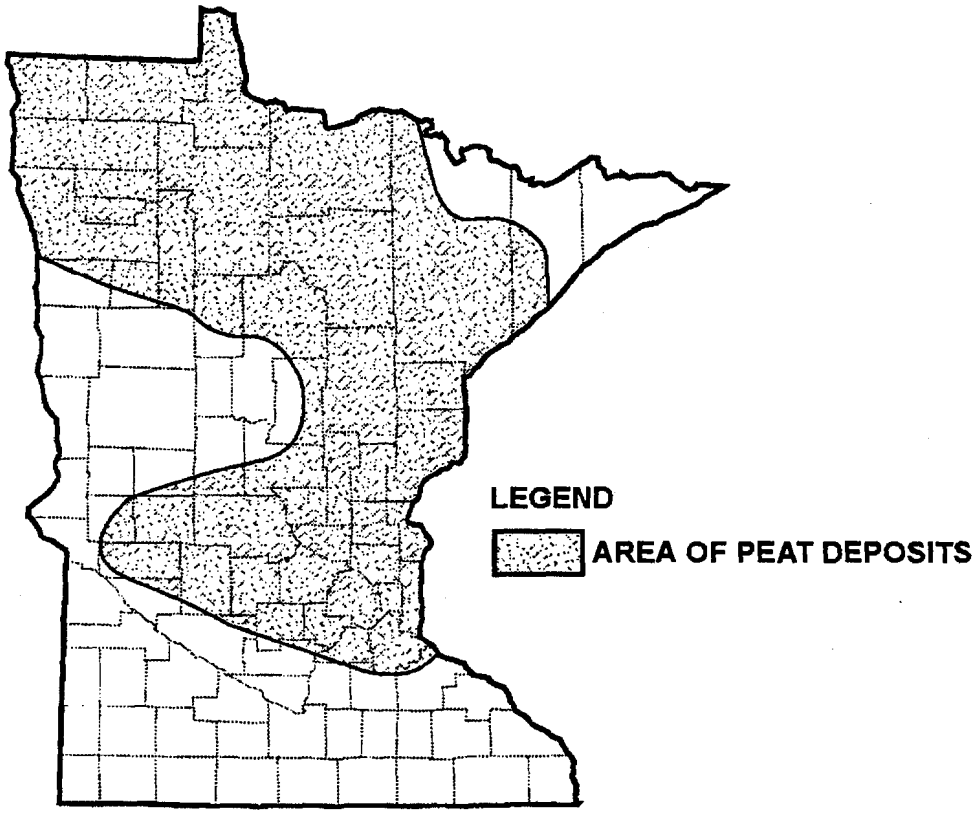


Table 2-1.1

Classification of organic soils

Classification	Organic Content by Weight (%)
non-organic	<2
slightly organic *	2 - 5
organic*	2 - 10
highly organic*	11 - 25
Peat (woody, fibrous, decomposed, etc.)	>25

*Insert specific soil type, e.g. slightly organic Silt Loam, highly organic Loam, etc.

Note: 1) The term non-organic should not be used in normal soil descriptions. (If no organic modifier is used, it is assumed that the soil is non-organic.) The term is included in the table for information only to complete the classification.

Note: 2) The term mineral soil includes those soils with 5% or less organic materials.

Peat (more than 25 percent organic content), can be further subdivided as follows:

- **Partially decomposed** peat is a short-fibered organic soil that may be fairly well decomposed and may contain mineral soil. Most of the fibers are less than approximately one-eighth inch in length.

- **Spongy** peat is a well-decomposed organic soil that has been subjected to certain consolidation conditions that cause it to appear and feel spongy. It varies in its mineral soil content, and there is little or no fiber content visible.

- **Well-decomposed** peat is an organic soil whose organic content has been subjected to a thorough decay process in which most fibers are invisible to the naked eye and which varies in its mineral content.

- **Semi-fibrous** peat is an organic soil whose plant fibers range from approximately one-eighth to one inch in length and are partially decomposed. These fibers may be mixed with some fairly well-decomposed organic matter and have a varied mineral soil content.

- **Fibrous** peat is an organic soil whose plant fibers are mostly one inch or more in length and are partially decomposed. These fibers may be mixed with some fairly well-decomposed organic matter and a small amount of mineral soil. The term "woody" is sometimes used for very coarse, minimally organic deposits.

2-1.02.04 MARL

Carbonate-rich, light gray to almost white silts and clays formed by precipitation of calcite at the bottom of lakes or swamps are known as **marls**. The carbonate generally precipitates directly out of the ground water but may also be deposited by carbonate-fixing organisms. An aggregate of carbonate shell material may also be called a marl. Due to their high carbonate content, marls will effervesce in dilute hydrochloric acid, differentiating them from the fine light gray sands also commonly found in swamp bottoms. Marls are unsuitable for road construction due to difficulty in compaction and instability in the presence of water.

2-1.02.05 COLLUVIUM

Loose, unsorted, rock fragments and soil materials produced by gravity or mass wasting are called **colluvium**. Landslides, mud slides, and talus are all colluvial deposits. These heterogeneous deposits are generally identifiable in the field and typically lie in a slump at the base of a hill or rock outcrop. The presence of such material usually indicates an unstable area subject to debris flow, slides, slumps, or down-slope creep. Highly variable soil conditions should be expected, and additional soil borings may be appropriate. Variable subsurface conditions or possible boulders may give misleading information as to soil type and depth to bedrock. The presence of colluvial material suggests an area of possible

slope instability and should be identified to prevent or control additional movement during or after construction.

2-1.02.06 RESIDUAL SOIL

The residual products of the weathering of Precambrian igneous and metamorphic rocks are common in southwestern and east-central Minnesota. The principal products of this in-place weathering are clay minerals, such as kaolinite and chlorite, that are generally dense (high Standard Penetration Test blow counts - up to 50/0.3 feet), and often contain remnants of the preexisting rock structure. Exposures of these weathering remnants are mainly confined to the Minnesota River valley west of Mankato, and in the St. Cloud area, and can be as thick as 100 feet. The clays are often brightly colored, with hues of green, red, brown, and white.

A residuum was also developed on the Ordivician carbonate rocks in Goodhue, Wabasha, Fillmore, Olmsted, and Mower Counties in southeastern Minnesota. The ferruginous clays of this deposit make up the Iron Hill Member of the Cretaceous Windrow Formation. These clays are typically gray to white but often contain brightly colored layers of reds and yellows associated with limonite deposition. In Fillmore, Olmsted, and Mower Counties, the weathering products contained concentrated iron ore (primarily goethite with minor amounts of hematite), which was commercially mined between 1942 and 1968.

Figure 2-1.9 shows the distribution of various Pleistocene and Holocene (Quaternary) deposits across the state of Minnesota.

Figure 2-1.9. Geologic Map of Minnesota, Quaternary Geology (from Goebel and Walton, 1979).

FOLLOWING PAGE

d and plant growth, and by glacial
edrock, which are common only in

from the base of an ice lobe forms a smooth
et called a ground moraine. Such till is stiff
lds little ground water.

at ice margins or from stagnating masses of
egular pitted to hilly topography with many
These landforms are called end moraines,
and stagnation moraines. These deposits
of sand, gravel and boulders with some local

TILL—Smooth to undulating moraine (pms)
moraine (pmh). The Des Moines lobe is the
lobe. It advanced through the Red River
ublobes extended eastward into the St. Louis
theastward across Minneapolis and St. Paul,
from earlier lobes. Des Moines lobe till is
It is mainly composed of gray (olive-brown
areous silt and clay, with lesser amounts of
ale and limestone are diagnostic.

TILL—Smooth to undulating moraine (pss)
oraine (psh). Ice of the Superior lobe moved
Superior basin in several pulses, spreading
e Mille Lacs area and southward across
Paul area. It interacted with the partly
tainy lobe along the Laurentian Divide.
generally reddish-brown, sandy to stony, and
contains abundant fragments of volcanic,
nd metamorphic rocks, red sandstone and
re it incorporates earlier lake deposits, it is
ey.

—Smooth to undulating moraine (prs) and
ne (prh). The Rainy lobe moved southward
g a broad front from Lake of the Woods
rior, where it met ice from the Lake Superior
entian divide and moved southwestward. It
cinity of Little Falls overriding drift and
ice remaining from the earlier Wadena lobe.
be drift area was later overridden by the St.
Des Moines lobe. Rainy lobe till is grayish
rown where oxidized), non-calcareous and
th abundant fragments of granitic, meta-
one volcanic rocks.

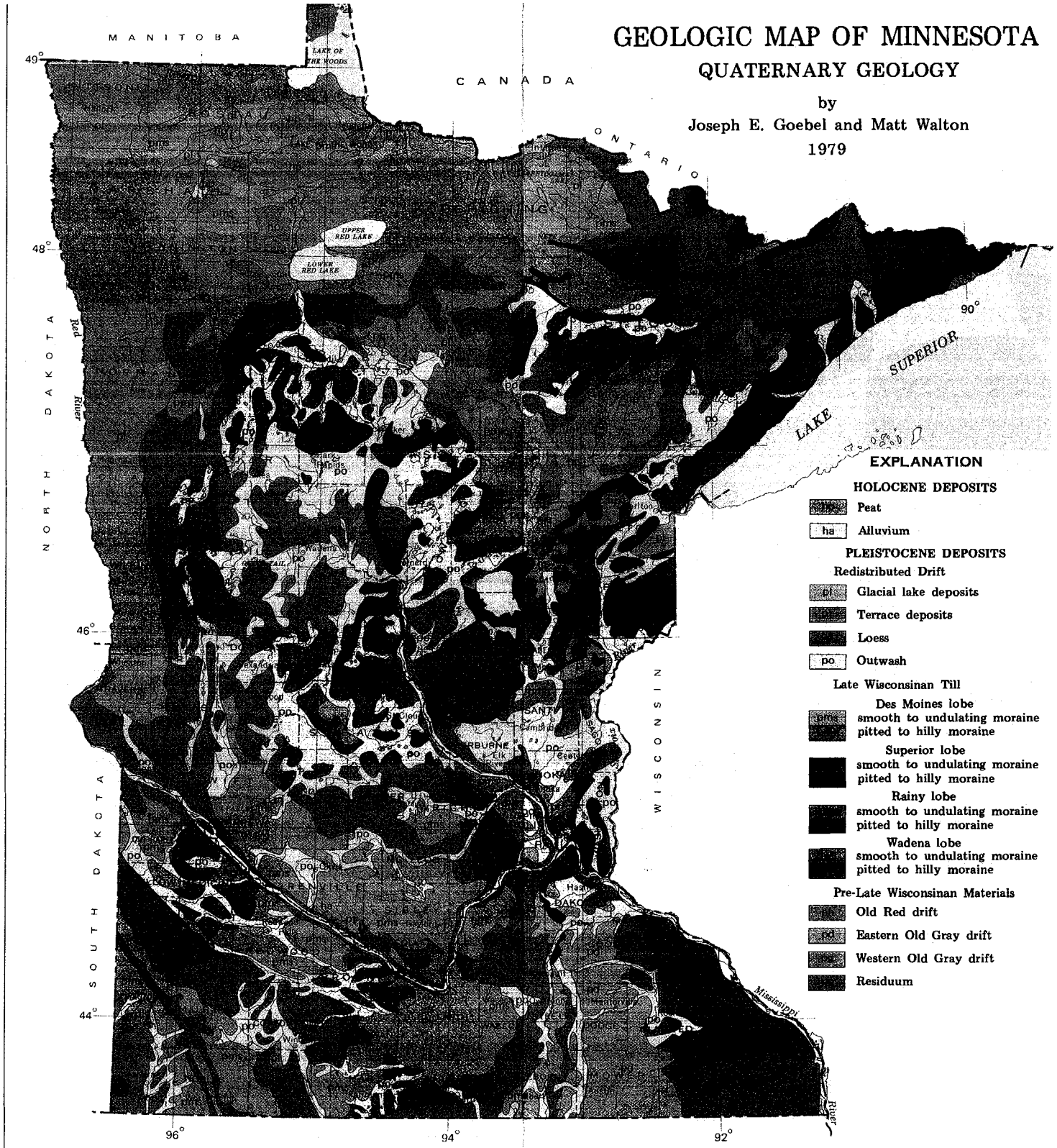
ILL—Smooth to undulating moraine (pws)
moraine (pwh). The Wadena lobe was the
Wisconsinan glacial lobes. A large remnant of
survives in northwest-central Minnesota in an
erridden by any of the three later lobes. A
ndicates movement of ice from the north or
. Wadena lobe till is gray (yellowish brown
l calcareous with fragments of igneous and
some limestone and little or no shale.

Late Wisconsinan Materials

r another, prior to the Late Wisconsinan, all
have been covered by glaciers. Evidence is
Late Wisconsinan drift except in the south-
astern corners of the state where there are
ed and stream-dissected drift that are older
n and could be Illinoisan or Kansan in age.

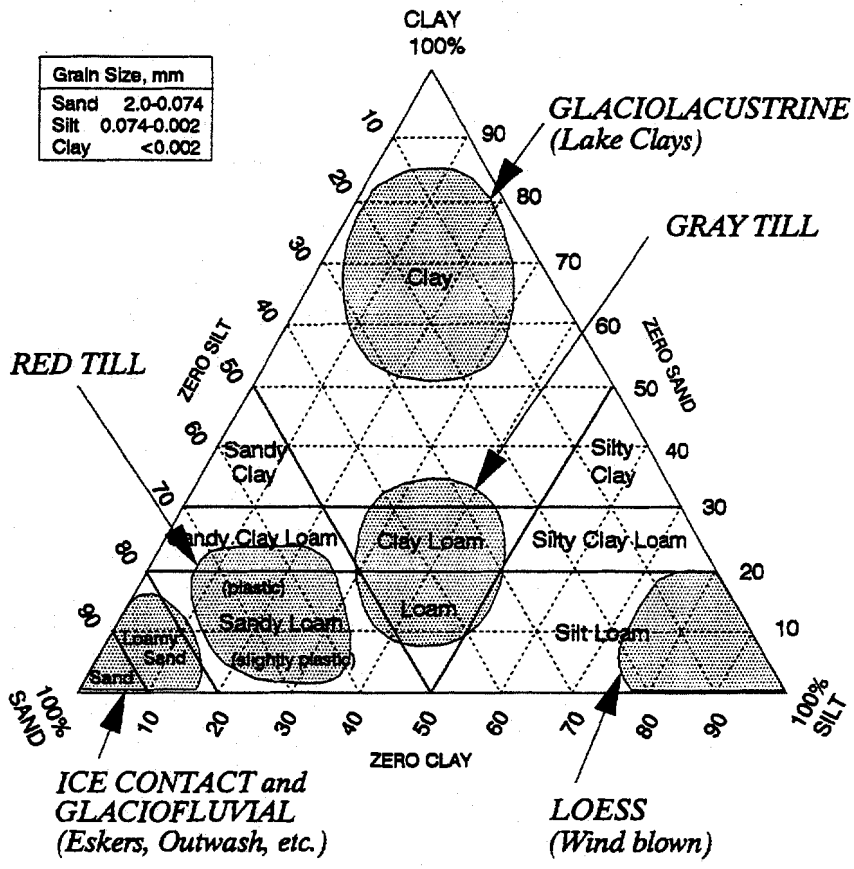
Moderate to dusky-brown till and outwash
akota and southern Washington Counties.
o, felsite and red sandstone are notable.
w a distinct weathered profile overlain by

RAY DRIFT—Moderate yellowish-brown
and outwash. It contains fragments of
orphic rocks, limestone and sandstone, but
ears to underlie Old Red Drift in southern



It is common for the above Quaternary deposits (soils) to fall into specific areas of the triangular textural classification chart (Section 3-2.02.01), as shown in Figure 2-1.10. Note that this figure is illustrative and intended to show general relationships only.

Figure 2-1.10
Generalized relationship of typical glacial materials and their textural classifications.



2-2.0 SOIL PROFILE AND HORIZONS

The soil profile is the result of weathering of the original parent material, which may be unconsolidated materials deposited by glacial, wind, or fluvial processes, or bedrock. The soil profile is divided into **horizons** based on the degree of chemical and physical weathering of the parent material. At any one location, these horizons would always occur, from the surface downward, in the order described below; however, not all of these horizons may be present at any one location.

2-2.01 O HORIZON

The O Horizon is not really a "soil" at all, but merely undecomposed organic debris and black humus (a relatively stable residuum of decomposed organic matter). Naturally, this unit is present only in well-vegetated areas.

2-2.02 A HORIZON

The A Horizon, the uppermost portion of the soil profile, is a mixture of mineral material and partially degraded organic debris. Soils in this horizon crumble easily due to their lack of consolidation. Usually, water percolating through this horizon winnows out the finer silts and clays leaving a sandier lighter textured material. Dark colors are common, due to a high organic content. The mineral assemblage is quite variable.

These materials are generally undesirable for engineering purposes, due to their high degree of compressibility, variability, and high organic content. An increased organic content often makes both the O and A horizon soils suitable for use as topsoil.

2-2.03 B HORIZON

The B Horizon is often referred to as the **zone of accumulation**. Here, materials weathered out of the A horizon - such as clay, and iron and aluminum minerals - are deposited. This results in a fairly uniform, lighter colored, denser soil which may have a blocky structure and may be more brownish or reddish in color than its overlying counterpart.

Throughout much of the state of Minnesota, the combined A and B horizons are approximately two feet thick.

2-2.04 C HORIZON

The C horizon represents the parent material of the soil profile - either glacial drift or weathered parent rock (saprolite). C horizon materials are generally lighter in color than both the A and B horizons and remain in essentially the same form as when originally deposited. This material retains its original structure but is leached of more mobile elements, such as calcium or magnesium. Where the A and B soil horizons are thin, the C horizon may be encountered during exploration or construction.

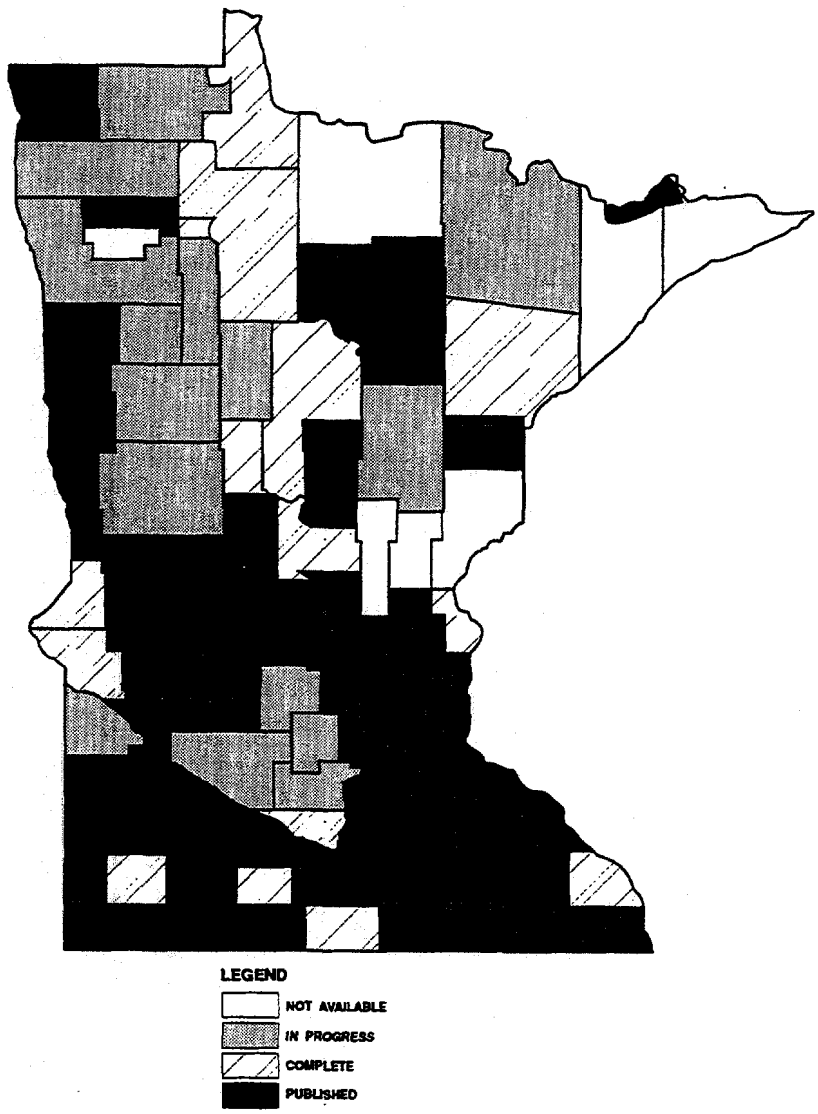
Ground water levels will often influence the color of soil materials, especially gray drift, but to a lesser degree also red drift. Above the water table, iron minerals oxidize and even gray soils take on a brown to rusty color. Below the water table, unoxidized colors remain. Within the historic zone of water table fluctuation, a mottling of colors is often evident.

2-2.05 D HORIZON

The D Horizon may also be referred to as the R Horizon. It indicates consolidated unweathered bedrock which is not the parent material for the soil, but which may influence the overlying soils by controlling drainage or surface morphology. For the purpose of a geotechnical survey, the D horizon is of interest only when there are outcrops or when borings indicate that bedrock will be encountered within the roadway section.

Detailed soil survey information is currently available for 53 counties, as shown in Figure 2-2.1; however, some surveys are quite old and may be out of print. Some of the reports include detailed engineering data.

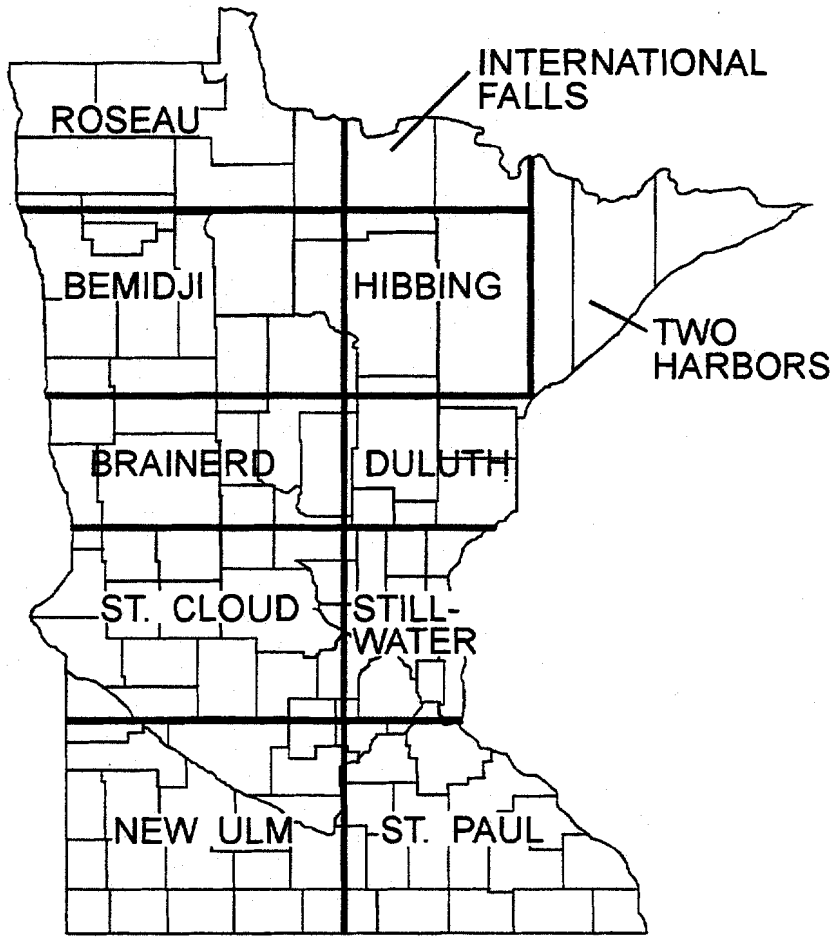
Figure 2-2.1
Status of soil surveys
(after U.S. Department of Agriculture, Soil Conservation Service, July 1990).



Other soil information is available at a larger scale (less detail) in the Minnesota Soil Atlas. The atlas consists of 11 sheets (Figure 2-2.2) covering the entire state. The soil atlas sheets consist of one to 250,000, 50-foot contour interval, topographic, colored maps, overlain with soil landscape units and geomorphic areas. A companion booklet discusses the soils in detail and offers additional information, such as climate and landform development. A section for the

engineer may provide useful hints for locating sources of sand and gravel, limestone aggregate, peat, shallow bedrock, etc. Additional soils and landform information can be found in the Hydrologic Atlases discussed in Section 2-4.03.02.

Figure 2-2.2
Index map showing areas covered by soil atlas sheets
(from Department of Soil Science, University of Minnesota).



2-3.0 BEDROCK

The majority of bedrock exposures in Minnesota are found in the extreme northeastern and southeastern portions of the state. While limited bedrock is actually exposed in Minnesota, the state is rich in the variety of rocks which exist beneath the glacial cover. There are extensive units of sandstone, shale, and limestone, especially in the southeastern quarter of the state. The remainder of the state is notable for granite, basalt, gabbro, quartzite, and metamorphosed igneous and sedimentary (greenstone belts) bedrock (Figure 2-3.1).

Maps of the bedrock geology are currently available from the Minnesota Geologic Survey in the form of Geologic Atlas Sheets (Figure 2-3.2) for St. Paul, New Ulm, Hibbing, International Falls, Roseau, East-Central, and Two Harbors. It should be noted that these atlas sheets do not directly correspond to the sheets published in the Minnesota Soils Atlas. A full-sized bedrock geology map is also available from the Minnesota Geologic Survey.

A generalized geologic column for sedimentary bedrock formations and their member units occurring within the state is presented in Table 2-3.1. Included is a brief description of the bedrock unit, thickness, geologic time period, and approximate age. This column, when used in conjunction with the bedrock geology map (Figure 2-3.1), should yield pertinent bedrock information in most areas of investigation.

Additional, more detailed geologic information is available for a few counties: Scott (1982), Winona (1984), Olmsted (1988), Hennepin (1989), Washington (1990), and Dakota (1990). These county atlas sheets contain detailed information on various subjects, such as surficial and bedrock geology, soil and bedrock aquifers, depth to bedrock, and sensitivity of ground water sources to pollution. The county atlas sheets provide the most detailed geologic information available.

Figure 2-3.1
Geologic Map of Minnesota, Bedrock Geology (from Morey, 1993).

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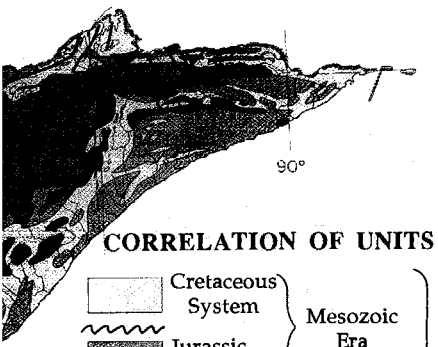
ROCK GEOLOGY

1993

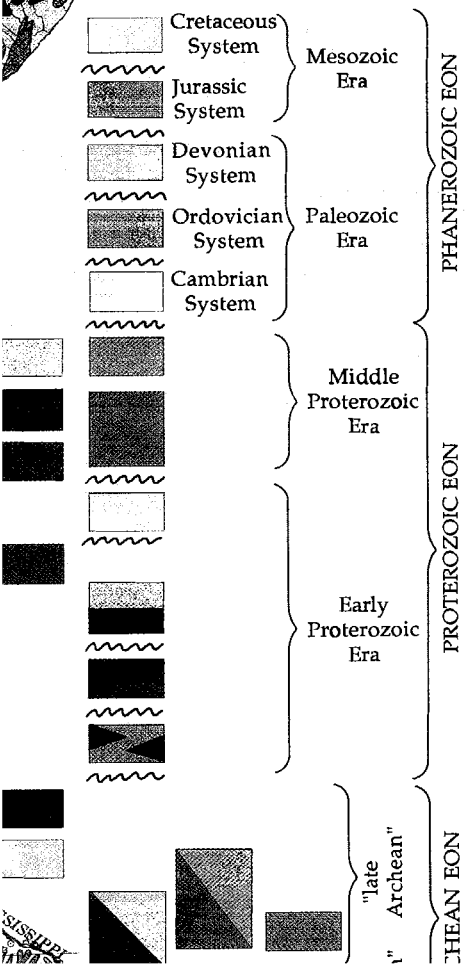
SCALE 1:3,000,000

0 — 50 MILES

50 — 100 KILOMETERS



CORRELATION OF UNITS



MESOZOIC ROCKS

CRETACEOUS ROCKS, UNDIVIDED — Dominantly marine shale, some sandstone, and lesser amounts of limestone and their non-marine sandy and shaly equivalents to the east and northeast.

JURASSIC ROCKS, UNDIVIDED — Reddish-brown dolomitic shale; minor cherty dolomite and gypsum.

PALEOZOIC ROCKS

DEVONIAN ROCKS, UNDIVIDED — Limestone and dolomite.

ORDOVICIAN ROCKS, UNDIVIDED — Limestone, dolomite, and some sandstone and shale.

CAMBRIAN ROCKS, UNDIVIDED — Quartzose and glauconitic sandstone and siltstone; lesser amounts of carbonate rocks.

MIDDLE PROTEROZOIC ROCKS

KEWEENAWAN SUPERGROUP

Red to buff shale and feldspathic to quartzose sandstone. Includes Fond du Lac Formation and Hinckley Sandstone.

Basalt and related volcanic and volcaniclastic rocks of the North Shore and Chengwatana Volcanic Groups.

Subvolcanic gabbroic and related hypabyssal rocks. Includes the Beaver Bay Complex.

Anorthositic, troctolitic, and gabbroic rocks, undivided, of the Duluth Complex.

Granite and granophyric rocks of the Duluth and Beaver Bay Complexes.

Diabasic gabbro in dikes and sills. Includes the Logan Intrusions.

EARLY PROTEROZOIC ROCKS

Sioux Quartzite and related claystone (catlinite).

Granitic rocks of the Penokean orogen, undivided; also includes a variety of older country rocks and coeval cover rocks.

ANIMIKIE GROUP

Shale, siltstone, graywacke, and associated volcanic rocks of the Virginia, Thomson, and Rove Formations.

Iron-formation. Biwabik and Gunflint Iron Formations and subjacent quartzitic and conglomeratic units.

NORTH RANGE GROUP

Slate, metagraywacke, and associated metavolcanic rocks of the Rabbit Lake Formation and quartz-rich rocks of the Mahnomen Formation.

Iron-formation. Trommald Formation and several other unnamed units of iron-rich strata.

MILLE LACS GROUP

Metamorphosed quartzose sedimentary rocks; intercalated with carbonaceous strata and minor carbonate rocks, mafic and intermediate volcanic rocks, and iron-formation.

Iron-formation. Glen Township Formation and several other unnamed units of iron-rich strata.

LATE ARCHEAN ROCKS

Monzonite, syenite, and monzodiorite, together with mafic to ultramafic intrusions; generally late-tectonic in age.

Granodiorite, granite, and smaller bodies of tonalite and monzonite; generally syntectonic to pre-tectonic in age.

Granite-rich migmatite. Granitic gneiss, paragneiss, schist, and migmatite. Grades into granitoid rocks.

Schist-rich migmatite and paragneiss. Grades into metasedimentary rocks.

Metasedimentary rocks, including graywacke, slate, conglomerate, arenite, graphitic argillite, fine-grained felsic, volcanogenic, and reworked volcaniclastic rocks.

Mafic volcanic rocks. Dominantly basalt to andesite with rare interflow clastic rocks and hypabyssal bodies of mafic composition; also includes mixed sequences of mafic to felsic volcanic, volcaniclastic, and epiclastic rocks, and iron-formation.

Iron-formation. Soudan Iron Formation of the Vermilion district as well as many other units of uncertain stratigraphic position.

Various metasedimentary and volcanic rocks; intrusive bodies of granite, tonalite, and gabbro. Rocks lack magnetic signature because of regional low-grade alteration of oxides and ferromagnesian minerals.

MIDDLE ARCHEAN ROCKS

Migmatitic gneiss and amphibolite; includes units of younger Archean granite, as well as units of Early Proterozoic gneiss and granite.

Contact. Located principally by airborne magnetic surveys except in a very few places where exposed.

Chapter Three
MATERIALS CHARACTERIZATION

3-1.0 ROCK

Rocks are highly variable in composition and much practical experience must go into detailed and accurate geologic descriptions of rock materials. An accurate, detailed, geologic description can, in most cases, be made only by a qualified geologist. However, field personnel can produce a usable description of bedrock materials encountered in the field using a basic rock classification system. This will facilitate correlation between borings and outcrops and may provide insight into engineering properties and rock material behavior which is necessary for the design and construction of projects that encounter bedrock. In addition to describing the rock, the extent of its occurrence relative to the project must also be documented. To be useful to both engineers and contractors, geologic data and descriptions must be presented in a form useful to each.

Rock classification for geotechnical purposes consists of two basic assessments:

- Sample characteristics, consisting of classification of the intact rock specimen or core; as to its Formation and constituent minerals; and its texture, color, and degree of weathering, and

- Rock mass characteristics, consisting of classification of the inplace rock mass, which includes description of structural or lithological discontinuities, such as bedding, joints, faults, and formational contacts.

An important facet of rock classification is the determination of what constitutes rock, as opposed to extremely weathered material which approaches soil in its character and engineering properties. Geologists and geotechnical engineers determine the degree of weathering in rock and determine whether its properties have become "soil-like" and will thus be referred to as **residual soil**. The contact between residual soil and rock is labeled the "top of bedrock" and should be noted on boring logs, profiles, and cross sections.

3-1.01 VISUAL DESCRIPTIONS

For small projects, or during the early stages of large projects, rocks are initially classified by visual means.

3-1.01.01 COLOR

When describing color, only common colors, such as brown, black, gray, or white, etc., or simple prefixes representing density of the color, e.g., light or dark, and other colors to represent shading, e.g., reddish or grayish, should be used.

3-1.01.02 TEXTURE

Texture deals with the size, shape, surface characteristics, and arrangement of individual grains or crystals in a rock. In igneous rocks, a significant and readily identifiable texture is grain size, as given in Table 3-1.1 below.

Table 3-1.1 Grain size description for Igneous Rocks

Term	Size of Grains
Aphanitic	Individual crystals are not visible to the unaided eye.
Phaneritic	Individual crystals are readily visible.
Coarse grained	Crystals are larger than .2 inch
Medium grained	Crystals range from .04 to .2 inch
Fine grained	Crystals are visible, but smaller than .04 inch.

The same grain size descriptions can be applied to metamorphic rocks, although such descriptions are often difficult because the metamorphic grains have been deformed into flat or elongated grains. This arrangement of flat or elongated grains into parallel plains or bands is known texturally as **foliation.**

In sedimentary rocks, the two major textural groups are **clastic** and **nonclastic.** The clastic rocks are identified principally by their grain size as described in Table 3-1.3. Most nonclastic rocks are formed as chemical precipitates and have crystal grains that are interlocked. Phaneritic (coarse-grained) chemical sediments have somewhat the same appearance as igneous rocks with crystalline texture, and thus the same grain size descriptions are used. Aphanitic precipitates lack visible crystals and are described as **microcrystalline** or **cryptocrystalline**, depending on whether they have microscopic or submicroscopic grains, respectively. Microcrystalline rocks generally are visibly granular or have an earthy luster, whereas cryptocrystalline rocks have a porcelaneous luster.

3-1.01.03 VOIDS

Voids are open spaces in the subsurface rock that are generally due to the removal of rock materials by chemical dissolution or the action of running water. Voids can be related either to intact properties or to rock mass properties, depending on their size, as described in Table 3-1.2.

Table 3-1.2. Description of void sizes.

Term	Size of Voids
Pit	Barely seen with the naked eye, up to 0.25 in. (6 mm) in diameter.
Vug	0.25 in. (6 mm) to 2 in. (50 mm) in diameter.
Cavity	2 in. (50 mm) to 24 in. (600 mm) in diameter.
Cave	Larger than 24 in. (600 mm) in diameter.

3-1.01.04 LITHOLOGY

Rocks are divided into three general categories according to their origin; igneous, sedimentary and metamorphic.

Rocks are classified under three main categories, depending on their method of formation. Thus, igneous rocks are formed by the cooling of molten rock magma, sedimentary rocks by the breakdown through weathering of an existing rock mass, and metamorphic rocks by alteration of existing rocks under conditions of high temperature and/or pressure.

1. **Sedimentary.** Sedimentary rocks are classified on the basis of their depositional mode, grain size, mineralogy, mode and extent of lithification and relationship between grains. Sedimentary rocks are separated into two major categories, clastic or chemical, depending on their depositional mode. Table 3-1.3 gives the basic classification system for sedimentary rocks.

   a. **Clastic.** Clastic sedimentary rocks are composed of rounded or angular fragments of previous rocks, and are classified by their particle size, ranging from gravel to clay.

      (1) **Conglomerate and Breccia.** These are sedimentary rocks in which clastic sediments are predominantly gravel sized or larger. The term **conglomerate** is usually applied to rocks in which the coarse fragments are subangular or more completely rounded, whereas the term **breccia** is used if the grains are angular. The individual fragments that make up a conglomerate or breccia may be of any lithology, such as the Saganaga Tonalite (igneous) boulder conglomerate found in Knife Lake Group of northern Minnesota. Conglomerates are also found as basal layers within sedimentary stratigraphic units such as the St. Peter sandstone (sedimentary). It is also common to find both conglomerates and breccias within the Sioux Quartzite (metamorphic).

(2) **Sandstone.** Sandstones are composed predominantly of quartz grains, cemented with silica, carbonate, clays, or iron. They are typically described by their grain size, mineral constituents and degree of cementation (friability). A well sorted sandstone that contains more than 95% quartz grains is given the name **quartzose sandstone.** Other special terminologies for sandstones include **arkose** - rich in feldspar, or **graywacke** - a mixture of quartz, feldspar, and fragments of other rock in a fine-grained matrix.

Sandstones may make excellent aquifers, carrying large amounts of water which may cause drainage or erosional problems in deep excavations or cuts. Sandstones are also generally competent for loading of structural footings when confined, but are susceptible to erosion or scour when exposed to the elements.

(3) **Siltstone and shale.** This group of sedimentary rocks includes all those in which the grains are smaller than sand size (silt and clay). **Siltstone** is the term applied to fine grained rocks that are composed of 2/3 or greater percentage of silt, and lack the fissility of shale. In practical terms silt and clay sizes are too small to be visible to the unaided eye, but silts can be detected by their gritty nature between the teeth. In contrast, clay sizes behave as a paste between the teeth. The term **shale** is often used to describe the remaining rocks in this group, although technically, the term shale should be applied only to those rocks which show bedding-plane fissility or laminations. Table 3-1.3 shows the less often used terms of mudstone and claystone which can be applied to silt and clay-sized massive, nonfissile sedimentary rocks.

Siltstones are often white or light gray. The most common occurrence is the Blue Earth Siltstone bed, part of the Oneota Formation, found extensively along the Minnesota River Valley near St. Peter. It also occurs in Cretaceous rock found in the southern and western portion of the state.

Shales are typically platy and have thin zones or beds that are typically sandier or siltier, fossiliferous, or that contain sedimentary features. The Decorah Shale of southern Minnesota is a good example. It is exposed on eroded slopes in southern Minnesota where it occurs between the Platteville and Galena Limestones. The Decorah is a greenish gray, fossiliferous shale with occasional thin limestone beds scattered throughout.

Erosion, slides and expansion are potential problems in shales.

b. **Chemical.** Chemical sedimentary rocks include all those rocks that precipitate from solution, such as carbonates (limestone and dolostone) and chert.

(1) **Carbonates.** Calcium carbonate (calcite) is precipitated out of the water by chemical or biologic means, or it collects as a mass of shell material. Subsequent lithification and chemical alteration produces crystalline **limestone.** Chemical substitution of magnesium carbonate (dolomite) for calcite yields **dolostone** or dolomitic limestone. This circumstance usually obscures primary structures, such as bedding or fossils. Limestone will effervesce freely in dilute hydrochloric acid. Dolostone will react with acid only after powdering, yielding a good tool for differentiating limestones from dolomitic limestones and dolostones.

Carbonates situated near the water table - either present day or ancient - may have undergone dissolution by slightly acidic ground water, creating enlarged joints, cavities, or caves. There may be more than one horizon of caving or dissolution due to changes in ground water level through time. Small surface depressions, known as **sinkholes**, are the result of loss of soil into these solution cavities, or the roof collapse of near-surface caves. The topography produced by the progressive dissolution and collapse is known as **karst**. Areas of karst topography or known caves may possess marginal foundation conditions due to past collapses and continued dissolution of the bedrock. The majority of Minnesota's karst topography is located in the southeastern counties of Fillmore, Winona, and Olmsted.

(2) **Chert. Chert** is a microcrystalline rock, composed of minute quartz crystals with submicroscopic pores, having a conchoidal to splintery fracture. Chert is hard, typically white to light gray, and weathers to a soft, dull white, chalky crust, often around an unchanged chert core. Appreciable quantities of chert may occur as nodules in limestones or shales and may cause difficulty in drilling shafts or piers.

Table 3-1.3 Classification and Identification of Sedimentary Rocks (Modified from "Petrology of Sedimentary Rocks", R.L. Folk, 1980.)

TABLE A

CLASTIC SEDIMENTARY ROCKS				
Composition		Comments	Rock Classification	
Gravel > .08 in.		Rounded grains	CONGLOMERATE	
		Angular grains	BRECCIA	
Sand .002 - .08 in. V. Coarse .04-.08 in. Coarse .02-.04 in. Medium .01-.02 in. Fine .005-.01 in. V. Fine .002-.005 in.		Mostly quartz grains	Quartz Sandstone	SANDSTONE
		Mostly feldspar grains	Arkose	
		Poorly sorted mixture of quartz, feldspar & rock fragments in fine matrix	Graywacke	
Silt .00015-.002 in.	> 2/3 Silt	Nonfissile (blocky)	SILTSTONE	
		Fissile (splits easily along closely spaced planes)	Silt-Shale	SHALE
	Subequal Silt & Clay	Nonfissile	Mudstone	
Clay <.00015 in.		Fissile	Mud-Shale	
	> 2/3 Clay	Nonfissile	Claystone	
		Fissile	Clay-Shale	

TABLE B

CHEMICAL SEDIMENTARY ROCKS			
Composition	Comments	Rock Classification	
Mainly Calcite (Calcium Carbonate) $CaC0_3$	Crystals formed as precipitates	LIMESTONE	CARBONATE
	Spherical grains with concentric laminations	Oolitic LIMESTONE	
	Abundant fossils	Fossiliferous LIMESTONE	
Mainly Dolomite $CaMg(CO_3)_2$	Commonly altered from limestone	DOLOSTONE	
Quartz - SiO_2	Layers, lenses & nodules	CHERT	
Halite - NaCL	crystals formed as precipitates	ROCK SALT	
Gypsum - $CaSO_4{*}2H_2O$	crystals formed as precipitates	GYPSUM	

2. **Igneous.** Igneous rocks are classified based on their grain size, which is a result of the cooling rate of the parent magma, and mineral composition.

Extrusive rocks cooled quickly when molten material was brought to the surface during volcanic activity, resulting in a fine-grained (aphanitic) texture, often too fine to see without magnification. Certain features are common in extrusive igneous rocks. **Vesicles** are small openings in the rock caused by bubbles of gas escaping from the lava during cooling. Later on, calcite, silica, zeolites, or other minerals may have been deposited inside vesicles by fluids moving through the rock. These filled cavities are known as amygdules. (Agates are formed in this way.) **Intrusive** rocks cooled and crystallized from magma remaining deep within the earth, producing larger grains, typically easily seen with the unaided eye.

Table 3-1.4, in which variations in composition of the magma are shown horizontally (mafic, felsic, and intermediate) and textural variations are shown vertically, provides an effective system for classifying and naming igneous rocks.

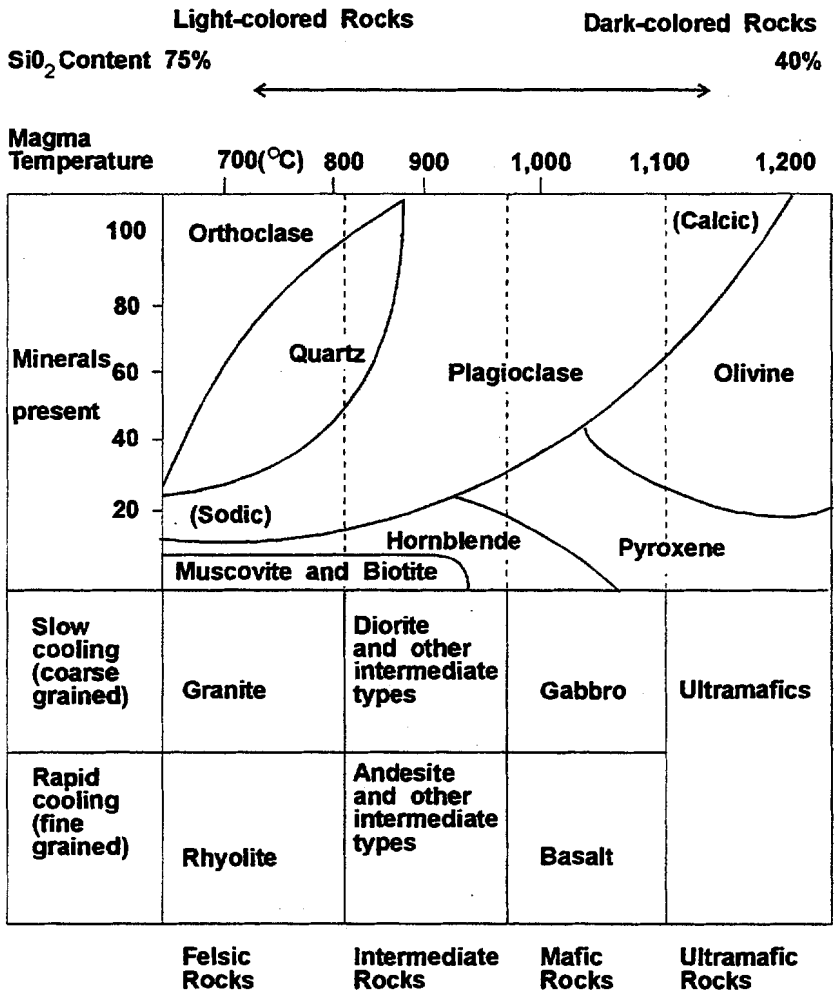
a. **Mafic.** Magmas that are rich in iron, magnesium, and calcium are referred to as **mafic** and produce greater quantities of dark colored minerals, such as olivine, pyroxene, hornblende, and calcic plagioclase, resulting in "dark" colored rocks. Colors considered dark are dark gray, green, and black. Mafic rocks are classified as **basalt** if they are fine grained and **gabbro** if they are coarse grained.

b. **Felsic.** Magmas that are rich in silica and aluminum are referred to as **felsic** and tend to produce more quartz, potassium feldspar, and sodic plagioclase, which generally form "light" colored rocks. Colors considered light are white, light gray, pink, and red. An abundance of quartz in the rock indicates a felsic composition. Felsic rocks are classified as **rhyolite** if they are fine grained and **granite** if they are coarse grained.

c. **Intermediate.** Intermediate rocks, as the name suggests are intermediate between mafic and felsic rocks. They often contain both dark and light minerals. A coarse-grained intermediate rock, such as **diorite**, will have a "salt and pepper" appearance due to the mixture of light and dark minerals. **Andesite**, a fine-grained intermediate rock, is typically green, dark gray, or red, but is difficult to distinguish from either basalt or rhyolite because its color can be similar to either.

The greatest exposure of igneous rocks in Minnesota is along the north shore of Lake Superior, where basalt and rhyolite lava flows, as well as several large intrusive bodies of gabbro, are exposed. Granite and metamorphosed granites outcrop in the St. Cloud area and along the Minnesota River Valley between Ortonville and New Ulm.

Table 3-1.4. Classification and identification of igneous rocks (from Ojakangas and Matsch, 1982).



3. **Metamorphic.** Metamorphic rocks result when rock materials are altered or deformed by heat and pressure. Classification is based on composition and texture. Frequently, metamorphic rocks are not referred to by their metamorphic rock equivalent terms, but are instead described as being metasedimentary, metabasaltic, metavolcanic, etc. This general use of the prefix "meta" is commonly, although not always, used when the original rock has undergone lower grades of metamorphism, with the parent rock being easily identified. Occasionally, primary structure, such as original bedding or ripple marks, may be preserved, albeit in a distorted form.

In Table 3-1.5, metamorphic rocks are separated into the categories of foliated and non-foliated or massive.

a. **Foliated.** Foliation is a planar element in metamorphic rocks that may be expressed as closely spaced fractures (slatelike cleavage), parallel arrangement of platelike minerals (schistosity), or by alternating layers of differing mineralogic composition (gneissic layering). Common foliated rocks, in order of decreasing degree of metamorphism, are gneiss, schist, and slate.

(1) **Gneiss.** Gneiss is a coarse- to medium-grained banded rock composed predominantly of quartz, feldspar, mica, and amphiboles. Dark and light colored bands result when minerals segregate during metamorphism. Commonly quarried as building stone, gneisses can be found in the Minnesota River Valley, upstream from New Ulm.

(2) **Schist.** Schist is a coarse- to medium-grained rock, in which the foliation is due to parallel arrangement of platey minerals, such as mica, chlorite, and talc. It is usually classified on the basis of the constituent minerals present, which indicate the degree or intensity of metamorphism to which the rock has been subjected, e.g., chlorite, biotite-muscovite, garnet, kyanite, or staurolite schist.

(3) **Slate.** Slate is an extremely fine-grained rock derived principally from shale. The particles have a very strong alignment, which results in a well-developed platelike cleavage. This cleavage, known as fissility, causes the slate to split easily along closely spaced parallel planes. The presence of adversely oriented cleavage may make excavation of cuts difficult and result in potentially unstable slopes.

(4) **Other.** Other metamorphic rocks not described in Table 3-1.5 include phyllite and argillite. **Phyllite** is a fine-grained rock, intermediate in grade between slate and schist. Minute crystals of mica or chlorite impart a silky sheen to its cleavage surfaces. **Argillite** is commonly used to describe a rock derived from shale, that is intermediate between shale and slate, but lacks both the fissility of a shale and the cleavage of a slate.

b. **Non-foliated**. Non-foliated or massive metamorphic rocks do not possess the planar structures resulting from the flattening or segregation of the constituent grains. Their grains are typically fused and are often difficult to recognize as individual grains. Common non-foliated rocks are quartzite and marble.

(1) **Quartzite.** Quartzite is a metamorphosed sandstone, and is principally quartz. It results from a recrystallization of sand or chert grains, fusing them together; or, less commonly, from the complete secondary cementation of an unmetamorphosed sandstone. In either case, the rock is very hard and breaks across or through the grains rather than around them. The most common occurrence in Minnesota is the Sioux Quartzite, quarried near Courtland and Jeffers.

(2) **Marble.** The result of the metamorphism of limestone or dolostone is **marble**. The crystals are commonly large and interlock to form a dense crystalline rock. Marble is characterized by its softness and its effervescence with dilute hydrochloric acid. (Little, if any, marble is known to occur in Minnesota.)

Table 3-1.5. Classification and identification of metamorphic rocks (from Ojakangas and Matsch, 1982).

Name	Structure	Texture	Mineralogy	Location in Minnesota and Other Remarks
			FOLIATED	
Gneiss	Coarsely to finely banded or streaked	Coarse to fine grained	Quartz, feldspar, micas, and hornblende common;	Minnesota River Valley. Common building stone; bands usually of different composition.
Schist	Finely foliated or schistose; breaks along foliation	Coarse to fine grained	Micas predominant; talc, chlorite, hornblende, garnet, quartz, etc.	South of Little Falls; northern Minnesota. Staurolite crystals can be collected near Little Falls; many platy or elongate minerals
Slate	Finely foliated; mineral grains not visible; has slaty cleavage	Extremely fine grained	Variable; mineral grains not visible to the naked eye	Thomson, Carlton; Vermillion district. Very smooth rock cleavages, used for roofing, etc.
			MASSIVE	
Quartzite	Massive, occasionally gneissic or schistose	Coarse to fine grained; equi-granular	Predominantly quartz, may have micas, feldspars, etc.	Iron Range; southwestern Minnesota. Very glassy luster; breaks through grains
Marble	Massive, occasionally gneissic or schistose	Coarse to fine grained; usually equigranular	Calcite, dolomite, sometimes with ser-pentine or other minerals	Effervesces in dulute HCL; cleavages visible
Granofels	Massive	Coarse to fine grained; usually equigranular	Variable; usually non-platy minerals (quartz, fledspar, garnet, pyro-xene, calcite, etc.)	General name for any massive metamorphic rock

3-1.01.05 WEATHERING

Weathering and chemical alteration are important aspects of rock classification that can affect both intact rock and rock mass properties. In its earliest stages, weathering is manifested by the discoloration of intact rock and only slight changes in rock texture. With time, significant changes in rock hardness, strength, compressibility, and permeability occur, and the rock mass is altered until the rock is eventually reduced to soil. Chemical alteration may occur at depths far below that of normal rock weathering.

Table 3-1.6 shows Mn/DOT's weathering nomenclature which is used for all rock types except shale (Table 3-1.7) and pure quartzose sandstones, such as the St. Peter and Jordan Sandstone (Tables 3-1.8 and 3-1.9).

Table 3-1.6. "Weathering" nomenclature for rocks (Mn/DOT August 1991).

Degree of Weathering	Description
ROCK TYPE, fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
ROCK TYPE, generally fresh with slight weathering	Rock generally fresh, joints stained, some joints may show thin clay coatings. Crystals in broken face shine brightly. Engineering characteristics essentially the same or very slightly reduced from those of fresh rock.
ROCK TYPE, slightly weathered	Rock slightly weathered, joints stained with discoloration extending into rock up to one inch. Joints may contain clay coatings. Engineering characteristics slightly reduced from those of fresh rock.
WEATHERED *ROCK TYPE*	Rock moderately weathered, significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Rock has dull sound under hammer and shows a significant loss of strength compared to fresh rock.
HIGHLY WEATHERED *ROCK TYPE*	Rock is highly weathered, all rock except quartz is discolored. In granitoid rocks, all feldspars are dull, discolored, and the majority show kaolination. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
SEVERELY WEATHERED *ROCK TYPE* (Residual Soil)	Rock is severely weathered, all rock except quartz is discolored or stained. Rock "fabric" is clear and evident, but reduced in strength to a strong soil. Some fragments of strong rock usually remaining.
VERY SEVERELY WEATHERED *ROCK TYPE* (Residual Soil)	Rock is very severely weathered, all rock except quartz is discolored or stained. Rock "fabric" is discernible, but the mass is effectively reduced to "soil" with only fragments of strong rock remaining.
RESIDUAL SOIL, parent rock is *rock type*	Rock reduced to "soil." Rock "fabric" not discernible or is discernable only in small scattered locations. Quartz may be present as dikes or stringers.

Table 3-1.7 relates shale classification to its Standard Penetration Test (SPT) value. It applies principally to the Decorah Formation, which is typically bluish green to greenish gray in color.

Table 3-1.7. Nomenclature for shale classification (revised August 1992).

Type of Shale	Typical SPT (bpf)
1. Weathered	
a. Mixed (not in-place) - clayey shale with foreign material intermixed (> 50% shale; for < 50% shale use soil description with indication of presence of weathered shale).	< 30±
b. Disturbed (not in-place)	
(1) Disturbed clayey shale* - fragments or bedding plane remnants that are not horizontally oriented. Bedding plane remnants are frequently difficult to see.	< 30±
(2) Disturbed fractured shale** - fragments or small pieces of relatively firm to hard fresh shale with bedding planes that are not horizontally oriented.	30 - 70±
c. In-place (top of weathered bedrock)	
(1) In-place clayey shale - has not been moved. Bedding plane remnants are horizontal.	< 30±
(2) In-place fractured shale - fragments or small pieces of relatively fresh horizontally oriented shale. Essentially similar to fresh shale below, except for blow counts.	30 - 70±
2. Fresh Shale (top of bedrock) - Dense, fractured, somewhat brittle. Bedding planes are horizontally oriented.	> 50/0.7'

Notes: Transitional forms between categories are likely to be encountered.

* Shale colored material which is generally quite plastic, uniform in texture, and has physical properties similar to clay.

** When a sample is broken open, small cubic or rectangular pieces of rather firm shale are evident. The pieces bounded by fracture planes are relatively unweathered and thus less plastic and more firm in contrast to clayey shale.

The empirical weathering criteria given in Table 3-1.8 was established by Mn/DOT for use with St. Peter Sandstone, but is also applicable to other quartzose sandstones that are predominantly free of finer grained (silt or clay) materials, such as the Jordan Sandstone. The primary determining factor is the number of blows per foot obtained from the Standard Penetration Test.

Table 3-1.8. Weathering criteria for St. Peter Sandstone (August 1992).

Term	Description	Typical SPT (bpf)
Sandstone Sand	Sand of generally uniform of rounded size and color, composed predominantly of rounded quartz grains; may contain up to 50% foreign glacial type sand or fine gravel.	30 - 50/0.7'±
Sandstone, weathered	Typical sandstone without foreign material. If present, such material is designated as the "Top of Bedrock." (Note: Three tenths of a foot penetration is borderline or transitional into fresh sandstone.)	50/0.7±' - 50/0.3'±
Sandstone, fresh	Dense, typical quartzose sandstone. Frequently, no samples are recovered.	50/0.3'± - 50/0'

3-1.02 ROCK MASS CHARACTERISTICS

Structural elements of the rock mass should be assessed in an attempt to define the overall engineering characteristics of the mass. Discontinuities are the major elements of rock mass classification. These features should be described in terms of frequency, spacing, roughness, bonding quality, and general continuity.

3-1.02.01 GEOLOGIC DISCONTINUITIES

1. Definitions. Geologic discontinuities are those breaks or visible planes of weakness in the rock mass that separate it into discrete units. They include structural features, such as joints and faults; and depositional features, such as bedding planes. Fractures include all breaks in a rock body or core sample, regardless of origin. Fractures may be of geological origin or they may be manmade.

   a. Joints. A **joint** is a surface of fracture or parting in a rock, along which there has been no visible movement parallel to the joint surface. Movement may occur at right angles to the joint surface causing the joints to separate or "open up." Joint surfaces are usually planar, and often occur with parallel joints to form a joint set. Two or more joint sets that intersect define a joint system. Joints may range from perpendicular to parallel in orientation with respect to bedding.

   b. Bedding Planes. A **bedding plane** is a planar or nearly planar surface that visibly separates each successive layer of stratified rock (of the same or different lithology) from the preceding or following layer. It may or may not be physically separated (appear as a fracture).

   c. Faults. A **fault** is a major fracture along which there has been appreciable displacement. The presence of gouge (pulverized

rock), bedding offset, and/or slickensided surfaces (commonly with mineral or clay coating) may be indicators of fault movement.

In practice, a precise distinction between joints and faults may not be possible nor significant.

2. Descriptions of Discontinuities in Cores. All geologic discontinuities that are visible in the core should be logged, whether or not they are manifested as a fracture. Care should be taken to distinguish between geologic discontinuities and fractures that are induced by drilling or handling. Induced fractures are not logged.

Properties of geologic discontinuities that are measured in core samples include attitude, spacing, Rock Quality Designation (RQD), average core length, and fractures per two-foot spacing.

a. Attitude. **Attitude** refers to the inclination of a discontinuity measured from horizontal. The inclination may be expressed in degrees or by using the quantitative descriptive terms given in Table 3-1.9.

Table 3-1.9. Description of attitude.

Term	Angle (degrees)
Horizontal	0 - 5
Shallow or low angle	5 - 35
Moderately dipping	35 - 55
Steep or high angle	55 - 85
Vertical or near vertical	85 - 90

b. Spacing. The **spacing** refers to the distance between discontinuities in the core, either joints or beds. (Faults are normally identified as joints since displacement is difficult to observe in the core.) Joint and bedding terms used to describe spacing are given in Table 3-1.10.

Table 3-1.10. Description of spacing.

Joint Term	Bedding Term	Spacing (in.)
-----	Laminated	< 0.5
Very close	Very thin	0.5 - 2
Close	Thin	2 - 12
Moderately close	Medium	12 - 36
Wide	Thick	> 36

c. Rock Quality Designation. **Rock Quality Designation** (RQD) is the percentage of intact pieces of rock greater than four inches in length relative to the total length of core run. RQD is determined for each core run. Fresh breaks created by field personnel or by drilling operations are not counted when calculating RQD. RQD may not be applicable for rocks of very low strength, or fissile or foliated rocks (such as shales), as they may break apart easily making identification of natural versus mechanical breaks nearly impossible.

RQD has been related to the overall engineering quality of the rock, with higher RQD values indicating more intact and better performing rock. This relationship is shown in Table 3-1.11.

Table 3-1.11. Relationship between RQD (Rock Quality Designation) and rock quality (from Deere, 1969).

RQD (%)	Rock Quality
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

d. Core Recovery. The percentage of rock core retrieved from the core hole divided by the total length of the core run is the **core recovery.** This is often an indicator of rock quality, with higher percentages suggesting a more intact rock mass.

e. Average Core Length. **Average Core Length** (ACL) is the average measurement of the length of core segments that are greater than four inches long. This value (expressed in feet) is a further indication of the relative spacing of the discontinuities, and is calculated for each core run.

f. Fractures per Two-foot Interval. **Fractures per two-foot interval** refers to the total number of fractures, excluding those induced by drilling or handling, that occur in a two-foot interval of core. Intervals with more than ten discontinuities are recorded as "> 10." Zones within a two-foot interval that have more fractures than can be readily measured are designated as "rubble."

3. Descriptions of Discontinuities in Outcrops. In some cases it may be desirable to further investigate the rock mass characteristics of a rock body. This will normally be accomplished by geologists from the Geotechnical Section. Additional descriptions might include orientation of discontinuities (strike and dip) and continuity (lateral extent of a discontinuity). These types of assessments are important when designing items such as the angle of a rock backslope, tunnel support, bearing capacity near rock cuts, or excavation methods.

3-1.03 RELATIVE ROCK HARDNESS

Rock hardness is a measure of rock strength, and is controlled by many factors including degree of induration, cementation, crystal bonding, and/or degree of weathering. Rock hardness may be estimated through manual tests on core samples or outcrops to yield a "field" identification (described in Table 3-1.12), which can be refined through further laboratory testing. The relative hardness of rock should be determined for each rock core sample or outcrop. Multiple designations would be required for variable rock conditions, such as lithological changes or weathering.

Table 3-1.12. Scale of Relative Rock Hardness, Mn/DOT, 1992

Term	"Field" Identification
Extremely Soft	Loose sand to soft core, crumbles or falls apart (very friable) upon removal from core barrel/split tube, or under slight pressure; (Uncemented Sandstone).
Very Soft	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure. In outcrop, can be excavated readily with point of geology pick. Sandstone can be deformed or crushed with fingers.
Soft	Can be scratched with fingernail. Can be peeled with a pocket knife. Crumbles under firm blows with geology hammer/pick. Sandstone cannot be deformed with finger, but grains can be rubbed from surface and small pieces can be crushed between fingers with some difficulty.
Moderately Hard	Cannot be scratched with fingernail. Can be peeled with difficulty with a pocket knife. Specimen can be fractured with a single firm blow of geology hammer/pick. Sandstone can be scratched with a knife, grains do not rub off surface.
Hard	Can be scratched by knife or geology pick only with difficulty. Several hard hammer blows required to fracture specimen.
Very Hard	Cannot be scratched by knife or geology pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.

3-1.04 STRENGTH TESTS

When quantitative values for rock strength are desired, strength tests, such as Unconfined Compressive Strength, Schmidt Hammer, or Point Load Tests can be performed. See the Geology Unit for assistance.

3-1.05 SAMPLE DESCRIPTION

It is important to give as complete a description as possible of the appearance and occurrence of rock materials in the field, for use in preliminary engineering and design. The following is a recommended list for the description of rock as viewed in an outcrop in the field, or in the lab as core samples:

- Lithology (with modifying mineral description, if possible);
- Weathering;
- Discontinuities (frequency and orientation of joint, faults, and beds);
- Color;
- Grain size and/or texture;
- Formation, member, or unit name; and
- Occurrence (extent of outcrop or depth below surface).

An example description of a rock outcrop would be:

Limestone, microcrystalline, with fossils, slightly weathered, thin to medium bedded, gray to dark gray (Platteville Formation). Outcrops between Sta 150 and Sta 160, approx. 10 feet above shoulder.

Habitual use of such a list will help in ensuring completeness and consistency of rock descriptions and aid in comparability of descriptions prepared by different field personnel.

3-2.0 SOIL SUBGRADE

3-2.01 FIELD AND LABORATORY TESTING

Soil subgrade materials are first identified by performing relatively simple field tests. Then, the field identifications can be verified or refined with laboratory testing.

3-2.01.01 FIELD IDENTIFICATION

Identification of soil types in the field, which is typically limited to defining the color and estimating the basic characteristics of texture and plasticity, is normally done without the benefit of major equipment, supplies, or time. It is necessary for a general assessment of sites during field reconnaissance activities and during the initial phases of more detailed work, such as the investigation of an emergency remediation or a planned geotechnical or pavement survey (Section 4-1.02.03). It may, in some instances, be the only effort ever expended towards classifying the encountered soils, but in most cases it will serve as an aid in assigning more detailed or elaborate laboratory tests.

With increased experience, field personnel should become more competent and skilled in accurately classifying the encountered soils, based solely on field techniques. Regardless of experience level, however; laboratory testing should be performed whenever possible to validate and sharpen the field technician's ability.

1. Texture. The following methods may be used in the field to estimate the soil's texture, which is defined as the relative size and distribution of the individual soil particles or grains. Although these methods are relatively simple to perform, they are not generally done by the Department's soils crew in the field.

   a. Visual Examination. By carefully looking at the soil, it can be divided into at least its gravel, sand, and fines (silt and clay combined) components. Since the naked eye can only distinguish particle sizes down to about 0.05 millimeters, silt- and clay-sized particles cannot be separated without further magnification.

      The examination is done by drying a sample, spreading it on a flat surface, and then simply segregating it into its various components and estimating the relative percentage of each. The percentage refers to the dry weight of each soil fraction, as compared to the dry weight of the original sample. Table 3-2.1 provides the defined particle sizes for each component and a common reference to aid in identifying the various particle sizes.

Table 3-2.1. Visual grain-size identification.

Classification		Approximate Size Limits Measured	Sieve	Comparison Example
Boulder		Over 3 in.	> 3 in.	Grapefruit
Gravel				
	Coarse	3 in. - 1 in.	3 in. - 1 in.	Lemon
	Medium	1 in. - 3/8 in.	1 in. - 3/8 in.	Diameter of penny
	Fine	3/8 in. - 2.0 mm	3/8 in. - No. 10	Pencil diameter to pea or rock salt
Sand				
	Coarse	2.0 mm - 0.42 mm	No. 10 - No. 40	Broom straw diameter to sugar or table salt
	Fine	0.42 mm - 0.075 mm	No. 40 - No. 200	Human hair diameter to powdered sugar
Silt and Clay		< 0.075 mm	< No. 200	Cannot be discerned with the naked eye

b. Sedimentation/Dispersion. This test is done by shaking a portion of the sample into a jar of water and allowing the material to settle. The material will settle in layers. The gravel and coarse sand will settle almost immediately, the fine sand within about a minute, the silt requiring as much as about an hour, and the clay remaining in suspension indefinitely. The percentage of each component is estimated by comparing the relative thickness of each of the layers in the bottom of the jar, keeping in mind that the larger sized particles will typically settle into a denser mass than the fines.

2. Plasticity. The ability to be molded within a certain range of moisture contents is termed plasticity. It is dependent upon the percentage and type of clay component, and it therefore requires differentiation between silt (non-plastic fines) and clay (plastic fines). The following methods can be used in the field for this differentiation.

a. Ribbon/Thread. In the ribbon/thread test, a roll of soil, moist enough to have workability, approximately one-half to three-quarters inch in diameter and about three to five inches long, is pressed between the thumb and index finger into a ribbon about one-eighth inch thick. The longer the ribbon can be formed before the soil breaks under its own weight, the higher the soil's plasticity. Highly plastic clays can be ribboned to perhaps four inches longer than their original cast. Clays of low plasticity can be ribboned only with some difficulty into short lengths, while non-plastic materials cannot be ribboned at all. Refer to Table 3-2.2.

b. Dry Strength/Breaking. The dry strength/breaking test is normally made on a dry pat of soil about one-half inch thick and about 1-1/4 inches in diameter, that has been allowed to air dry completely. Attempts are made to break the pat between the thumb and fingers, with very highly plastic clays being resistant to breakage or powdering and highly plastic clays being broken with great effort. Caution must be exercised with highly plastic clays to distinguish between shrinkage cracks, which are common in such soils, and a fresh break. Clays of low plasticity can be broken and powdered with increasing ease, while non-plastic soils have very little dry strength, crumbling or powdering on being picked up by the hands. This test is not generally done by the Department.

c. Shaking/Dilatency. In the shaking/dilatency test, a pat of soil about three-quarters inch in diameter is moistened to a putty-like state and placed in the palm of the hand. The hand is then shaken vigorously or jarred on a table or other firm object. If the sample's surface begins to glisten, it is an indication that moisture within the sample has risen to the surface. When this does not occur, the soil is probably clayey. Where this occurs sluggishly or slowly, the soil is predominantly silty, perhaps with a small amount of clay. For silts or very fine sands, the moisture will rise to the surface rapidly, and the test can be repeated over and over by simply remolding and then reshaking the pat. This test is not generally done by the Department.

The triangular textural classification of a soil is totally dependent upon its grain-size distribution and is discussed more fully in Section 3-2.02.01. However, the soil's classification can be reasonably estimated by determining its plasticity by any of the above methods. Table 3-2.2 shows the probable classification, based on plasticity, and in particular relates the classification to the results of the ribbon/thread test as performed by the Department.

Table 3-2.2. Probable classification based on plasticity.

Plasticity	Length of Ribbon (in.)	Mn/DOT Triangular Textural Classification
Non-plastic	0	Gravel
	0	Sand
	0	Loamy Sand
Slightly plastic to plastic		Sandy Loam
	0 - 1/2	Slightly Plastic
	1/2 - 1	Plastic
	1/4 - 1	Loam
	0 - 1	Silt Loam
Plastic to highly plastic	1 - 2	Sandy Clay Loam
	1 - 2	Clay Loam
	1 - 2	Silty Clay Loam
Highly plastic	> 2	Sandy Clay
	> 2	Silty Clay
	> 2	Clay

Quantitatively, a soil's plasticity is defined by its Atterberg limits, which are discussed in Section 4-2.06.02 and used in the AASHTO and Unified classification systems (Sections 3-2.02.02 and 3-2.02.03, respectively).

3. Organic Content. Generally, a sample can be adequately classified relative to organic content based on smell and feel. If it has a distinctive, musty, slightly offensive, or foul odor, which is enhanced in fresh samples or when exposed to heat; or if the sample has the feel of fresh to decomposed vegetable matter, it is generally organic. Organic soils are generally undesirable in the highway subgrade and are most often excavated and wasted. See Table 2-1.1 for classification of organic materials.

3-2.01.02 LABORATORY TESTING

In order to assign the proper classification to a soil, the texture (sieve and hydrometer analyses), plasticity (Atterberg limits), and the general organic content of the sample are required. These properties can be estimated in the field, as described above; or they can be developed more precisely through laboratory testing at the district level, as described in Section 4-2.06. The Atterberg limits and indices are described in Section 3-2.03.01.

The amount of testing to be performed depends upon the complexity of the stratigraphy, the experience level of the field personnel describing the obtained samples, and other factors. Obviously, the more complex the stratigraphy, or the less experienced the technician, the more the need for laboratory testing to

adequately describe the encountered conditions and verify the field classifications.

3-2.02 CLASSIFICATION SYSTEMS

The purpose of any classification system is to categorize soils by relating their appearance and behavior with previously established and documented engineering properties and performance. Simplicity of the classification system; reproducibility of the sample's classification at different times, locations, and by different field personnel; and applicability to all soils likely to be encountered are all attributes of a good classification system. The system of choice should make distinctions of practical importance to local designs and problems expected.

Mn/DOT, and many other state highway departments and agencies concerned primarily with subgrade performance, routinely uses the triangular textural and AASHTO classification systems to categorize soils. These systems are described in detail in the following two sections. Most private consultants use the Unified Soil Classification System, which is also briefly described below, to categorize various soils accordingly to their engineering characteristics. Finally, a rough correlation among the three classification systems is provided.

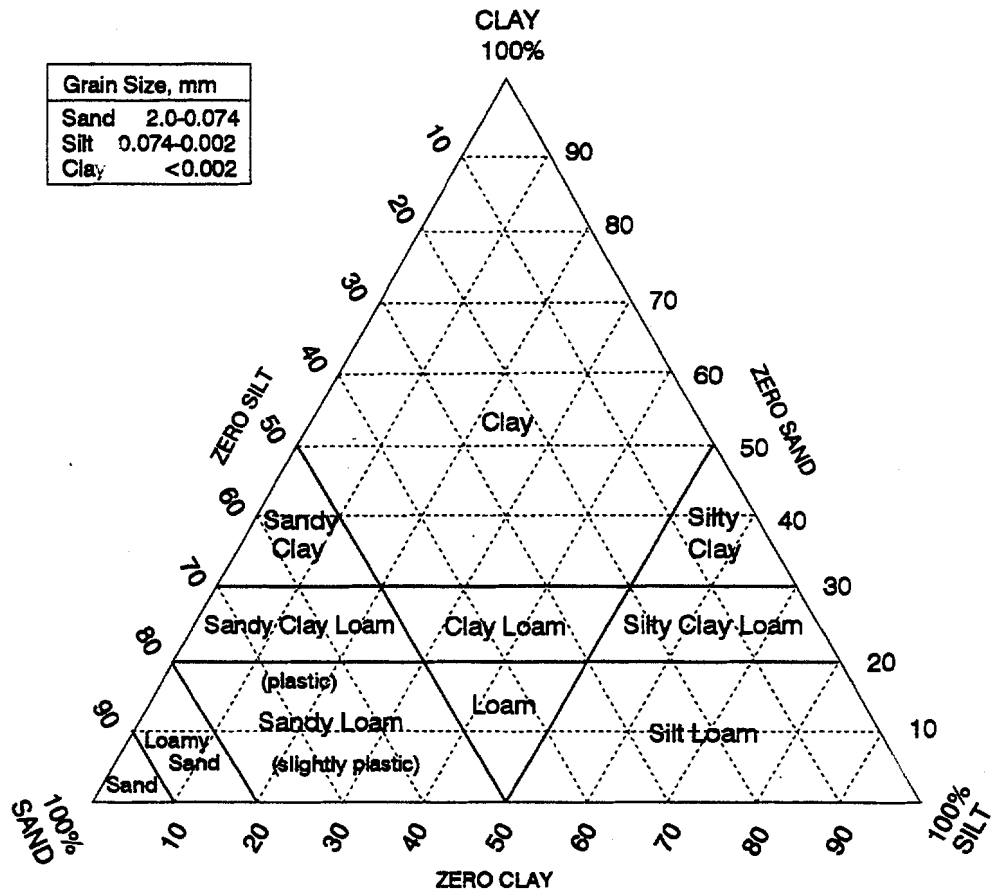
3-2.02.01 TRIANGULAR TEXTURAL

Textural systems of classification, based solely on the soil's texture or grain size distribution, have been developed by a variety of engineers and agencies since the earliest days of soil mechanics. Such systems include those developed by the Bureau of Soils (1890 - 95), Atterberg (1911), MIT (1931), and the U.S. Department of Agriculture (1938). This latter system is still in use and has been slightly modified by several highway departments, including Mn/DOT, presumably to better differentiate between local soils.

The following procedure is used to categorize a soil using Mn/DOT's triangular textural classification system. First, the sample's composition, by percentage of each of the following components, must be determined. Sand is between 2.0 millimeters (No. 10 sieve) and 0.075 millimeters (No. 200 sieve) in size; silt is between 0.075 and 0.002 millimeters in size; and clay is smaller than 0.002 millimeters (two microns) in size. Gravel and larger stones (larger than 2.0 millimeters) are disregarded. Then, each of the three axes of the diagram shown in Figure 3-2.1 are entered with the percent of sand, silt, and clay. Finally, the soil classification is determined by locating the common point of intersection.

In addition to the eleven possible classifications shown on Figure 3-2.1, **gravel**, defined as smaller than three inches in diameter and larger than 2.0 millimeters in size, is an acceptable classification. Any sample with more than about 25 percent gravel should include the term "gravelly" as a descriptor. Other modifiers to the textural classification should be used with restraint, but are permissible if the result of their use is clear and beneficial.

Figure 3-2.1. Triangular textural classification chart.



Three examples of obtaining the proper classification of a soil using the triangular textural classification chart (Figure 3-2.1) are given below:

Example 1. Given a soil sample with 18% sand, 58% silt, and 24% clay, what is its classification?

Entering the left axis at 18%, the bottom axis at 58%, and the right axis at 24%, and moving to the intersection point, the soil's classification is Silty Clay Loam.

Example 2. Given a soil sample with 47% sand, 32% silt, and 21% clay, what is its classification?

Entering the left axis at 47%, the bottom axis at 32%, and the right axis at 21%, and moving to the intersection point, the soil's classification is Clay Loam.

Example 3. Given a soil sample with 32% gravel, 38% sand, 22% silt, and 8% clay, what is its classification?

Entering the left axis at 70% (32% plus 38%), the bottom axis at 22%, and the right axis at 8%, and moving to the intersection point, the soil's classification is sandy loam. Since more than 25% of the soil sample is gravel, the term "gravelly" is added to the description, resulting in a classification of Gravelly Sandy Loam.

A brief description of each of the acceptable classifications, along with additional discussion, has been provided in Section 5-692.600 of Mn/DOT's Grading and Base Manual. To increase the ready usability of this manual, these descriptions are summarized as follows:

- **Gravel** consists of individual stones smaller than three inches and larger than 2.0 millimeters (No. 10 sieve) in diameter. Coarse gravel is larger than one inch; fine gravel is smaller than three-eighths inch in diameter. It is easily identified by visual inspection.

- **Sand** is loose and granular, with individual grains that can be readily seen and felt. If it is squeezed into a cast when dry, it will fall apart when the pressure is released; when moist, it will hold the cast until touched or jarred. Coarse sand is larger than 0.42 millimeters (No. 40 sieve); fine sand is smaller than 0.42 millimeters in diameter.

- **Loamy Sand** particles are all smaller than 2.0 millimeters (No. 10 sieve) in diameter. When dry, it is loose and granular, with the individual grains seen by the eye. It appears dirty when compared to sand because of its increased silt and clay content. When moist, it will form a cast which withstands some jarring. This stability of the moist cast differentiates loamy sand from clean sand.

- **Sandy Loam** is slightly plastic to plastic, depending on its clay content. It can be ribboned when moist. The individual sand grains can be readily seen and felt; however, it contains enough silt or clay to provide stability.

- **Loam** is a relatively even mixture of sand and silt with some clay. It is a mellow soil. It has a somewhat gritty feel and yet is smoother than sandy loam. It may be, but is not necessarily, topsoil or highly organic.

- **Silt Loam** normally contains a moderate amount of fine sand and usually a small amount of clay. When dry, it may appear quite cloddy, but the lumps can be readily broken up. When pulverized, it feels soft, fluffy, and floury. At normal moisture content, it feels smooth, slippery, or velvety. Although silt is non-plastic, it may ribbon depending on its clay content. In its natural state, it is frequently found to be very wet, due to its capillary affinity for water. In this condition it is very soft and could become "running" when subjected to jarring.

- **Sandy Clay Loam** contains mostly sand-sized particles. It feels gritty compared to silty clay loam. The sand particles can be readily seen and felt, but it is plastic and can be ribboned. It is found in a natural state only occasionally.

- **Clay Loam** is fine textured and uniform in structure. When dry, it breaks up into hard clods or chunks which are difficult to pulverize. It offers moderate resistance when ribboned.

- **Silty Clay Loam** contains mostly silt-sized particles. It is a fine-textured soil. When dry, it will form hard clods or chunks; when moist, it will ribbon without breaking and without much resistance. It feels softer than clay loam. At normal moisture content it feels slippery or smeary due to its silt content. It presents a dull appearance when compared to clay or clay loam. It is usually found in the form of soil pockets when the natural formation consists of glacial till.

- **Sandy Clay** contains mostly sand-sized particles, but has a high clay content. It is very plastic, yet has a gritty feel. When moist, it will form a long flexible ribbon, with considerable resistance to the fingers. It is very seldom encountered in a natural state.

- **Silty Clay** contains mostly silt-sized particles, but has a high clay content. It is very plastic, with a smooth feel. It will form long thin ribbons, with considerable resistance. In its natural state, it has the appearance and smoothness of butter. It normally occurs as soil pockets, rather than as a general soil mass.

- **Clay** is very fine textured and very plastic. When dry, it forms very hard clods or lumps which are extremely hard to pulverize. When moist, it forms long, thin, flexible ribbons, with marked resistance to the fingers; and can be rolled into thin threads. When smeared, it appears smooth and shiny.

In general, gravel and coarser sands are excellent for upper embankment materials; finer sands, loamy sand, sandy loam, and loam are excellent to good; clay loam and sandy clay loam are good to fair; silt loam and silty clay loam are fair to poor; and sandy clay, clay, and silty clay are poor.

3-2.02.02 AASHTO

In 1928, the Bureau of Public Roads introduced a classification system with eight soil groups, designated A-1 through A-8, to be used for assessing the suitability of road subgrade. Major revisions to the system, most recently in 1987, have resulted in the currently used classification system shown in Table 3-2.3. This system is based on the grain-size distribution (sieve Nos. 10, 40, and 200, only) and plasticity, and allows for a quick rational method of categorizing either undisturbed natural soil or fill and associating their properties relative to performance as a subgrade material. The system has been found to be applicable in areas with vastly different soil types and origins. In addition to the seven classifications shown in Table 3-2.3, an eighth classification, Group A-8, has been added to include highly organic soils (peat or muck).

Table 3-2.3. AASHTO classification of soils and soil-aggregate mixtures (from AASHTO M 145-87).

General classification	Granular Materials (35% or Less Passing No. 200)							Silt-Clay Materials (More than 35% Passing No. 200)			
	A-1			A-2							A-7
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve analysis, percent passing:											
No. 10	50 max.	-	-	-	-	-	-	-	-	-	-
No. 40	30 max.	50 max.	51 min.	-	-	-	-	-	-	-	-
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35. max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit		-	-	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index		6 max.	N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments` gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade			Excellent to good					Fair to poor			

*Plasticity index of A-7-5 subgroup is equal to or less than L.L. minus 30. Plasticity index of A-7-6 subgroup is greater than L.L. minus 30

There are three broad types under which the AASHTO groups and subgroups are divided. These are "granular" (A-1, A-3, and A-2), "silt-clay" (A-4 through A-7), and highly organic (A-8) materials. The transitional group, A-2, includes soils which exhibit the characteristics of both granular and silt-clay soils, making

subdivision of the group necessary for adequate identification of material properties.

The engineering considerations with granular and silt-clay soils are significantly different. The following discussion highlights major differences between these two types.

1. Granular. **Granular** materials include mixtures of rock fragments ranging from fine to coarse grained. Granular materials may include a non-plastic to slightly plastic soil binder, but are limited to 35 percent or less of the soil passing the No. 200 sieve (Note that Mn/DOT's Specification 3149 limits granular backfill to no more than 20 percent passing the No. 200 sieve). Granular materials generally provide the most desirable subgrade.

   It is possible, however, that some granular materials that border near the silt-clay boundary may have characteristics unsuitable beneath roadways under certain conditions, e.g., in the presence of water. This is because capillarity or chemical affinity for water may cause volume change or softening of the material when it is used as a subgrade. Therefore, where the subgrade is composed of these transitional soils, the relative levels of the encountered ground water and proposed grade should be carefully considered. Also, frost heave becomes a major concern as the materials grade into silts.

2. Silt-clay. **Silt-clay** materials are soils having more than 35 percent passing the No. 200 sieve. The behavior of these soils is dominated by the fines portion of the soil mass. Often, silt-clay materials (A-4 through A-7) may provide suitable road subgrade - when the soil's shortcomings are accounted for by proper design or design construction practices. Subgrades classified as A-6 or A-7 usually dictate a thickened pavement section and strictly maintained grading tolerances. A-7 materials are generally considered the poorest performers with regard to roadway construction.

   Reference is made to Mn/DOT's Grading & Base Manual, Section 5-692.606 for a more detailed discussion.

   Rather than an abrupt behavioral change at the 35 percent fines content, there is a transition from the granular materials to the silt-clay soils, as discussed above. Attention must be given to specific project details and construction practices where transitional soils are encountered.

   Developing the AASHTO classification for a given soil requires two steps. Initially, the soil is categorized into one of the eight groups designated by a group symbol, such as "A-1." This is accomplished by beginning at the left side of Table 3-2.3 and moving to the right across the chart, comparing the gradation and plasticity of the soil with each group's limits. Generally, soils further to the left of the chart, with lower-numbered group symbols, are preferable subgrade materials to those to the right; however, this is not necessarily the case, as with A-3 and A-2.

Subdivision of some of the groups is made for varying characteristics within the group, e.g., A-2-6 and A-2-7, for materials considered transitional from granular to silt-clay.

Two examples of obtaining the proper classification of a soil using the AASHTO system (Table 3-2.3) are given below:

Example 1. Given a soil sample with 75% passing the No. 10 sieve, 55% passing the No. 40 sieve, and 12% passing the No. 200 sieve; and a liquid limit of 20 and a plasticity index of 4. What is its classification?

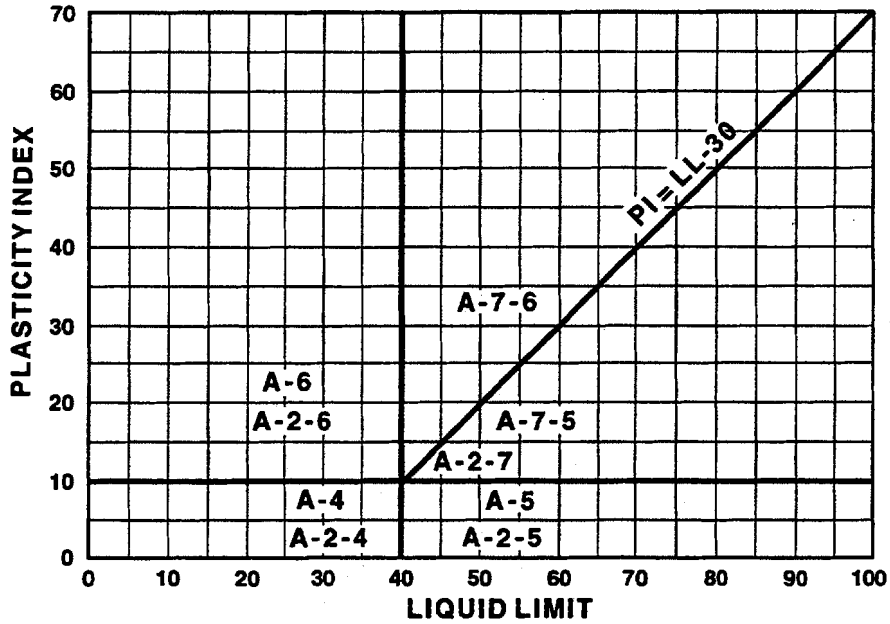
Starting at the left of the table and moving to the right, check the criteria first for the general classification and second for the group classification. First, the soil is granular because 35% or less passes the No. 200 sieve. Second, the soil is not an A-1-a because 50% or more passes the No. 10 sieve, not an A-1-b because 50% or more passes the No. 40 sieve, not an A-3 because 10% or more passes the No. 200 sieve; but it meets all of the requirements of an A-2-4, because 35% or less passes the No. 200 sieve, its liquid limit is 40 or less, and its plasticity index is 10 or less.

Example 2. Given a soil sample with 100% passing the Nos. 10 and 40 sieves, and 72% passing the No. 200 sieve; and a liquid limit of 45 and a plasticity index of 25. What is its classification?

Starting at the left of the table and moving to the right, check the criteria first for the general classification and second for the group classification. First, the soil is a silt-clay because 36% or more passes the No. 200 sieve. Second, the soil is not an A-4 because its liquid limit is 40 or more, not an A-5 because its plasticity index is 10 or more, not an A-6 because its liquid limit is 40 or more; but it meets all of the requirements of an A-7 because 36% or more passes the No. 200 sieve, its liquid limit is 41 or more, and its plasticity index is 11 or more. Further, since its plasticity index (25) is larger than its liquid limit minus 30 (15), the soil is an A-7-6.

Classifications of A-2, A-4, A-5, A-6, and A-7 soils can be checked graphically using Figure 3-2.2.

Figure 3-2.2. Relationship between liquid limit and plasticity index for silt-clay groups (from AASHTO M 145-87).



The subgrade quality of the silt-clay soils can vary from poor to good within each major group. Therefore, added to the group symbol found in Table 3-2.3 is a group index (G.I.), which indicates the plastic nature of the portion of the material passing the No. 200 sieve. Calculation of this group index is the second and final part of the AASHTO classification. Generally, the higher the value of the group index for a given group classification, the poorer the performance as a subgrade material.

The formula used to compute the group index is

$$G.I. = (F - 35)\,[0.2 + 0.005\,(LL - 40)] + 0.01\,(F - 15)\,(PI - 10) \qquad \text{Eq. 3-2.1}$$

where

G.I. = group index, reported as a positive whole number or zero;
F = percentage passing the No. 200 sieve, expressed as a whole number. This percentage is based only on the material passing the three-inch sieve.;
LL = liquid limit; and
PI = plasticity index.

Note that the first term of the above equation is ignored and only the second term, which accounts for the effect of the plasticity index, is used when the soil's group classification is either A-2-6 or A-2-7.

The group index is added in parenthesis after the group symbol, i.e., A-4(5) or A-7-5(17), etc. Two examples of how to compute the G.I. are given below:

Example 1. Assume that a soil classified as A-7(5) has 80% passing the No. 200 sieve, a liquid limit of 90, and a plasticity index of 50. What is its complete classification?

Then, the G.I. = (80 - 35) [0.2 + 0.005 (90 - 40)]
+ 0.01 (80 -15) (50 - 10) = 46.

And, the complete classification is A-7-5(46).

Example 2. Assume that a soil classified as A-2-7 has 30% passing the No. 200 sieve, a liquid limit of 50, and a plasticity index of 30. What is its complete classification?

Then, using only the second term in Equation 3-2.1, the G.I. = 0.01 (30 - 15) (30 - 10) = 3.

And, the complete classification is A-2-7(3).

The interplay of fines content, plasticity, and liquid limit, and how they influence the group index, is shown graphically in AASHTO M145-87.

The following descriptions provide profiles of each of the groups within the AASHTO classification system shown in Table 3-2.3:

- **Group A-1** includes well-graded gravel through fine sand with little or no non-plastic binder. Subgroup A-1-a includes stone fragments and gravel, with or without fines. Subgroup A-1-b includes predominantly coarse sand with or without fines. When properly placed and compacted, these materials perform well as road subgrades, as they are free draining and possess ample strength when properly placed.

- **Group A-3** is mostly poorly graded fine sand with few fines. Typical examples include blow sand, some beach sands, or poorly graded stream or river sand with minimal gravel content. A-3 soils are relatively free draining and possess desirable strength characteristics, but they may be somewhat difficult to compact due to their uniformity.

- **Group A-2** consists of transitional granular materials, all of which have less than 35 percent fines. Subgroup A-2-4 and A-2-5 have fines which are silty (non-plastic). Subgroups A-2-6 and A-2-7 have fines that are similar to A-6 or A-7 soils; that is, the fines are more plastic. A-2 soils, usually having group indices up to four, may range from good to fair as road subgrade. Frost susceptibility begins to be a problem in the A-2 soils, especially where the water table is in proximity to the zone of yearly frost depth.

- **Group A-4** soils are non-plastic to moderately plastic silts. Sand and gravel contents can range up to 64 percent. Group indices usually range

up to eight, with lower values indicative of higher gravel and/or sand contents. Again, where drainage is poor and free water is available to the silty subgrade, frost heave should be considered as a significant factor affecting the desirability of this material.

- **Group A-5** soils are similar in grain-size distribution to A-4 soils, but have higher liquid limits, indicative of diatomaceous or micaceous soils. The elastic nature of these soils, especially in the absence of sand, causes group indices to be higher than the A-4 soils, perhaps as high as 12. Frost considerations are, again, a significant factor affecting usage of these soils as road subgrade.

- **Group A-6** soils are clays, usually plastic with 75 percent or more passing the No. 200 sieve. With increasing sand content, up to 64 percent, the group index may be held low; but the group index can range up to 16 if the soil is devoid of sand. Usually, significant changes of volume will occur between dry and wet states. These materials may compact sufficiently at proper moisture content, but they will generally require a thicker pavement section to provide a non-yielding road surface. Frost considerations are usually outweighed by their affinity for water and the resulting volume changes and strength reductions that can result.

- **Group A-7** soils may be very elastic and plastic, subject to very high volume change with variations in moisture content. Strength can be low to high, but all A-7 soils are quite impermeable. A-7 soils are only utilized as road subgrade where nothing else is available.

- **Group A-8** soils are highly organic peats or mucks. These soils are highly undesirable for road subgrades and generally require removal.

3-2.02.03 UNIFIED

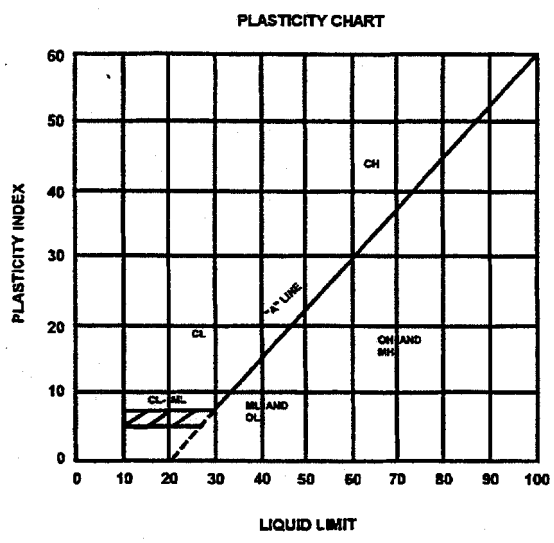
Another classification system widely used throughout the engineering community is the Unified Soil Classification System (USCS). The present system, modified by the U.S. Army Corps of Engineers and the Bureau of Reclamation, was introduced during World War II by Casagrande of Harvard University to assist engineers in the design and construction of airfields. As with the AASHTO system, the USCS utilizes grain-size distribution and plasticity characteristics to classify soils. The USCS, however, categorizes soils into one of 15 major soil groups, additionally accounting for the shape of the grain-size distribution curve.

Table 3-2.4 shows the USCS and the criteria utilized for associating the group symbol, such as "CL," with the soil. In this chart, symbols such as D_{60} refer to the maximum diameter of the soil particles of which 60 percent is smaller by weight as indicated on the gradation curve. Similarly, D_{10} would relate to the maximum diameter of the smallest 10 percent, by weight.

Table 3-2.4. Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM 3-357, 1953).

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS GREATER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF SAND AND GRAVEL FROM GRAIN-SIZE CURVE, DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN NO. 200 SIEVE), COARSE-GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5 PERCENT - GW, GP, SW, SP; MORE THAN 12 PERCENT - GM, GC, SM, SC; 5 TO 12 PERCENT - BORDERLINE CASES REQUIRING DUAL SYMBOLS (b)	$Cu = \frac{D60}{D10}$ GREATER THAN 4; $Cc = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3		
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR GW		
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	(a) GM d u	SILTY GRAVELS, GRAVEL-SAND MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	ABOVE "A" LINE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS	
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE WITH P.I. GREATER THAN 7		
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$Cu = \frac{D60}{D10}$ GREATER THAN 6; $Cc = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3		
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW		
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	(a) SM d u	SILTY SAND, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	LIMITS PLOTTING IN HATCHED ZONE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS	
			SC	CLAYEY SAND, SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE P.I. GREATER THAN 7		
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	PLASTICITY CHART		
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH	INORGANIC SILTS, MICACEOUS OR DIOTOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS			
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
	HIGHLY ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			



a) Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.
b) Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well graded gravel-sand mixture with clay binder.

The plasticity chart shown in the lower right-hand portion of Table 3-2.4 is a graphical representation of the USCS, based solely on the plastic and liquid limits (Section 4-2.06.02) of the material finer than the No. 40 sieve. Clays will plot above the "A-line" and silts below. The chart further divides the clays and silts into low (less than 50) and high liquid limits.

Two examples of using Table 3-2.4 to obtain the soil's proper Unified Classification are:

Example 1. Given a soil sample with 88% passing the No. 4 sieve, 38% passing the No. 200 sieve, a liquid limit of 15, and a plastic limit of 4, what is its classification?

Initially, it is determined that the soil is coarse grained because more than half of it (62%) is retained on the No. 200 sieve. It is then determined to be a sand because more than half of the 62% that is retained on the No. 200 sieve is smaller than the No. 4 sieve. Since there is more than 12% passing the No. 200 sieve, the soil is a sand with fines. Then, using the plasticity chart, the horizontal axis is entered at 15 and the vertical axis is entered at 11 (15 minus 4). The intersecting point plots above the "A line" (clay) and the soil is an SC.

Example 2. Given a soil sample with 77% passing the No. 200 sieve, a liquid limit of 44, and a plastic limit of 18, what is its classification?

Initially, it is determined that the soil is fine grained because more than half of it (77%) is smaller than the No. 200 sieve. Then, using the plasticity chart, the horizontal axis is entered at 44 and the vertical axis is entered at 26 (44 minus 18). The location of the intersecting point indicates the classification, which is a CL.

3-2.02.04 CORRELATION OF CLASSIFICATION SYSTEMS

The triangular textural, AASHTO, and USCS classification systems are all attempts toward the same end - to associate pertinent engineering properties with identifiable soil groupings. There *are*, however, some relatively minor differences between the systems. For instance, the triangular textural and AASHTO classification systems distinguish gravel from sand at the 2.0 millimeters size, whereas the USCS uses a break at the 4.76 millimeters size. The same coarse-grained soil could, therefore, have a larger percentage of gravel and a smaller percentage of sand in the triangular textural and AASHTO classification systems, as compared with the USCS.

Because of such differences, a direct comparison of soil classifications resulting from each of the three classification systems cannot be made; however, a general correlation can be drawn. Table 3-2.5 shows approximate equivalent classifications.

Table 3-2.5. Approximate equivalent classifications.

Mn/DOT Triangular Textural	AASHTO (Group Index)	Unified (USCS)
Gravel	A-1-a(0)	GW, GP
Sand	A-1-b(0)	SW, SP
Coarse	A-1-a, A-1-b(0)	SW, SP
Fine	A-1-b, A-3(0)	SW, SP
Loamy Sand	A-2-4, A-2-5(0)	SM, SC
Sandy Loam		
a) Slightly plastic	A-2-4, A-2-6, A-2-7(0)	SM, SC
b) Plastic	A-4(0-4)	SM, SC
Loam	A-4(0-4)	ML, OL, MH, OH
Silt Loam	A-4(0-4)	ML, OL, MH, OH
Sandy Clay Loam	A-6, A-5(0-16)	SC, SM
Clay Loam	A-6(0-16)	ML, OL, CL, MH, OH, CH
Silty Clay Loam	A-6, A-5(0-16)	ML, OL, CL, MH, OH, CH
Sandy Clay	A-7, A-7-6(0-20$^+$)	SC, SM
Silty Clay	A-7, A-7-5(0-20$^+$)	OL, CL, OH, CH
Clay	A-7(0-20$^+$)	CL, CH, OH, OL

3-2.03 ENGINEERING PROPERTIES

This section discusses the soil properties of principal interest for analysis and design of highway subgrades/embankments and pavement structures. Reference is made to Section 4-2.06 for more detailed test information.

3-2.03.01 ATTERBURG LIMITS

The engineering properties of fine grained soils vary with the amount of water present. As the water content changes, the consistency of these soils will also change as the material passes through the various physical states (namely liquid, plastic, semisolid, solid).

In 1911, A. Atterberg established limiting water contents for the different physical states/engineering behaviors. These limits are known as the Atterberg limits and consist of the liquid limit, the plastic limit and the shrinkage limit. The Atterberg limits are water contents (expressed as percentages) where the soil behavior changes. A description of the various states, limits, and indices is shown in Figure 3-2.3.

Above the liquid limit, LL, the soil-water system is a suspension. Below the liquid-limit and above the plastic limit, PL, the soil-water system is said to be in a plastic state. In this state the soil maybe deformed or remolded without the formation of cracks and without change in volume. The range of water content over which the soil-water system acts as a plastic material is frequently referred to as the plastic range, and the number difference is called the plastic index, PI:

Plastic Index PI - LL - PL

Somewhat below the plastic limit, the soil-water system reaches the shrinkage limit, SL. Reduction of the water content by drying below the shrinkage limit is not accompanied by a decrease in volume, instead air enters the voids of the system and the material become unsaturated.

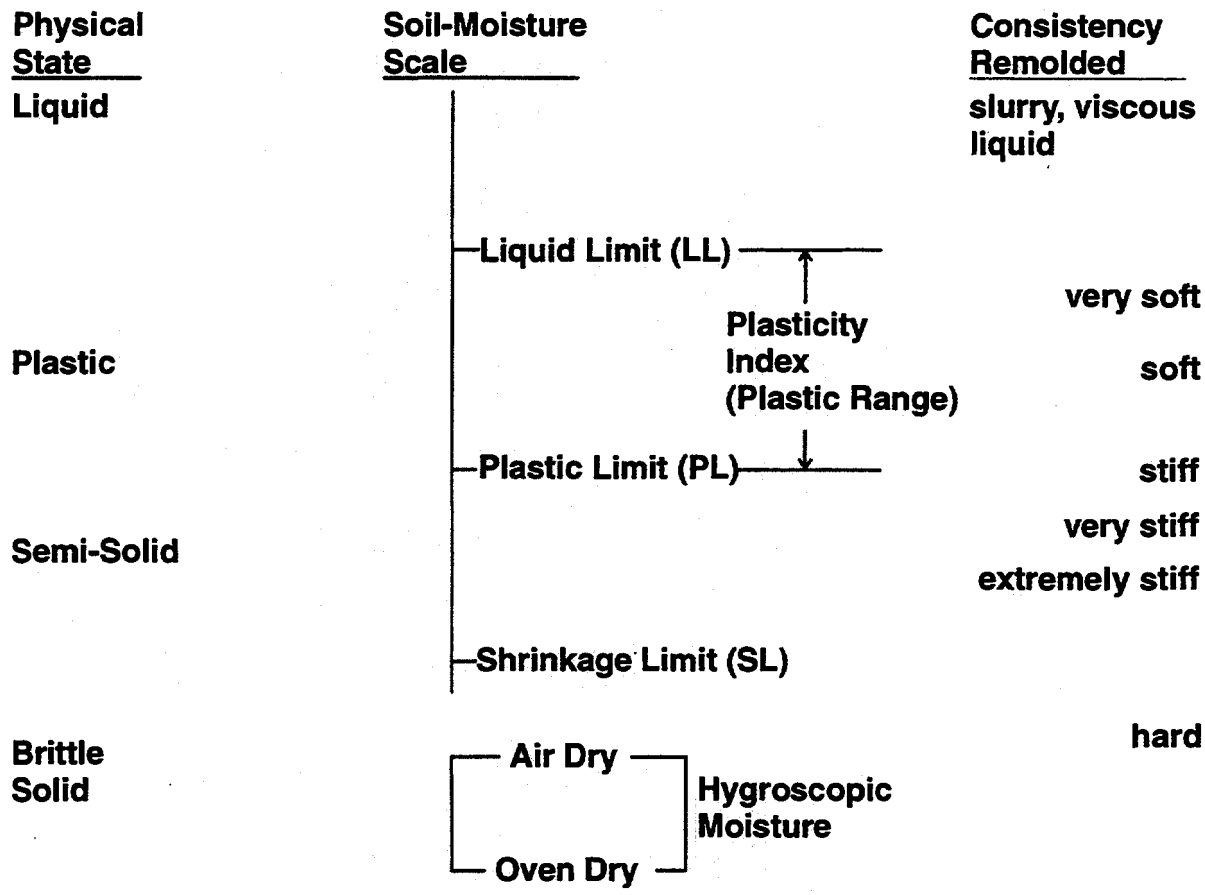
The Atterberg limits vary with the amount of clay present in a soil, the type of clay mineral, and the nature of the ions absorbed on the clay surface.

The most common use of the Atterberg test results is for soil classification. Soils with comparable limits and indices are classed together. The number is used to classify fine-grained soils and the indices to characterize soil behavior.

Generally, soils with high liquid limits are clays with poor engineering properties. A low plasticity index indicates a soil with little or no cohesion and plasticity, such as a granular type soil. Both the liquid limit and plasticity index are used to some degree as a quality measuring device for pavement materials.

Test methods for establishing the limits and the associated indices are described in Section 4-2.06.

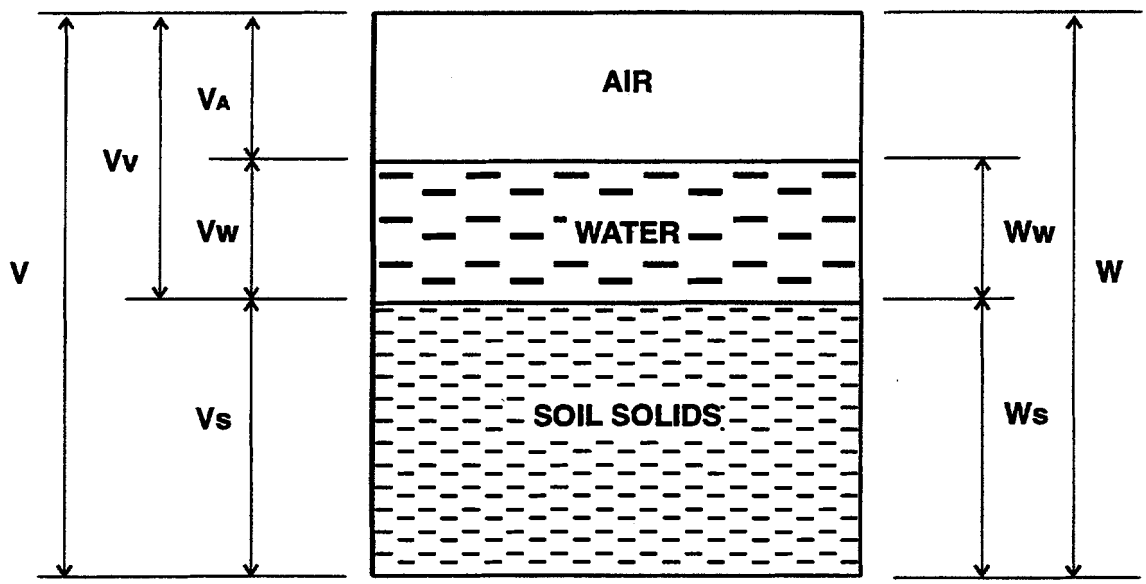
Figure 3-2.3 Atterberg Limits Relationships



3-2.03.02 IN-PLACE VOLUME AND WEIGHT RELATIONSHIPS

In-place soil is comprised of a mixture of soil solids, water, and air. The relative proportion of each of these constituents determines many of the properties of the soil. A soil block diagram, with symbols for each of its volume and mass components, is shown in Figure 3-2.4.

Figure 3-2.4. Volume and weight relationships for a soil.



The moisture content is the ratio of the weight of water to that of the dry soil solids, expressed as a percent; and is determined as follows.

$$w = \frac{W_w}{W_s} * 100 \qquad \text{Eq. 3-2.2}$$

where:

w = moisture content, %;
W_s = dry weight of solids, gm.; and
W_w = weight of water, gm.

The porosity is the ratio of the volume of voids to the total volume and may be expressed as either a percent or decimal; and is determined as follows.

$$n = \frac{V_v}{V} \qquad \text{Eq. 3-2.3}$$

where

n = porosity;
V_v = volume of voids, cm^3; and
V = total volume, cm^3.

The degree of saturation is the ratio of the volume of water to the total volume of voids, expressed as a percent; and is determined as follows.

$$S = \frac{V_w}{V_v} \times 100 \qquad \text{Eq. 3-2.4}$$

where

S = saturation, %;
V_w = volume of water, cm^3; and
V_v = volume of voids, cm^3.

The void ratio is the ratio of volume of voids to volume of solids and may be expressed as a percent or decimal; and is determined as follows.
where

$$e = \frac{V_v}{V_s} \qquad \text{Eq. 3-2.5}$$

e = void ratio;
V_v = volume of voids, cm^3; and
V_s = volume of solids, cm^3.

The density, or unit weight, of the soil mass is further divided into moist density and dry density. Moist density is the weight of water and soil solids divided by the volume of the soil mass. Dry density is the weight of only the soil solids divided by the volume of the soil mass. These values are determined by the following formulas.

$$\gamma_m = \frac{W_w + W_s}{V} \qquad \text{Eq. 3-2.6}$$

where

γ_m = moist density, pcf;
W_s = weight of solids, lb.;
W_w = weight of water, lb.; and
V = total volume, ft^3.

$$\gamma_d = \frac{\gamma_m}{1 + \frac{\omega}{100}} \qquad \text{Eq. 3-2.8}$$

or

$$\gamma_d = \frac{W_s}{V} \qquad \text{Eq. 3-2.7}$$

where

γ_d = dry density, pcf;
γ_m = moist density, pcf;
W_s = weight of solids, lb.;
w = moisture content, %; and
V = total volume, ft^3.

The density of the soil mass affects the strength of the soil. Generally, as the dry density increases, the strength increases. Also the potential for the soil to take on water at later times is decreased by higher densities. This is due to the decreased presence of air space in the soil mass.

The in-place moisture content of a soil can often be used, along with the soil classification, to determine the suitability of the material as a subgrade. Generally, as the moisture content of a soil increases its strength decreases. This is further illustrated by comparing the moisture content of the soil with its Atterberg Limits. For example, if the natural moisture content is near the liquid limit, the soil will quickly be disturbed by earth moving equipment and is unlikely to be suitable subgrade material. On the other hand, a natural moisture content below the plastic limit indicates a relatively firm material, which, provided additional moisture is not added, could provide a suitable subgrade. The moisture content of a soil should be expected to vary somewhat with the seasons and with rainfall.

3-2.03.03 SWELL/SHRINKAGE

Soils may undergo volume changes as the moisture content of the soil varies. This phenomenon is known as shrinkage or swell. Volume changes occur for moisture contents varying between the shrinkage limit and that required for 100 percent saturation.

Shrinkage is primarily related to the particle size and structure of the soil. Shrinkage is caused by the capillary action of the water in the soil mass. As water evaporates, tension is exerted on the soil solids causing stress. This stress in turn causes the soil solids to move closer together. As this stress can sometimes be great, significant volume changes can result.

The amount of swelling is dependent on many factors, including the clay minerals present, the soil structure, and several chemical aspects of the soil. Swell pressures have been recorded in excess of 20 ksf, which is equivalent to the weight of an embankment over 150 feet in height.

Shrinkage and swelling of the soil subgrade can be detrimental to the pavement structure. Table 3-2.6 can be used as a guide to determine which soils have swell potential.

Table 3-2.6. Probable Expansion as Estimated from Classification Test Data* (Swell potential of soils.)

Degree of Expansion	Volume Change*	Plasticity Index (PI)	Shrinkage Limit (SL)
Very High	> 30	> 35	< 11
High	20 - 30	25 - 41	7 - 12
Medium	10 - 20	15 - 28	10 - 16
Low	< 10	< 18	> 15

* After Holtz (1959) and U.S.B.R. (1974)

** Volume change as a percent of total change going from a dry to saturated condition (under a surcharge of 6.9 kPa).

3-2.03.04 FROST SUSCEPTIBILITY

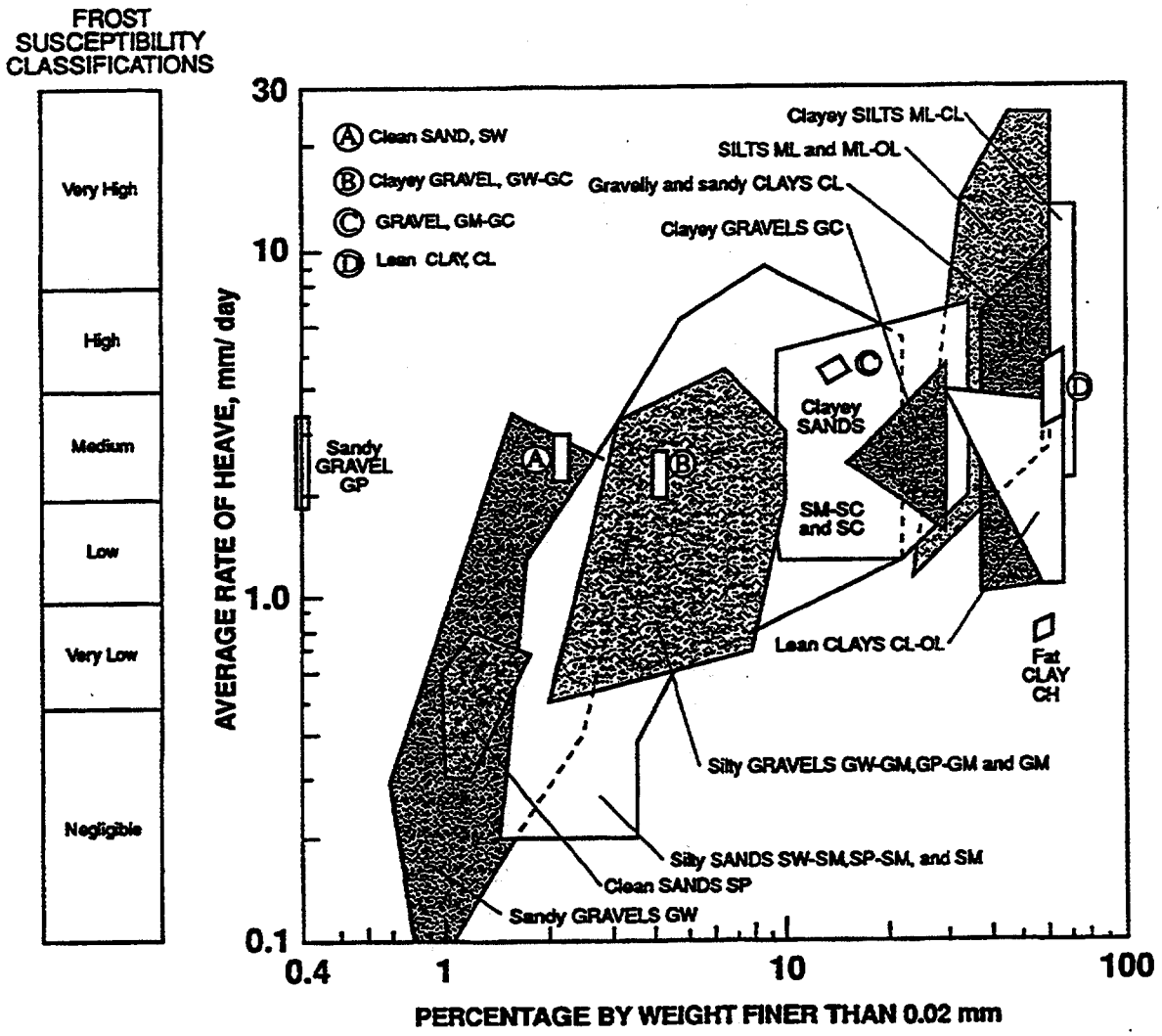
Frozen soils can exhibit frost heave and, sometimes, detrimental strength loss during the subsequent thawing process. When the ground water is shallow or when water is drawn above the ground water table through connected voids, sometimes called soil tubes, by capillary action and freezes, it can cause up to about nine percent expansion in volume of the soil. The depth to which freezing can occur is dependent upon the available moisture in the subgrade, the subgrade soil type, and the freezing index.

Frost heave can be reduced by keeping the ground water well below the pavement structure through natural or improved drainage; or by removing the

frost-susceptible materials and replacing them with non-frost-susceptible materials, or by mixing low-susceptible materials with existing highly susceptible materials.

Inorganic mineral soils that contain particles finer than 0.02 millimeters are susceptible to frost action. Generally, the greater the percent finer than 0.02 millimeters, the greater the susceptibility to frost action. Figure 3-2.5 shows the relative degree of frost susceptibility of various soil types. As can be seen, silts exhibit the highest rate of heave, while clean sands and gravels exhibit the lowest rate of heave.

Figure 3-2.5. Frost susceptibility of various soil types tested by the Corps of Engineers, 1950-1970 (from FHWA-TS-80-224, "Highway Subdrainage Design," August 1980).



The frost depth for a pavement section can be calculated using the following formulas and procedure.

$$L = 1.43w\gamma_d \qquad \text{Eq. 3-2.9}$$

where

L = volumetric heat of latent fusion, BTU/ft^3;
γ_d = dry density, pcf; and

$$F = \frac{h^2L}{48k} \qquad \text{Eq. 3-2.10}$$

w = moisture content, %

where

F = freezing index, degree days;
h = layer thickness, in.;
L = volumetric heat of latent fusion, BTU/ft^3; and
k = thermal conductivity, BTU/ft^2/hr./° F/in. (Values of 6.5 and 10.0 are generally used as constants for portland cement concrete and bituminous pavements, respectively.)

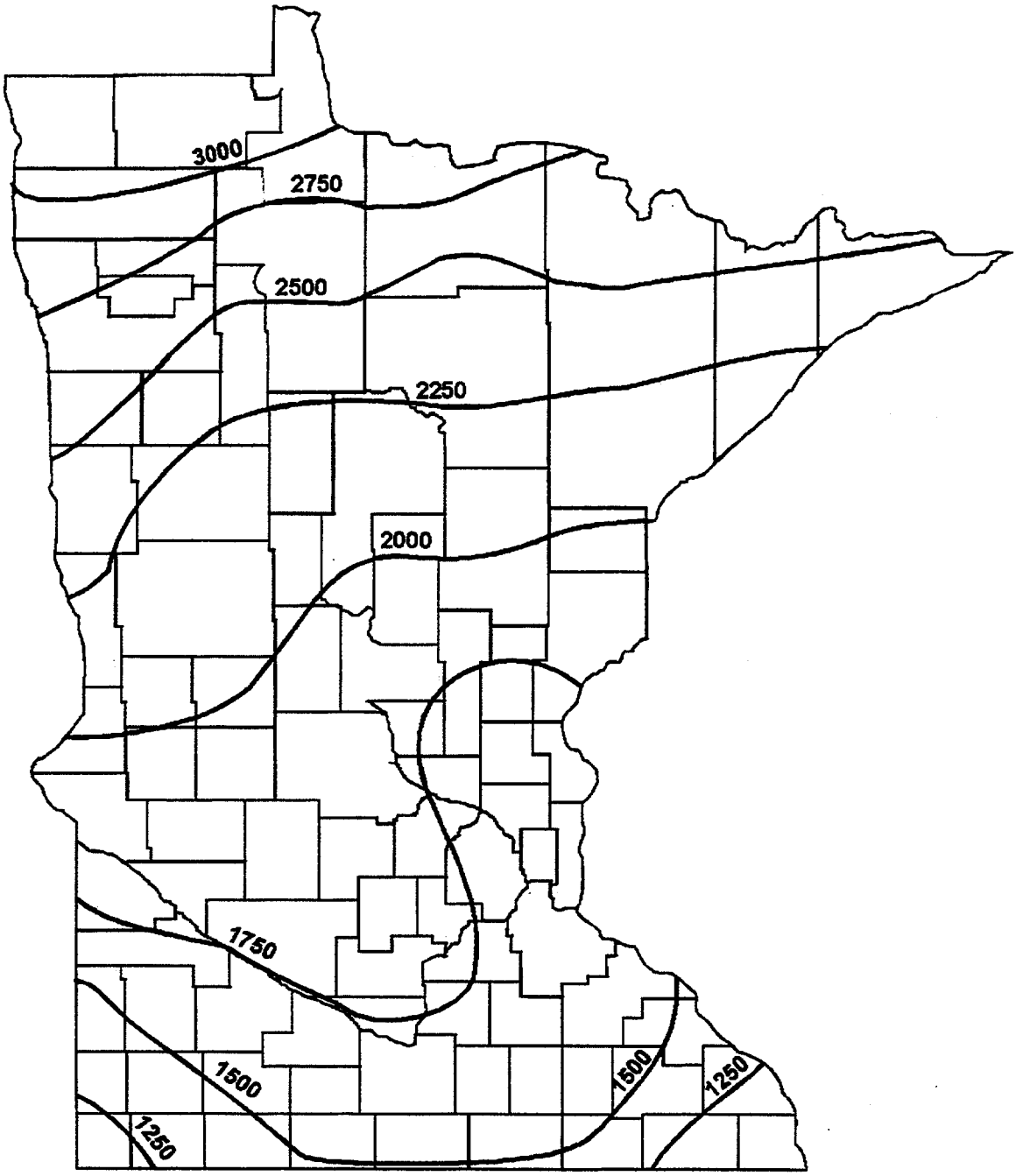
$$z = \sqrt{\frac{48kF}{L}} \qquad \text{Eq. 3-2.11}$$

where

z = frost depth, in.;
F = freezing index, degree days;
L = volumetric heat of latent fusion, BTU/ft^3; and
k = thermal conductivity, BTU/ft^2/hr./° F/in.

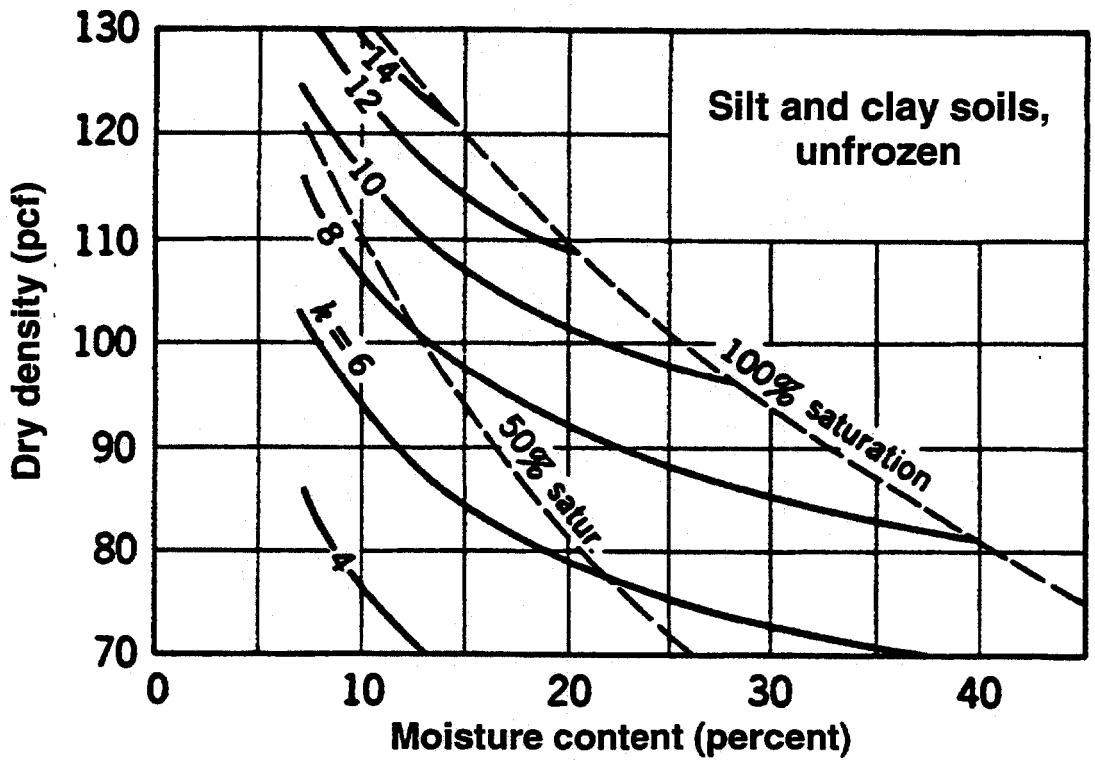
Values for w and γ_d are determined from field investigations. Values for freezing index and k are determined from Figures 3-2.6 and 3-2.7, respectively. The freezing index is determined from a cumulative plot of degree days versus time. A degree day represents one day with a mean air temperature one degree below freezing. Thus, 10 degree days may result when the air temperature is 31° F for 10 days or when the air temperature is 22° F for one day. Figure 3-2.6 shows a map of the freezing index for the state of Minnesota.

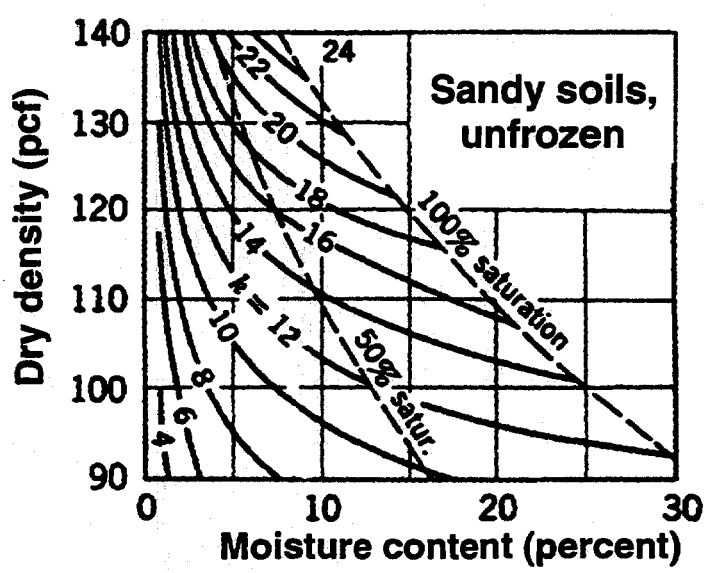
Figure 3-2.6. Freezing Index (from Corps of Engineers, EM 1110-345-306).



NOTE: MEAN FREEZING INDEX VALUE IN DEGREE DAYS BELOW 32° F

Figure 3-2.7. Thermal conductivity (from Kersten, 1952).





First, the number of degree days required to freeze the pavement must be determined using Equation 3-2.10. Next, the number of degree days required to freeze the base course is similarly determined using Equation 3-2.10. The freezing index determined from Figure 3-2.6 is then reduced by the number of degree days required to freeze the pavement and the base course. Lastly, the depth of frost penetration into the subgrade is determined using the reduced freezing index and Equation 3-2.11. If additional layers of base course or different soil layers are present, then the first two steps are repeated until the last layer is reached or until the reduced freezing index is equal to or lower than zero, which indicates that the maximum depth of frost penetration has been reached.

An example of calculating the depth of frost penetration for design purposes follows:

Example. Assume a 4-in. thick PCC pavement overlies 10 in. of base rock above a silty clay subgrade. The PCC has a dry density of 140 pcf and a moisture content of 2%. The base rock has a dry density of 120 pcf and a moisture content of 10%. The silty clay has a dry density of 110 pcf and a moisture content of 18%. The project location has a freezing index of 2,000 degree days (Figure 3-2.6). Calculate the frost depth as follows:

Using Equation 3-2.9,

$$L_{PCC} = (1.43)\ (2)\ (140) = 400,$$

$$L_{base\ rock} = (1.43)\ (10)\ (120) = 1{,}716, \text{ and}$$

$$L_{silty\ clay} = (1.43)\ (18)\ (110) = 2{,}831.$$

For PCC, k = 6.5 (constant). For base rock, k = 18 (Figure 3-2.7). For silty clay, k = 12 (Figure 3-2.7). Then, using Equation 3-2.10,

$$F_{PCC} = (4)^2\,(400) \div (48)\ (6.5) = 21, \text{ and}$$

$$F_{base\ rock} = (10)^2\ (1{,}716) \div (48)\ (18) = 199.$$

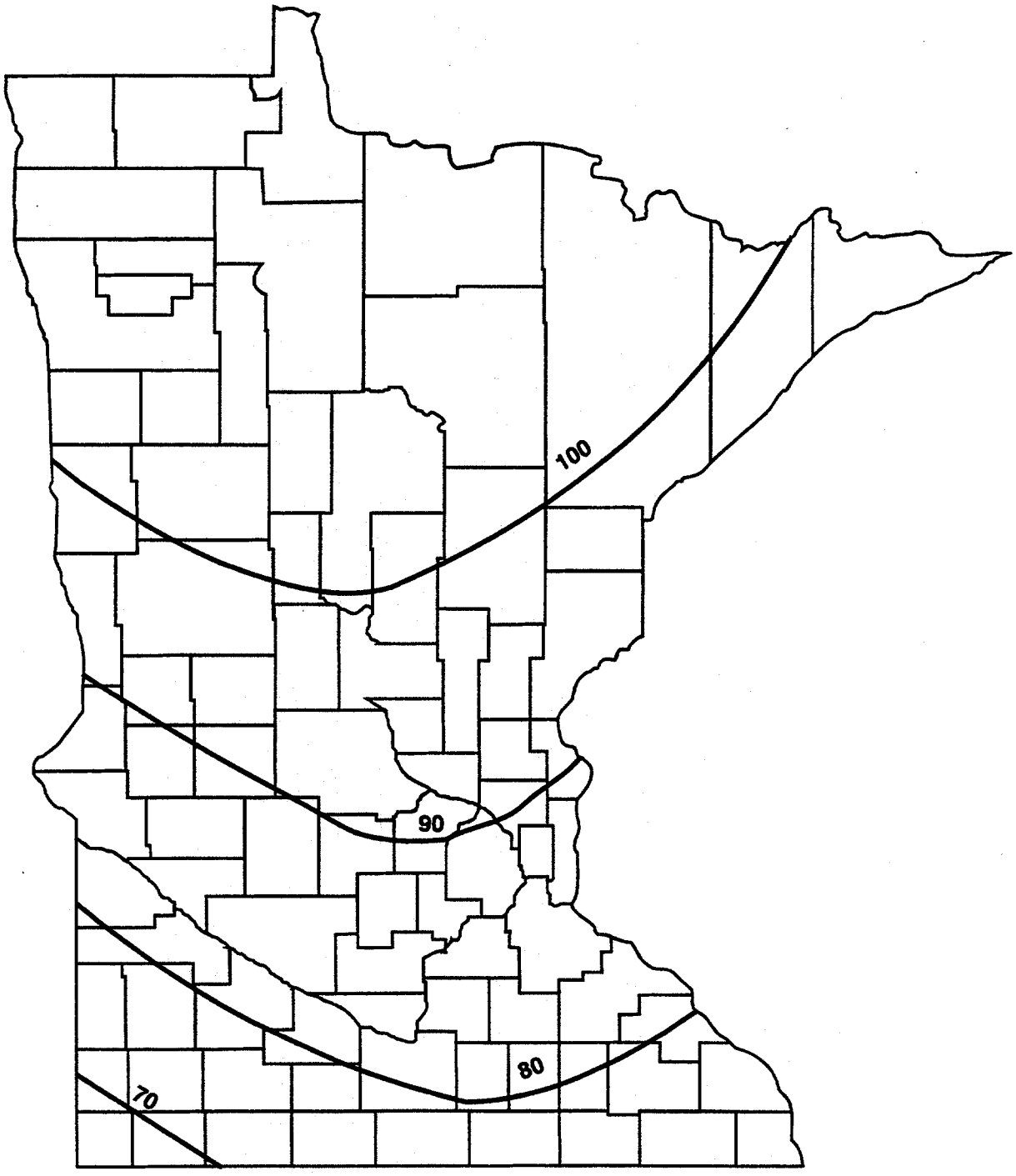
Therefore, $F_{silty\ clay} = 2{,}000 - (21 + 199) = 1{,}780$, and using Equation 3-2.11,

$$z_{silty\ clay} = \sqrt{(48)\ (12)\ (1{,}780) \div (2{,}831)} = 19.0 \text{ in.}$$

And, the frost depth = 4 + 10 + 19.0 = 33.0 in.

Figure 3-2.8, which follows, can be used to obtain an estimate of the maximum depth of frost penetration in Minnesota.

Figure 3-2.8. Estimated maximum depth of frost penetration (from FHWA-TS-80-224, "Highway Subdrainage Design," August 1980).



NOTE: DEPTH IN INCHES

3-2.03.05 STRENGTH

The strength, or resistance to deformation, of soils for use in pavement design is usually based on one of the following methods.

1. R Value. The test for R value was developed by Francis Hveem of the California Highway Department for use in designing flexible pavements. This test is based on the ratio of the horizontal reaction of a soil due to a standardized vertical load.

   The results of the test are expressed on a scale of one to 100. Assumed values, based on the Department's laboratory testing and past experience, are presented in Table 5-3.2(a) and (b).

2. California Bearing Ratio. The California Bearing Ratio (CBR) is a penetration test wherein a standardized piston is used to penetrate the soil at a standard rate. The load at each 0.1 inch of penetration is recorded, and the CBR is computed as the ratio of this load to the standard loads. The standard loads were obtained by testing a high-quality crushed stone material and are given in Section 4-2.06.05.

   This test was developed by the California Division of Highways as a result of surveys made during the years 1928 and 1929. The test procedures were standardized in 1929 and revised by the Corps of Engineers in the 1940s. As this test is arbitrary in nature, it is important that the standard procedures be followed completely. Results will depend on the test method used and the grain size, moisture, and density of the soil.

   The strength of a given soil is affected by the moisture content and density of the tested samples. Therefore, CBR values in the field will vary, depending on the moisture and density of the placed materials. Typical CBR values are shown in Table 3-2.8.

Table 3-2.8. Typical CBR values.

Mn/DOT Triangular Textural Classification	CBR Value
Gravel	20 - 80
Sand	10 - 40
Loamy Sand	5 - 40
Sandy Loam	5 - 40
Loam	< 15
Silt Loam	< 15
Sandy Clay Loam	5 - 40
Clay Loam	< 15
Silty Clay Loam	< 15
Sandy Clay	5 - 40
Silty Clay	< 15
Clay	< 15

The Department does not presently use the CBR for strength determination.

3. Resilient Modulus. The resilient modulus (M_r) is a dynamic test response, defined as the ratio of the repeated axial deviator stress to the recoverable strain. M_r is dependent on the axial deviator stress, the additional load applied to the top of the sample during the test, the number of load repetitions, and the confining stress. Note that M_r is a measure of the elasticity of the soil and is not a strength test per se.

   For subgrade soils, typical resilient modulus values vary between 3,000 and 15,000 pounds per square inch depending on the soil type, moisture content, density, deviator stress, and confining pressure.

   Additional information and equations are provided in Section 4-2.06.05.

4. Shear Strength. The shear strength of a soil is the soil's resistance to interparticle movement. This resistance is derived from two sources - cohesion and friction. Cohesion is the attraction of one particle to another. Friction results when one particle attempts to move past another. The shear strength is determined from the following formula.

$$s = c + (\sigma - u)\tan(\phi) \qquad \text{Eq. 3-2.12}$$

where

c = cohesion, psf;
s = shear strength, psf;
σ = normal stress, psf;
u = pore pressure, psf; and
ϕ = angle of internal friction, degrees.

The values of cohesion (c) and angle of internal friction (ϕ) are dependent upon the stress history of the soil, the current stress state, and the type of test used to determine the values.

There are two forms of shear strength - drained and undrained. The drained shear strength represents the condition where there is no increased water pressure due to the applied load. The undrained shear strength is the strength if the increased water pressure is not allowed to dissipate. Generally, the undrained strength is indicative of short term or construction-phase conditions, and the drained strength is indicative of long term conditions.

Unconfined compression tests can be used to determine the undrained strength of cohesive samples. Triaxial compression tests can be used to determine the drained and/or undrained strength of any soil sample. Direct shear tests are used primarily on rock samples but can also be used to determine the drained strength of soil samples. Undisturbed samples, obtained by the Foundations Unit, are required for all of the above strength tests.

Typical values of c and ϕ are shown in Table 3-2.9.

Table 3-2.9. Typical cohesion and angle of internal friction values.

Mn/DOT Triangular Textural Classification	c Cohesion (psf) Compacted	Saturated	ϕ Angle of Internal Friction (degrees)
Gravel	0	0	> 37
Sand	0	0	37 - 38
Loamy Sand	1,000 - 1,500	200 - 400	31 - 34
Sandy Loam	1,000 - 1,500	200 - 400	31 - 34
Loam	1,300 - 1,800	200 - 400	28 - 32
Silt Loam	1,300 - 1,800	200 - 400	25 - 32
Sandy Clay Loam	1,000 - 1,500	200 - 400	31 - 34
Clay Loam	1,300 - 2,200	200 - 400	18 - 32
Silty Clay Loam	1,300 - 2,200	200 - 400	18 - 32
Sandy Clay	1,000 - 1,500	200 - 400	31 - 34
Silty Clay	1,800 - 2,200	200 - 400	18 - 32
Clay	1,800 - 2,200	200 - 400	18 - 28

While these values can be used in pavement design, they are generally more applicable to slope stability and bearing capacity calculations. Therefore, these are most often used by the Foundations Unit, rather than at the district level.

The method of determining each of the above strength values is presented in Section 4-2.06.05.

3-2.03.06 PERMEABILITY

Permeability, in general terms, describes the rate of flow of a fluid through a porous medium. When that fluid is ground water, the terms **hydraulic conductivity** (K) and **coefficient of permeability** (k) are essentially equivalent. But for fluids other than water, such as oil, the permeability coefficient includes additional factors relating to the viscosity of the fluid.

Hydraulic conductivity and coefficient of permeability are used interchangeably in much of the present literature. Although hydraulic conductivity is the preferred usage, the general term **permeability** is often more easily used and understood. Both terms are expressed in units of length per time, such as feet per day (fpd) or centimeters per second (cps).

If water is present in the soil mass, at or near a level that must be excavated, then the water which will flow into the excavation must be accounted for. The greater the coefficient of permeability or hydraulic conductivity, the greater the volume of water that must be controlled. Therefore the value of k (or K) will impact both design and construction.

With respect to grain size, finer soils can, in general be expected to have lower permeabilities, and well-graded soils can be expected to be less permeable than more uniform soils. With respect to density, a decrease in permeability should be expected with increased dry density. Permeability values, as related to the soil's triangular textural and Unified classification are given in Tables 3-2.10 and 3-2.11, respectively. Figure 3-2.9 is a nomograph which may be used to estimate the coefficient of permeability of granular drainage and filter materials.

Table 3-2.10. Typical permeability values for various triangular textural classifications.

Mn/DOT Triangular Textural Classification	Relative Permeability	Coefficient of Permeability (fpd)
Gravel	Pervious to very pervious	10^{2} - 10^{5}
Sand	Pervious	10^{0} - 10^{2}
Loamy Sand	Impervious to semipervious	10^{-4} - 10^{-1}
Sandy Loam	Impervious to semipervious	10^{-4} - 10^{0}
Loam	Impervious to semipervious	10^{-4} - 10^{-3}
Silt Loam	Impervious to semipervious	10^{-4} - 10^{-3}
Sandy Clay Loam	Impervious to semipervious	10^{-4} - 10^{-2}
Clay Loam	Impervious	10^{-4} - 10^{-3}
Silty Clay Loam	Impervious	10^{-4} - 10^{-3}
Sandy Clay	Impervious	10^{-5} - 10^{-2}
Silty Clay	Very impervious to impervious	10^{-5} - 10^{-3}
Clay	Very impervious	10^{-7} - 10^{-5}

NOTE: Mn/DOT 3139, Class 5, Aggregate Base, typically has a coefficient of permeability between 0.3 and 0.5 fpd.)

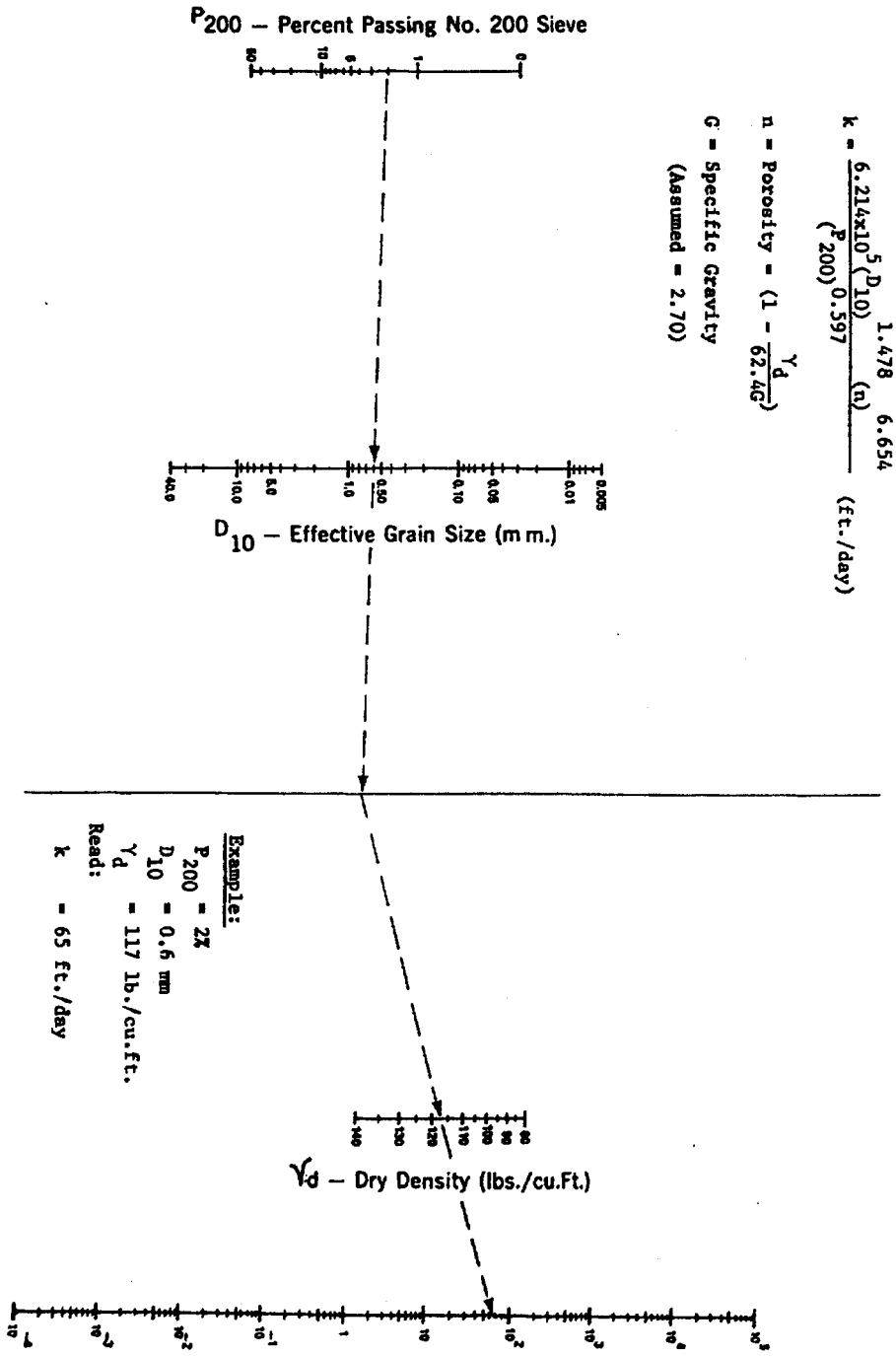
Table 3-2.11. Permeability values for various Unified classifications

(From FHWA-TS-80-224, "Highway Subsurface Drainage" August 1980.)

Unified Classification	Relative Permeability	Coefficient of Permeability (fpd) Typical	Average*
GW	Pervious	10^0 - 10^2	$7.4 \pm 3.6 \times 10^1$
GP	Pervious to very pervious	10^1 - 10^4	$1.8 \pm 0.9 \times 10^2$
GM	Semipervious	10^{-4} - 10^1	8.2×10^{-4}
GC	Impervious	10^{-5} - 10^{-2}	8.2×10^{-4}
SW	Pervious	10^0 - 10^2	-
SP	Semipervious to pervious	10^{-1} - 10^0	4.1×10^{-2}
SM	Impervious to semipervious	10^{-4} - 10^0	$2.1 \pm 1.3 \times 10^{-2}$
SM - SC	-	-	$2.2 \pm 1.6 \times 10^{-3}$
SC	Impervious	10^{-5} - 10^{-1}	$8.2 \pm 5.5 \times 10^{-4}$
ML	Impervious	10^{-5} - 10^{-1}	$1.6 \pm 0.6 \times 10^{-3}$
ML - CL	-	-	$3.6 \pm 1.9 \times 10^{-4}$
CL	Impervious	10^{-5} - 10^{-3}	$2.2 \pm 0.8 \times 10^{-4}$
OL	Impervious	10^{-5} - 10^{-2}	-
MH	Very Impervious	10^{-6} - 10^{-4}	$4.3 \pm 2.7 \times 10^{-4}$
CH	Very Impervious	10^{-7} - 10^{-5}	$1.4 \pm 1.4 \times 10^{-4}$

*From more than 1,500 soil samples compacted to the Standard Proctor maximum dry density. The ± entry indicates 90% confidence limits for the average values.

Figure 3-2.9. Chart for estimating coefficient of permeability of granular drainage and filter materials (from FHWA-TS-80-224, "Highway Subsurface Drainage," August 1980).



3-2.03.07 COMPRESSIBILITY

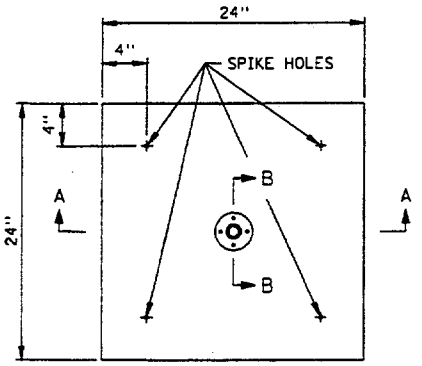
All soils compress when load is applied. Some, such as gravels and sands, compress only very slightly or almost none. Others, such as those containing large amounts of silt and clay, compress much more. When a load is applied to this latter group, water between the soil solids becomes pressurized. This pressure causes the water to flow out of the soil mass. As the water flows from the soil, the soil solids are allowed to move closer together (Figure 3-2.4). When the soil solids move closer, the overall volume of the soil mass decreases, which is reflected at the top of the soil mass as settlement or compression. This movement is largely unrecoverable, i.e., if the load is removed, the soil mass will expand only a small percentage of the amount it has compressed.

Generally, natural silts and clays are not going to compress enough to be of major concern to highway projects. Areas of high fill or soft clays and peat or muck areas are exceptions to this general rule. Highly compressible areas should have special studies made by the Foundations Unit to determine the extent of the area and the compressibility of the material. Once the properties of the material are known, settlements can be calculated. If anticipated settlements are too great, then the area can be pre-loaded by placing additional fill and monitoring the rate of settlement until it stops or slows to a tolerable pace.

Pre-loading is the surcharging of soft soils with additional fill to consolidate the underlying soils and offset settlements of the completed structure that would otherwise occur. Settlements of the original ground surface should be closely monitored during and after placement of the surcharge by the use of settlement plates. Settlement plates are installed at the ground surface before fill is placed. A section of pipe is attached to a flange on the plate; and, as the fill is placed, additional sections of pipe are attached. A typical settlement plate installation is shown in Figure 3-2.10.

Field settlement data should be plotted against time in a graphical manner. Fill height at the settlement plate for the same reading date should be plotted on the same graph, but above the settlement data. Fill height will thus be a rising line, until the final height of fill is reached and the settlement data will be represented by a falling line with time.

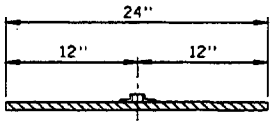
Figure 3-2.10. Typical settlement plate installation.



PLAN VIEW

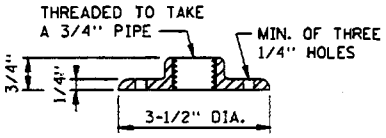
CONSTRUCT SETTLEMENT PLATE OF 3/4" PLYWOOD (EXTERIOR).

DRILL FOUR HOLES IN THE PLATE AS SHOWN, TO PROVIDE A SNUG FIT FOR FOUR 80d SPIKES.



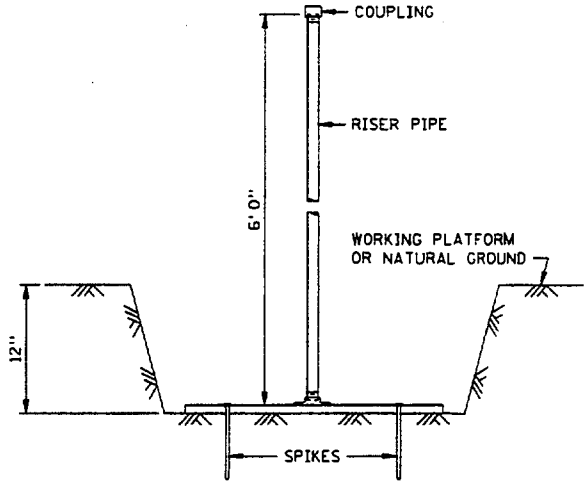
SECTION A-A

BOLT FLOOR FLANGE TO THE PLATE WITH A MIN. OF THREE 1/4" X 1-1/2" LONG BOLTS.



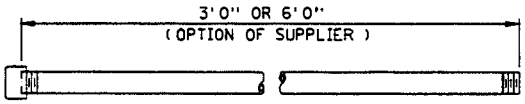
SECTION B-B

STANDARD BLACK 3/4" FLOOR FLANGE



SETTLEMENT PLATE INSTALLATION

TO SEAT THE PLATE, DRIVE FOUR 80d SPIKES THROUGH THE PLATE INTO THE UNDERLYING MATERIAL



RISER PIPE AND COUPLING

RISER PIPE SHALL BE 3/4" STANDARD WEIGHT BLACK STEEL PIPE. (A.S.T.M. A120)

SETTLEMENT PLATE

Typical values of compressibility are shown in Table 3-2.12.

Table 3-2.12. Typical compressibility values.

Mn/DOT Triangular Textural Classification	Compressibility (%) @ 1.4 tsf	Compressibility (%) @ 2.8 tsf
Gravel	0.3 - 0.8	0.6 - 1.4
Sand	0.6 - 0.8	1.2 - 1.4
Loamy Sand	0.8 - 1.1	1.4 - 2.2
Sandy Loam	0.8 - 1.1	1.4 - 2.2
Loam	0.9 - 2.0	1.7 - 3.8
Silt Loam	0.9 - 2.0	1.7 - 3.8
Sandy Clay Loam	0.8 - 1.1	1.4 - 2.2
Clay Loam	0.9 - 2.6	1.7 - 3.9
Silty Clay Loam	0.9 - 2.0	1.7 - 3.8
Sandy Clay	0.8 - 1.1	1.4 - 2.2
Silty Clay	1.3 - 2.6	2.5 - 3.9
Clay	1.3 - 2.6	2.5 - 3.9

3-2.04 SAMPLE DESCRIPTION

A typical sample description would include the primary classification and additional descriptive information, such as secondary descriptors, the AASHTO classification, and special terms describing the material's geologic origin. Each of these are discussed below.

3-2.04.01 PRIMARY CLASSIFICATION

The primary classification is taken from the triangular textural classification chart. It is one of the twelve groups described in Section 3-2.02.01.

3-2.04.02 SECONDARY DESCRIPTORS

The use of secondary descriptors should be restricted to color, moisture, consistency, compactness, structure, or cementation. Each of these items is highlighted below.

1. Color. A soil's color is usually only described when the sample has some apparent moisture content, because many soils will fade to similar pastel shades when dry. The importance of color is particular to a geologic area; however, certain colors, like black, gray, or green, are indicators of wet environments and/or organics and should be noted. Bright reds and yellows are indicative of particular mineral contents and a high oxidation environment. Acceptable colors are black, blue, brown, green, grey, red,

tan, buff, white, and yellow; optionally modified with density of the color (light or dark) or shading (reddish, brownish, etc.).

2. Moisture. The moisture of the soil should be described as dry, damp, moist, wet, or saturated. Dry soils are in a powdery condition, with no indication of moisture. Damp soils are well below their optimum moisture content. Moist soils are at or near their optimum moisture content and exhibit some color change with short periods of exposure to the air. Wet soils are between their optimum moisture content and saturation, with a high degree of moisture to the touch. Saturated soils cannot take on any additional water and may have free surface water.

3. Consistency. The strength of cohesive soils is quantified by their consistency. Terms utilized to describe consistency are very soft, soft, firm (sometimes referred to as medium stiff), stiff, very stiff, and hard. Consistency is often thought of as relating to plasticity, since in clays short term strength is based on cohesion; however, it is possible to have a very plastic soil (high cohesion) appear very soft.

   The Standard Penetration Test (SPT), torvane, and pocket penetrometer tests, all utilized by the Foundations Unit, are widely used to identify the consistency of clayey soils. However, the SPT (which is conducted in accordance with ASTM D-1586) is generally considered to be somewhat unreliable for cohesive soils. Table 3-2.13 provides the relationship for these test methods to determine the soil's consistency. It also includes a reference to the manual penetration test which can readily be performed by field personnel at the district level.

Table 3-2.13. Consistency of fine-grained soils.

Consistency Test	SPT (bpf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Manual Penetration
Very Soft	< 2	0 - 0.25	0 - 0.125	Squeezes between fingers when fist is closed
Soft	2 - 4	0.25 - 0.50	0.125 - 0.25	Easily molded by finger
Firm	5 - 8	0.50 - 1.00	0.25 - 0.50	Molded by strong pressure of fingers
Stiff	9 - 15	1.00 - 2.00	0.50 - 1.00	Dented by strong pressure of fingers
Very Stiff	16 - 30	2.00 - 4.00	1.00 - 2.00	Dented only slightly by finger pressure
Hard	> 30	> 4.00	> 2.00	Dented only slightly by pencil point

4. Compactness. The strength of granular soils is quantified by their compactness. It is described as very loose, loose, medium dense (sometimes referred to as medium), dense, or very dense. Again, a sand with a high internal friction angle (indicative of high strength) may be encountered in a very loose condition, so associating strength, per se, with compactness is not necessarily correct.

   The SPT is widely used to evaluate the compactness of granular soils, as related in Table 3-2.14.

Table 3-2.14. Compactness of granular soils.

Compactness	SPT (bpf)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

   In the case of silts, it is probably better to associate terms of consistency rather than compactness, since silts are difficult to compact and behave, under many circumstances, similarly to low-plasticity clays.

5. Structure. The feel and appearance of the soil mass is its structure. It indicates the arrangement of individual soil grains, which often is not directly discernable to the human eye. Mn/DOT's Grading and Base Manual describes eight different structures in Section 5-692-604. These are cloddy, crumb, hard pan, clay pan, massive, laminated, mealy, and fluffy. Several other possible soil structures are described as follows:

   - **Fissured** soils break along definite fracture planes.
   - **Slickensided** soils have striated, polished, or glossy fracture planes.
   - **Blocky** soils are cohesive, with many, small, flat-sided blocks. When the soil mass is broken apart, the small blocks tend to retain their individual identity.
   - **Lensed** soils are different from laminated soils in that the lenses are thicker, distinct, soil layers in a more massive soil layer. They are, however, thin enough that they would not be considered a separate stratigraphic layer.
   - **Homogeneous** describes a consistent feel or appearance throughout the soil mass, whereas **heterogeneous** describes a varying feel or appearance.

6. Cementation. The adhesion or binding together of soil grains or aggregates by a cementing agent is cementation. When this occurs, the degree of cementation, from firm to soft, should be indicated.

3-2.04.03 SPECIAL TERMS

Additional descriptors, such as geologic origin or commonly accepted vernacular, may be added by using special terms. Some special terms identifying particular soil types are topsoil, peat, quicksand, gumbo, diatomaceous earth, marl, alluvium, colluvium, loess, blow sand, volcanic ash, till, or glacial outwash. Most of the above terms are discussed in detail in Chapter Two of this manual and also in Mn/DOT's Grading and Base Manual.

As a minimum, the sample description should include the primary textural classification and two secondary descriptors, color and moisture condition. More complete descriptions are desirable, including the additional secondary descriptors listed above, the AASHTO classification, and any applicable special terms. Examples of proper field sample descriptions are:

- Minimum - SILTY CLAY, gray, wet.
- More Desirable - SILTY CLAY, gray, wet, soft, with fine sand laminations, A-6, lacustrine.

3-2.05 ABBREVIATIONS

Standard terms and abbreviations are desirable, both for saving time when describing soil samples and for ease of interpretation of field notes, profiles, etc., by others. The following terms and abbreviations (Table 3-2.15) are approved by the Department for use in describing samples and preparing field notes.

Table 3-2.15. Approved terms and abbreviations.

Soils Nomenclature

TERMS USED IN BORING LOGS

Heading/Location Terms	Abbv.
Trunk Highway	TH
Station	Sta
Reference Point	RP
Offset	OS
Left	LT
Right	RT
Centerline	C/L
Plus	+
Northbound	NB
Southbound	SB
Eastbound	EB
Westbound	WB
Feet	FT
Tenths	Tenths
kilometers	km
meters	m
millimeters	mm
Months	Mo
Matls Engr's Name	MEngr
Crew's Name	Crew
Individual Letters A thru Z	(Upper Case)
Numbers 0 thru 9	
Ramp	Ramp
Loop	Loop
Frontage Road	FR
Service Drive	SrDr
Source Name/Number	Srce
Mainline	ML
Shoulder	SHLD
State Project No	SP
Control Section	CS
County Road	CR
County State Aid Highway	CSAH
Soils Engr's Name	SEngr
Section	Sec
Township	Twp
Range	Rng
Decimal Point	
One-quarter	1/4
One-half	1/2
Three-quarters	3/4
Plus/Minus	+/-
Approximate	Approx
Question Mark	?
Inplace	Inp

Surfacing Terms	
Concrete	CONC
Bituminous	BIT
Aggregate	AGG
Bit Treated Base	BTB

Material Terms	
Gravel	G
Sand	S
Sand and Gravel	S&G
Loamy Sand	LS
Loamy Sand and Gravel	LS&G
Sandy Loam	SL
Loam	L
Silt Loam	SiL
Silty Clay Loam	SiCL
Clay Loam	CL
Sandy Clay Loam	SCL
Clay	C
Silty Clay	SiC
Sandy Clay	SC

Boulder Terms	
Limestone	Lmst
Sandstone	Sst
Dolostone	Dolo
Shale	Shale
Boulder (over 3")	Bldr

Moisture Terms	
dry	dry
damp	damp
moist	moist
wet	wet
saturated	sat

Color & Shade Terms

black	blk
brown	brn
grey	gry
yellow	yel
tan	tan
blue	blu
white	wht
green	grn
red	red
orange	orng
dark	dk
light	lt

Textural Terms

Very Fine	VF
Fine	F
Coarse	Cr

Plasticity Terms

slightly plastic	slpl
nonplastic	nonpl
plastic	pl
highly plastic.	hpl

Consistency Terms

Very soft	Vsoft
soft	soft
firm	firm
stiff	stiff
Very stiff	Vstiff
hard	hard
Very hard.	Vhard

Compactness Terms

Very loose	Vloose
loose	loose
medium dense	meddense
dense	dense
Very dense	Vdense

Water Condition Terms

water level	H2O
Flowing Artesian	FlArt
perched water	perch

Peat Classification Terms

Peat	peat
spongy	spongy
fiberous peat	fpeat
semi fiberous peat	sfpeat
well decomposed peat	wdpeat
partially decomposed peat	pdpeat

Organic Content Terms

non organic	nonorg
slightly organic	slorg
organic	org
highly organic	horg

Descriptors

Deteriorated	Det
Stripped	Strpd
Sound	Snd
Unsound	UnSnd
weathered	wx
Bedrock	bedrock
debris	debris
chips	chips
seams	seams
layers	layers
marbled	mrbl
mottled	mtld
fill	fill
cut	cut
fat	fat
frozen	frzn
ice lenses	icelns
ice	ice
topsoil	ts
slope dressing	sd
wood	wood
woody	woody
roots	roots
shells	shells
Iron Oxide Stained	IOS
till	till

Miscellaneous

with	w/
without	w/o
variable	var
natural	nat

Not Applicable ..... N/A
and ..... &
or ..... or
to ..... to
included ..... inc
Gas Smell ..... GasSm
Road Tar ..... RdTar
sample ..... smpl
Soil ID ..... SID
R-Value ..... RVal
Gradation ..... Grad
Fertility ..... Fert
Extraction ..... Xtract
at ..... @
Time Of Drilling ..... TOD
hour ..... hr
no return. ..... noret
poor return. ..... prret
fluid ..... fluid
REFUSAL ..... REFUSAL

Equipment

Auger Truck ..... AT
Hand Auger ..... HA
50# Sounding Hammer 50SH
20# Sounding Hammer 20SH
Portable Auger ..... PA
Dynamic Cone Penetrometer ..... DCP

Other comments:

- no periods
- keep UPPER & lower case letters

Color & Shade Terms

black	blk
brown	brn
grey	gry
yellow	yel
tan	tan
blue	blu
white	wht
green	grn
red	red

Peat Classification Terms

Peat	peat
spongy	spongy
fiberous peat	fpeat
semi fiberous peat	sfpeat
well decomposed peat	wdpeat
partially decomposed peat	pdpeat

Organic Content Terms

3-3.0 PAVEMENT MATERIALS

Pavements are typically constructed in distinct layers, with each layer serving a purpose within the overall pavement structure. This section discusses those materials that are used in the construction of the various layers of a pavement. In addition to the different types of materials available, several of the material properties are of interest to the District Design Engineer. These are discussed where appropriate.

3-3.01 CLASSIFICATION

Pavement materials are generally classified according to their use within a pavement system and, within a given layer, according to the material's type. Thus, one level of classification would be subbase, base, and surface. This classification scheme describes the location of the materials within the pavement system. However, it also provides information about the purpose of the materials. For example, the surface layer is intended to provide a smooth, safe, and durable layer upon which vehicles can travel, and it contributes to the structural capacity of the pavement. The base course also contributes significantly to the pavement structure, particularly when treated with a stabilizing agent, and it also should provide positive drainage and protection against frost. Generally, a subbase or series of subbases are also included in a pavement cross section to increase structural capacity, to raise pavement layers above the frost line, or to provide a leveling course for the placement of base layers.

As noted above, in addition to classifying materials according to their location within a pavement, materials are also categorized by type. In addition to aggregate surfacing, there are two common types of pavement surfacings, bituminous and portland cement concrete (PCC). And underlying these surfacings are a wide variety of base and subbase materials. Common to all of the pavement materials is the use of some sort of aggregate, either with or without a binder.

3-3.01.01 AGGREGATE BASE

The construction of a high-quality aggregate base starts with the use of quality aggregates. The types and quality of aggregates to be used for base courses are described in Mn/DOT's Specification 3138. There are six different classes of aggregate described and a total of 12 different gradations included in this specification. The aggregates that meet these specifications are considered to be of high quality; therefore, restrictions are placed on their characteristics. In addition to a gradation specification for each of the classes, Los Angeles Rattler Loss limits are described and composition, crushing, and allowable percent shale are detailed. All of the gradations for aggregate base courses, with the exception of Classes 2 and 2A, permit the inclusion of recycled bituminous mix or concrete, subject to specified limitations on bituminous content and the presence of other contaminants. The use of crushed concrete for base, in the presence of edge drains, should be in accordance with current specifications, as leachates may clog filters.

There are two different approaches to the design and construction of high-quality aggregate bases. Such a base course may be permeable (drainable), or it may

be densely graded (relatively impermeable). Both aggregate base types are discussed below.

1. Permeable Bases. Permeable or drainable base courses are constructed for the purpose of rapidly draining moisture that infiltrates the pavement structure. The combination of base thickness and material permeability should be such that moisture rapidly flows through the base, preferably draining the layer in less than two hours following the end of a precipitation event. A study recently completed for the FHWA showed that, in general, the presence of a permeable base course enhanced the performance of the pavement, especially in areas where the pavement materials were affected by the presence of moisture. Essential to the good performance of such permeable bases, however, was the design and construction of adjacent layers, the design and placement of outlet drains, and the presence of a properly designed filter layer between the permeable layer and the subgrade soils beneath it.

   Permeable aggregate bases should consist of a hard, durable, crushed, angular aggregate with essentially no fines (material passing the No. 200 sieve). The LA abrasion index of the aggregate must be 40 or less and the aggregate must contain less than 10 percent minus 200 sieve size insoluble residue. Permeable bases may be unstabilized, or stabilized with either asphalt cement or portland cement. (Portland cement is currently not used by Mn/DOT.)

Table 3-3.1 gives the aggregate gradation specified by the Department for permeable aggregate bases.

Table 3-3.1. Gradation for permeable aggregate bases (PAB).

Sieve Size	Unstabilized Percent Passing	Stabilized* Percent Passing
1-1/4 in.	100	100
1 in.	95 - 100	95 - 100
3/4 in.	65 - 95	85 - 98
3/8 in.	30 - 65	50 - 80
No. 4	10 - 35	
20 - 50		
No. 10	3 - 20	0 - 20
No. 40	0 - 8	0 - 5
No. 200	0 - 3	0 - 3

* Typically stabilized with two to three percent asphalt.

The aggregate is also required to have a uniformity coefficient (D_{60} divided by D_{10}) equal to or greater than 4.0 for unstabilized base and 2.5 for stabilized base, where D_{60} is the diameter of the soil particle of which 60 percent is smaller by weight and D_{10} is the diameter of the soil particle of which 10 percent is smaller by weight. A minimum of 85 percent two-face

crushing is required for unstabilized bases and 65 percent for stabilized bases. See standard Special Provision for detailed specifications.

Unstabilized permeable aggregate bases are somewhat susceptible to deformation from construction equipment, and great care must be exercised to not damage it during construction.

As previously noted, the placement of a well-designed filter layer is essential to the performance of the permeable base layer. The purpose of this layer is to prevent fines from working their way up into the voids in the drainage layer, thereby clogging the voids and reducing or eliminating the permeability of the base. The cost of permeable aggregate bases in new design is generally more than conventional dense-graded materials.

The placement of a permeable base layer must be accompanied by the inclusion of longitudinal edge drains. The purpose of these drains is to remove the moisture collected in the drainage layer away from the pavement structure. There are a number of different types of edge drain designs, but the basic principle remains the same - to quickly remove the moisture that is collected in the drainage layer. In new designs this is most easily done with a properly designed longitudinal trench collector system, which includes the use of collector pipes, geotextiles, and properly sized backfill material. Geocomposite (fin) drains are not recommended for use with permeable bases.

Detailed information on the design and construction of the filter layer, edge drains, and other aspects of providing subsurface drainage is provided in the FHWA's Technical Paper 90-01, *Subsurface Pavement Drainage*. In addition, the FHWA recently developed an interactive computer program called DAMP (Drainage Analysis and Modeling Programs) that allows for the comprehensive consideration of drainage in the overall pavement design process. See Chapter 5 for additional discussion of permeable bases.

2. Dense-graded Aggregate Bases. The other type of aggregate base consists of a dense-graded granular material. Granular materials meeting Mn/DOT's Specifications for Classes 3, 4, 5, and 6 may meet the definition of a dense-graded material. Through the selection of the proper particle size, distribution, and range, a densely graded aggregate base, when not saturated, will provide adequate support to the pavement structure. The saturated base may lose fines due to pumping when subjected to loading, which can reduce the support provided to the surface layer by the base. In general, a saturated base will result in loss of stiffness for all unbonded aggregate materials. Reduction in modulus values of more than 50% have been reported in the literature. The performance of bituminous and PCC surfaced roadways is directly related to the degree of saturation of the aggregate base materials. The moisture in the bituminous surface can lead to moisture-related damage, a reduction in modulus and a loss in tensile strength; whereas water in the PCC surface has only a slight effect in the modulus and strength of PCC, but it can have an effect on curling and warping stresses in the slab. In order to reduce or eliminate the

moisture-related damage and increase the overall pavement service life, permeable bases and/or edge drains have been incorporated into some designs.

3-3.01.02 TREATED BASE

Treated bases consist of a granular material to which a stabilizing agent has been added. The most commonly used stabilizing materials in Minnesota are asphalt cement used in permeable asphalt stabilized base and asphalt emulsion used in cold mix recycling.

The information that follows is intended to serve as a general introduction to the treatment of granular materials for use within the pavement structure.

1. Bituminous-treated Base. Aggregates may be treated with a bituminous product, resulting in a bituminous-treated base. This treatment can either be done "in-place" with existing materials or at a mix plant. The function of the bituminous treatment is to coat the granular particles with bituminous binder, enhancing the mixture strength and waterproofing characteristics.

   a. Cold Mix Recycling. This process includes the milling of the in-place pavement, adding asphalt emulsion, mixing, and placing, all in one operation. The mixture is then overlaid with a hot-mixed surface. This procedure provides a sound base for a plant-mixed pavement without the requirement of transporting the millings back to the plant.

   b. Permeable. As with granular bases, bituminous-treated bases may be either permeable or dense graded. When properly constructed, permeable, bituminous-stabilized bases have similar characteristics as permeable unstabilized bases, with the added advantage of being more stable and better able to withstand construction traffic. Permeabilities for drainable bituminous-treated bases are typically 1,000 feet per day, or more.

      Most often, asphalt cement (such as an AC-20) is used as the binder for drainable bituminous-treated bases. Typical quantities are from two to three percent asphalt cement (by weight) for a very open-graded aggregate. The intent is to provide just enough asphalt to coat the particles and bind them together without filling the voids.

   c. Bituminous-treated Base. This work consists of constructing a bituminous-treated base course on a prepared subgrade as defined in Mn/DOT's Specification 2204. This process is very seldom used and is typically substituted with hot plant-mixed base course in accordance with Mn/DOT's Specification 2331.

2. Cement, Lime, and Lime-Fly Ash Treatments. Aggregate bases may also be treated with a number of different cementitious products, including portland cement, lime, and lime-fly ash. With any of the treatment

materials, it is recommended that trial mixes be constructed with the stabilizing agent to make sure that the desired reaction will take place. Many of the reactions are highly dependent upon the nature of the fines and their mineralogy, and there is no other fail-safe means of determining whether a treatment method will work without producing the necessary trial mix designs and performing the appropriate tests.

a. Portland Cement. Portland cement can be used for the treatment of both fine-grained and coarse-grained materials to give a wide variety of cement-treated base mixtures that are used in pavement construction. Such mixtures include those that are comprised of soil and small amounts of cement, the resulting material being much more stable than the original material, although there are no specific quality requirements set. Cement-treated base mixtures also include mixtures of pulverized soil, cement, and water, compacted at the optimum moisture content to a high density, to derive materials with specific requirements of volume stability, freeze-thaw and wet-dry durability, strength, and impermeability.

There are two applicable specifications that govem the design and construction of cement-treated bases. Mn/DOT's Specification 2206 covers soil-cement bases, which are a combination of portland cement and either in-place soil or imported soil with the specified qualities. Some soil-cement bases have performed poorly in the past, due to the appearance of cracks in the asphaltic concrete pavement surface that have reflected through from shrinkage cracks in the base. Mn/DOT's Specification 2201 details the requirements for concrete bases. This is a higher quality base material that is typically mixed in concrete batch plants and placed with concrete placement machinery. This base has significant load carrying capacity, but the development of reflection cracks in the surface layer from cracks in the base course can still be a problem.

b. Lime. Lime can be used to modify fine-grained soils and the fine-grained portion of granular soils. It is most effective in the treatment of plastic clays, with a minimum clay content of 10 percent and a plasticity index (PI) greater than 10.

The effect of the addition of lime is dependent upon such factors as soil type, lime type, lime percentage, curing conditions (time and temperature), and the interactions between these variables. The immediate benefits of the lime treatment of soils include a decreased PI, improved workability, reduced volume change characteristics, and increased strength. These benefits are the result of immediate reactions between the lime and the soil (cation exchange followed by flocculation and agglomeration). Over time, the treated soil may also show enhanced strength and durability properties because of the continued pozzolanic reaction that is occurring between the lime and the soil. However, this may not always happen, as the reaction is heavily influenced by the soil conditions and mineralogical properties. Keep in mind that by

modifying the soil structure, the lime treatment may make the soil more frost susceptible. In some cases, relatively rough pavements have occurred over treatment areas. Mix design is based on the results that are desired, but generally suggest the addition of two to four percent lime. At present, the Department has no established specifications for lime-treated bases.

c. Lime-Fly Ash. Lime-fly ash may be used to treat fine-grained materials, especially silty soils, with the lime reacting with the fly ash in a pozzolanic reaction. The quantity of lime used in lime-fly ash mixtures ranges from two to eight percent, with 2-1/2 to four percent typical, while the fly ash quantity ranges from eight to 36 percent, with 10 to 15 percent typical. The Department does not have established specifications for lime-fly ash/soil mixtures, and they are not currently used in pavement construction in the state.

3. Chemical and Other Treatment Agents. Other agents have been used to treat pavement materials. These include calcium chloride and sodium chloride. They have seen some use as dust palliatives, but have not seen widespread use as treatment agents in base courses. Sulfur, rice-husk ash, and various forms of slags have also been known to be used, either by themselves or in combination with the other treatment agents discussed above, for treating highway materials. The Department does not currently use any such stabilizing agents.

3-3.01.03 PAVEMENT SURFACINGS

The pavement surface must be constructed to a higher standard than other layers in the pavement structure. Not only is the pavement surface subjected to direct loadings from the applied traffic, it is also subjected to greater wear from environmental factors as well. The surface layer is designed to provide a smooth riding surface, while maintaining adequate friction, and it must contribute to the overall pavement structure by protecting the underlying pavement layers. Because of its importance to the overall pavement structure, materials used in the construction of the surface are generally of higher quality. The materials primarily used for pavement surfacings are aggregates, asphalt mixtures, and portland cement concrete (PCC).

1. Aggregates. Aggregates may be used as a wearing course (placed directly on the subgrade) on some low-volume roads. Aggregates used for such purposes should conform to Mn/DOT's Specification 3138. A good aggregate material consists mainly of gravel and coarse sand with finer particles to fill the voids and a small portion of clay to act as a binder.

   The primary concern regarding the selection of the aggregate relates to the stability of the aggregate mix, which should be stable enough to resist shoving or rutting from the anticipated loadings. The effect of moisture on the layer must also be considered, as it is desirable that the surface shed water.

Other considerations for an aggregate-surfaced road include the requirement that it not contribute to excessive dust and that the surface provide a relatively smooth ride and cause a minimum of vehicle (especially tire) wear. It is also desirable that such a surface be easy to maintain. Additional information on the design and rehabilitation of aggregate-surfaced roads can be found in FHWA's *Surface Design and Rehabilitation Guidelines for Low-Volume Roads.* Design guidelines are provided in Section 5-5.0, Aggregate-Surfaced Road Design.

2. Bituminous. Section 5-693.100 of Mn/DOT's Bituminous Manual provides information on the materials used for bituminous surfaces. Information on sampling and testing methods that should be consulted for the design and construction of bituminous surfaces is also included.

A number of different types of bituminous mixes may be employed as a pavement surface. These are briefly described below.

a. Road-mixed Bituminous Surfacing. Mn/DOT's Specification 2321 defines the materials and procedures for road-mixed bituminous surfacing. The asphalts used should be either cutbacks or emulsions, and the aggregates should conform to Mn/DOT's Specification 3139. Anti-stripping agents, conforming to Mn/DOT's Specification 3161, may be used. This operation is very seldom used in today's construction practices.

b. Plant-mixed Bituminous Pavement. Mn/DOT's Specification 2331 covers the use of plant-mixed bituminous pavement and includes several different types. If the construction is performed under Mn/DOT's Specification 2340, "Plant-Mixed Bituminous Pavement -Quality Assurance," the same mixture designations are used.

The mixture designations are based on predicted traffic. Type 31, Type 41, Type 47 or Type 61 mixtures are used relative to increased traffic levels (ESAL), in that order (See Table 5-3.4). Higher numbered mixtures have a higher percentage of crushed aggregate plus tougher aggregate quality specifications. Mn/DOT's Specification 3139 lists gradation, crushing, and quality requirements; however, the gradation and crushing requirements may be modified to accomodate different traffic levels so as to eliminate the potential for rutting of the various bituminous mixtures. Recycled mixtures Type 32, Type 42 and Type 48 may be substituted for Type 31, Type 41, and Type 47 respectively, at the Contractor's discretion. Salvaged materials may be incorporated into recycled mixtures in accordance with Table 3-3.2.

Table 3-3.2. Allowable salvage material for recycling.

	Maximum Percentage Permitted					
	Type 32 Mixtures			Type 42 and 48 Mixtures		
Shoulder Material Types	Non Wear	Shoulder Wear	Shoulder Wear	Base	Binder & Level	Binder & Wear
Salvaged Aggregate	100	100	100	100	100	100
Salvaged Asphaltic Pavement	50	50	30	50	30	30
Crushed Concrete	50	0	0	0	0	0
Salvaged Asphaltic Pavement and Crushed Concrete	85*	0	0	0	0	0

* Neither component shall exceed 50 percent of the total aggregate, by weight.

c. Bituminous Seal Coat. Mn/DOT's Specification 2356 covers the materials and procedures for the proper placement of a bituminous seal coat over an in-service bituminous pavement. The process consists of the application of a bituminous material followed by the spreading of an aggregate. The following bituminous materials may be used for the seal coat:

- RC Liquid Asphalt RC-250, 800, 3000.
- Emulsified Asphalt
  - Anionic RS-1, RS-2, and
  - Cationic CRS-1, CRS-2.
- Asphalt Cement Penetration as specified.

Mostly, cationic emulsions are used.

There are many reasons to place a seal coat, but the primary benefits are as follows:

- Waterproofing the surface,
- Improvement in surface friction and drainage,
- Retarding pavement weathering, and
- Improvement in surface appearance.

It should be noted, however, that seal coats provide no structural contribution to the overall performance of the pavement.

The Department has instituted a Quality Management plan for bituminous pavements. Quality Management Projects Specification 2340 governs the construction of projects designated to fall under this Quality Assurance plan. Section 5-693.400 of Mn/DOT's Bituminous Manual provides the appropriate background information for the administration of this plan.

3. Portland Cement Concrete. Portland cement concrete (PCC) is a commonly used surfacing material, composed of a carefully controlled blend of portland cement, aggregates, and water. Concrete pavements are covered under Mn/DOT's Specification 2301.

Different types of cements and aggregates may be specified, depending on the anticipated use of the concrete. Several admixtures may also be used in the construction of concrete pavements to alter certain characteristics of the mix. Both batch plant and ready-mix concretes can be used for pavement construction, provided that they are in accordance with the requirements of Mn/DOT's Specification 2461.

Two types of PCC obtained from each of the sources are used for construction in the state. Type 1 concrete is produced by using Type I portland cement, and Type 3 air-entrained concrete is produced by using either Type IA air-entrained portland cement or Type I portland cement with an approved air-entraining agent added. Type 1 concrete is not used for pavements because of its lack of air entrainment. Additional information on the different types of cement used by the Department are given in Section 3-3.02.03. Mn/DOT's Concrete Manual provides a great deal of information on the materials, mix design, batching and mixing, testing, placing, and inspection of concrete used in pavement construction.

The concrete is placed on a prepared subbase course, which generally consists of Class 5 aggregate. The requirements for concrete pavement constructed in this manner are given in Mn/DOT's Specification 2301. It is the normal practice in the state to construct concrete pavements that are either jointed plain or jointed reinforced. Transverse joints are dowelled and sealed with an approved joint sealant.

3-3.02 ENGINEERING PROPERTIES

The materials discussed in the preceding section are used in pavement structures because they possess desirable engineering properties that contribute to the overall performance of pavements. These properties are discussed in this section, along with the test methods that are used to quantify them.

3-3.02.01 AGGREGATES

Controls are placed on the quality of aggregates used in both aggregate bases and surface courses. For the most part, the requirements are identical and the few differences that exist for aggregates exposed to traffic are discussed herein. In general, Mn/DOT's Specification 3138 covers all aspects of aggregate properties.

1. Base. The properties of aggregates used in base courses are specified in order to obtain the desired quality level for the use of that material. The significance of each of the relevant properties is discussed below, in relation to their overall contribution to the performance of the aggregate.

   a. Gradation. The gradation of the aggregate is important for several reasons. A dense-graded aggregate includes particles of many different sizes. It will generally compact in a tighter matrix, as the smaller particles fill the voids between larger particles. The result is a base course that has comparatively higher strength and is more impervious to moisture than a more open-graded layer. An open-graded material is used in order to promote drainage, so the large voids between aggregate particles is, in this case, actually desirable.

      Under Mn/DOT's Specification 3138, the aggregate classes shown in Table 3138-1 are generally considered dense graded aggregate materials. The Classes 5 and 6 aggregates are normally used as base course aggregates placed directly under the pavement surfacing (concrete and bituminous); whereas, Classes 3 and 4 are used as subbase course aggregates beneath the Classes 5 and 6 base aggregates. Classes 1 and 2 aggregates are normally used as surfacing aggregates for low-volume roads and shoulders.

      In order to promote pavement drainage, open-graded aggregates are sometimes used in place of or in conjunction with dense-graded aggregates. The gradations used or suggested for this type of application were discussed in previous sections of this chapter.

   b. Crushing. Crushing of aggregates may be necessary for two reasons. An obvious need exists if the top size of the aggregate is too large to meet gradation requirements. However, crushed aggregate is often stronger than a similar aggregate that is not crushed. Crushing will often break the aggregate along planes of weakness that may exist, resulting in smaller stronger pieces. Crushing aggregate also gives a rounded aggregate a more angular shape. This is desirable when greater surface to surface contact is needed. Such contact is typical of densely packed aggregates and will increase the strength of the overall layer or material in which they are used.

      Crushing is required for Classes 5 and 6 aggregates, and for Class 1 aggregate when it is to be used for shoulder surfacing. Class 5 aggregate should have no less than 10 percent crushed material and Class 6 aggregate should have no less than 15 percent crushed material.

   c. Moisture/Density. The relationship between moisture and density in an aggregate base is determined by the Proctor test (AASHTO T 99, Method C). The test method, described in Section 5-692.222 of Mn/DOT's Grading and Base Manual, is used to identify

the maximum density of the soil, which is the greatest dry density that can be obtained when the moisture content of the material is varied and the sample is subjected to a standard compactive effort. The optimum moisture content (OMC), the value of interest, is defined as the moisture content of the material corresponding to the maximum density. The relationship is of engineering interest because an aggregate layer with a moisture content lower than the OMC will require greater compactive effort to achieve the desired density; if the moisture content is much greater than the OMC, the layer is likely to be very unstable.

d. Permeability. The permeability of an aggregate mixture is defined as its capacity to conduct or discharge water under a given hydraulic gradient. The common measure is the coefficient of permeability, k, which is the rate at which water passes through a unit area of material in a given amount of time under a given hydraulic gradient. Permeability of a pavement course is often discussed in units of feet per day.

For a densely graded aggregate, the permeability should be such that the OMC of the material does not vary by more than a few percentage points over the life of the pavement. The goal is to maintain the moisture content at a fairly low level so that the aggregate material is not adversely affected. It should be noted that this is not an applicable concept for highly permeable granular materials, as their degree of permeability is such that moisture content should not have an effect on layer compaction.

There are two laboratory tests available for determining permeability. The constant head permeability test is performed in accordance with AASHTO T 215; and the falling head permeability test is performed in conjunction with the U.S. Army Corps of Engineers method, described in *Engineering Manual EM 1110-2-1906, Appendix VII.* Permeability may be measured in the field using the field permeability testing device, which gives permeability coefficients that have correlated well with laboratory values; or by a field percolation test, which is a subjective measure which only provides an indication of whether or not a material is permeable.

Permeable aggregate layers should have a coefficient of permeability of at least 300 feet per day, although a minimum value of 1,000 feet per day is desirable. The permeability of dense-graded materials varies widely according to the gradation, quantity, and type of fines. (The permeability of Class 5 base is about 0.4 fpd.)

e. Strength. Strength is the engineering property of greatest interest for an aggregate layer. However, strength alone is not sufficient to compensate for other deficiencies that may exist in the aggregate layer. Thus, there are many other engineering properties that, if inadequate, can have as detrimental an effect on overall

performance as can low strength. Some different parameters for measuring aggregate strength are given below.

(1) R Value. The R value is an important value in pavement design in Minnesota. Section 5-3.05.02, Bituminous Pavement Design, of this manual, provides recommendations on the determination of the R value. While it is possible to obtain an R value on a base material, the Department currently performs this test only on subgrade soils with more than 15 percent passing the No. 200 sieve. Typical values of granular materials will be above 70, with good repeatability of the test results. Materials with a higher percentage of fines and plastic materials will have much lower R values.

The test result is empirical and does not represent an intrinsic property of the aggregate sample.

(2) California Bearing Ratio. The California Bearing Ratio (CBR) is a widely used measure of the strength of granular materials. This test is also empirical in nature. CBR is a measure of the load required to produce a 0.1-inch penetration in a standard soil sample, and is actually a relative indication of the soil strength compared to that of a well-graded crushed stone aggregate. The CBR of a well-graded crushed stone aggregate will typically be close to 100, while the CBR of fine-grained highly plastic materials is often less than five.

The CBR test is not used by the Department for characterizing the strength of aggregate bases.

(3) Resilient Modulus. The resilient modulus (M_r) test was developed as a way of more accurately characterizing the strength-deformation characteristics of a pavement material that is subjected to repeated traffic loading. It is a measure of the elastic modulus property of a material and is gaining increasingly widespread acceptance because it is more closely identified with intrinsic material properties and is not an empirical test like the others. It should be pointed out that this test is not a strength test per se.

The advantages of using the M_r in a design process include the existence of a design procedure that uses the value (1986 AASHTO design procedure) and its applicability to mechanistic pavement design and multilayered elastic layer analysis. However, there is currently considerable debate regarding the conditions and procedures to be followed for the conduct of the test. A standard testing procedure for determination of the resilient modulus value is expected to be developed shortly.

(4) Triaxial Test. The triaxial test is a means of evaluating the shear strength of various materials. Standard samples are loaded in a compression cell at different confining pressures to produce a Mohr failure envelope, which provides the cohesion (c) and angle of internal friction (ϕ) of the material. The relative values of these parameters may be used to estimate the shear strength of the material.

While a few states have developed flexible pavement design procedures based on the results from the triaxial test, Minnesota does not currently use the triaxial test.

(5) Granular Equivalency. The granular equivalency (G.E.) factor is a means of equating the structural performance of an aggregate course, in terms of the structural performance of a high-quality aggregate base, such as a Class 6 aggregate base. As such, the G.E. is also an empirical approach to classifying the strength characteristics of an aggregate material.

The G.E. concept was developed in Mn/DOT's Investigation No. 183 and provides a value, in inches, of material required to obtain the same structural performance as the high-quality aggregate base. Section 5-3.05.02 Bituminous Pavement Design further describes the concept, as well as providing the G.E. of granular materials corresponding to some of the commonly used specifications found in Minnesota.

While the G.E. concept is only used in Minnesota, it has been related to AASHTO's structural coefficients for layers in a bituminous pavement. The structural coefficient is roughly equal to the G.E. value divided by seven. However, such correlations must be applied with great care, as the G.E. and AASHTO's layer coefficients are indicators of different empirical aggregate properties. While they can loosely be used as an indicator of "strength," they are not measurements of an inherent material strength property.

f. Quality. The quality of an aggregate base course is an indication of the material's ability to retain its desirable properties over time. Aggregates in Classes 1 through 6 should conform to Mn/DOT's Specification 3138, which includes several quality requirements. In addition, coarse and fine aggregates used in PCC must comply with Mn/DOT's Specifications 3137 and 3126. Graded aggregate for use in bituminous mixtures prepared by road-mixed or plant-mixed methods must conform to Mn/DOT's Specification 3139.

(1) Los Angeles Rattler Test. The Los Angeles (LA) Rattler test (AASHTO T 96) provides a means of measuring the

toughness and durability of an aggregate. In the test, a sample of aggregate is placed in a hollow steel drum with steel ball bearings and rotated for 500 revolutions. The aggregate's resistance to wear is determined by measuring the material that is broken off of the original aggregate samples.

(2) Shale. Shale is a poor quality material, which, when present in significant quantities, may cause performance problems. A test procedure to determine the presence of shale and other lightweight materials in an aggregate is described in Mn/DOT's Grading and Base Manual, Section 5-692.216.

(3) Soundness. A test specified for the coarse aggregate fractions of PCC and asphaltic concrete (Mn/DOT's Specifications 3137 and 3141, respectively) is the test for soundness (AASHTO T 103 or T 104). This test measures the aggregate's ability to resist freeze-thaw action. It provides an accelerated means of simulating repetitive freeze-thaw cycles and will indicate whether or not the aggregate has a tendency to break down under such action.

(4) Other. It should be noted that the other limitations placed on the aggregate, namely gradation, composition, and crushing, also contribute to the assurance of the overall quality of the aggregate. These limitations must be strictly observed to ensure maximum performance.

2. Surfacing. As noted in Section 3-3.01.03 above, aggregates used as a surface layer may have different requirements than those same materials used in a base course. The engineering properties of aggregate surfaces are discussed briefly below.

a. Aggregate Gradation. Mn/DOT's Specification 3138 provides gradation requirements for the six different classes of aggregates. Classes 1 and 2 are typically used for aggregate surfacing.

b. Quality. Aggregates used in a surface course should be of suitable quality to serve their intended purpose. The tests described in Section 3-3.02.01, for the LA Rattler and presence of shale, are applicable.

3-3.02.02 BITUMINOUS MIXTURES (VIRGIN AND RECYCLED)

The properties of bituminous materials used in pavements are discussed in this section. These include the bituminous materials themselves, the mixtures made with the bituminous materials, and the properties of the bituminous mixtures as used in different layers within the pavement system.

1. Bituminous Materials. Bituminous materials consist of a variety of products in different types and grades. Bituminous materials are discussed in Mn/DOT's Specification 3151.

Three broad types of bitumens are used in pavements - asphalt cement, conforming to AASHTO M 20; cutback asphalts (AASHTO M 81, M 82, and M 141); and emulsified asphalts (AASHTO M 140 and M 208, modified). Each type can be used in a variety of applications, and each has specific characteristics that make them suitable for different uses. Asphalt cements are most often used in surface mixes and bituminous mixtures that are mixed and placed hot. Cutbacks, which are asphalt cements that are diluted with a lighter grade petroleum product, are more commonly used in thin surfacings. Their final properties are achieved through the evaporation of the lighter fraction. Asphalt emulsions consist of asphalt particles suspended in water. They are also used in thinner cold-applied layers, and they achieve their final properties through the evaporation of the water.

a. Penetration. The penetration test is a common means of characterizing the stiffness of the asphalt at a standard temperature. In the test, the penetration of a standard needle, under a standard load, applied over a fixed time period, is measured for an asphalt cement sample at a standard temperature (AASHTO T 49).

A temperature of 77° F (25° C) and a load of 100 grams applied for five seconds are the standard conditions for the general test. The penetration, recorded in units of 0.1 millimeters, is reported as the penetration of the asphalt. The penetration requirements for asphalt cements, cutback asphalts, and emulsified asphalts used by the Department are given in Mn/DOT's Specification 3151.

b. Viscosity. The viscosity of bituminous materials is a property that is used to grade asphalts. The viscosity, which is a measure of the consistency of the bituminous materials, is determined at 140° F (60° C) and 275° F (135° C). The temperature of 140° F is representative of the highest temperature bituminous mixtures are expected to experience in service; while 275° F is representative of the temperature bituminous mixtures are kept at during mixing and placement to facilitate construction.

A capillary tube viscometer is used in the determination of the absolute viscosity of asphalt cement (AASHTO T 202) at 140° F, the kinematic viscosity of liquid asphalts at 140° F, and asphalt cement at 275° F (AASHTO T 201). The Department's viscosity requirements are given in Mn/DOT's Specification 3151.

c. Aging. The ability of bituminous materials to withstand aging and weathering effects is another important property. Although no test exists for specifically measuring the durability of bituminous

materials, the results of the Thin Film Oven Test (AASHTO T 179) best represents weathering effects.

The effect of heat and air on selected properties of a thin film of bituminous material is determined as a representation of their aging during the mixing process. Criteria on the allowable levels of change in the properties of the bituminous materials to signify acceptable materials accompany the standard tests described.

d. Ductility. The test for ductility of bituminous materials is a quasi-adhesion and quasi-cohesion test to determine how the material will stick to aggregates and resist break up, respectively. In this test, a molded specimen of the bituminous material at 77° F is stretched at a constant rate of two inches (50 millimeters) per minute until it breaks (AASHTO T 51). The length of the sample just before it breaks, is a measure of the ductility of the material. The ductility requirements for the bituminous material used in Minnesota are given in Mn/DOT's Specification 3151.

(NOTE: SHRP (Strategic Highway Research Program) is presently developing new test methods and testing equipment for evaluating and characterizing asphalt binders and mixture properties. In the future these tests most likely will replace or supplement the previously listed tests. SHRP is also writing a new performance based specification for both binder and the mixture.)

2. Bituminous Mixtures. The bituminous mixtures used for pavement construction by the Department are listed in Section 3-3.01. The engineering properties of these bituminous mixtures are discussed in this section.

a. Bituminous-treated Base. Bituminous-treated bases are constructed under Mn/DOT's Specification 2204, but are very seldom used.

b. Plant-mixed Bituminous Pavement. The various bituminous mixtures used above a prepared subgrade or base were introduced in Section 3-3.01.03. They can consist of a combination of a base course, leveling course, binder course, and a wearing course. The base course is placed in variable thicknesses. The total bituminous pavement thickness is a function of predicted traffic levels (ESALs) and soil support value (R value). The leveling course is a layer of variable thickness which is placed on the surface to correct any surface irregularities that may exist and bring it to proper grade and cross-section. The binder course is generally a 1-1/2- to 2-1/2 inches thick layer that is placed directly below the wearing course. The wearing course is the topmost layer and riding surface of the pavement. The aggregate gradation and typical property requirements for these bituminous mixture courses are presented in this section.

(1) Aggregate Gradation. The aggregate gradation requirement for bituminous mixtures depends on their use and the nature of preparation of the bituminous mixture in which they are incorporated. Gradation requirements are listed in Mn/DOT's Specification 3139.

(2) Important Properties. Bituminous base, leveling, binder, and wearing course mixtures have to meet certain minimum criteria in regards to certain key properties. The properties include stability, resistance to low temperature cracking, stiffness durability, fatigue resistance, impermeability, and workability. Although the requirements for these properties are not addressed directly, the Department is able to meet these standards through quality management, with sampling and testing to determine mixture properties. Tests can be conducted to determine such key variables as the asphalt content, percent air voids, percent voids in mineral aggregate, aggregate gradation, and in-place density of the compacted mixture. All of these have an impact on the performance of the bituminous pavement.

Stability and resistance to low-temperature cracking are of particular interest for bituminous surface courses. A bituminous pavement with good stability will resist rutting, shoving, and other surface distortion. A bituminous pavement also needs adequate resistance to low-temperature cracking. However, the achievement of these two desirable properties is often in conflict because a mixture may be made more stable by using a higher viscosity asphalt and designing it at a low asphalt content and high air voids, whereas a mixture with a resistance to low-temperature cracking may need to be designed with a lower viscosity asphalt, low air voids, and high asphalt content. A satisfactory compromise between the two properties must be obtained in order to control the resulting distress to an acceptable level.

(3) Friction. A very important requirement of the wearing course bituminous mixtures is that they provide a pavement surface with adequate friction so that the tire is able to maintain contact with the aggregate particles in the mixture and thus prevent skidding.

3-3.02.03 PORTLAND CEMENT MIXTURES

Portland cement mixtures are a major paving material used by the Department. The engineering properties of the portland cement and the portland cement concrete mixtures used by the Department are discussed in this section.

1. Cement. The Department uses various types of portland cement that conform to AASHTO M 85. The portland cement should also comply with

Mn/DOT's Specification 3101. In general, there are five types of portland cement available. Three of these have air-entraining subtypes, which brings the total number of distinct cement types to eight.

Type I cement is normal portland cement without any of the special qualities of the following types. A subtype, Type IA, is air-entrained cement used in place of normal portland cement when air entrainment is desired. Type II is moderate heat of hydration and moderate sulfate-resistant cement. Type IIA is the air-entrained subtype of Type II cement. Type III cement is for high early-strength concrete mixtures, and Type IIIA is its air entrained version. The last two types are Type IV, a low-heat of hydration cement; and Type V, a high-sulfate-resistant cement.

The Department generally uses Type I, normal, non-air-entrained, portland cement. The addition of an approved air-entraining agent is a common method of obtaining air-entrained cement.

Two procedures are followed by the Department to obtain high early strength portland cement mixtures. They are the addition of calcium chloride to the standard mix; and the addition of 30 percent more cement with a reduction of the amount of fine aggregate, while keeping the water and air contents the same, with or without the addition of calcium chloride. The details are outlined in Section 5-694.214 of Mn/DOT's Concrete Manual.

2. Concrete Mixtures. The use of portland cement to stabilize base materials and to produce portland cement concrete constitute the two major uses of portland cement in paving. The engineering properties of cement-treated mixtures and PCC are discussed in this section.

   a. Cement-treated Mixtures. The engineering properties of the cement-treated mixtures used in practice depend on their purpose. There are cement-treated mixtures that are nothing more than soils with a small amount of cement added to get a slightly hardened material that essentially still acts as a soil, albeit an improved one. Others are soil-cement mixtures that are compacted at their optimum moisture content and cured to obtain specific properties. The soil-cement base used by the Department is of the latter kind. However, apart from the requirement of compacting the soil-cement to at least 98 percent of its maximum density during placement, there are no other required properties set by the Department. The density is determined as described in Mn/DOT's Grading and Base Manual.

   b. Portland Cement Concrete Pavements. As indicated previously, PCC pavements in Minnesota are either non-reinforced or reinforced and generally constructed on a Class 5 aggregate base. Some important considerations in the design of portland cement concrete mixtures are discussed below.

(1) Aggregate Gradation. The aggregates in the PCC used for pavement construction make up between 70 and 80 percent of the total volume; consequently, it is important to ensure that the aggregates used are of the right quality and gradation.

Generally, aggregates from a wide variety of sources can be used for portland cement concrete, provided they are strong, clean, and free of friable materials and impurities, such as silt, clay, and organic material. The general requirements for the aggregates used for PCC in Minnesota can be found in Mn/DOT's Concrete Manual. The specific requirements that have to be met with respect to the type and quality of aggregates that can be used for PCC are given in Mn/DOT's Specifications 2301, 2461, 3126, and 3137. Carbonate aggregates may be susceptible to D-cracking (a common form of pavement distress) and use of such material must be carefully evaluated.

Aggregate gradation is the measure of the acceptable composition of aggregates required to yield strong concrete. The gradation requirements for the coarse and fine fractions of the aggregate for PCC are given in Mn/DOT's Specifications 2461 and 3137, and 3126, respectively. Further information can also be found in Mn/DOT's Concrete Manual.

(2) Admixtures. Admixtures are any materials, other than cement, water, and aggregate, that are added to the concrete mixture immediately before or during its mixing to modify the properties of the concrete. Mn/DOT's Specification 3113 identifies three classes of admixtures, described as follows:

- Class I. Accelerating, Retarding, and Water-reducing Admixtures.

  Type A. Water-reducing,
  Type B. Retarding,
  Type C. Accelerating,
  Type D. Water-reducing and retarding,
  Type E. Water-reducing and accelerating,
  Type F. Water-reducing and high-range, and
  Type G. Water-reducing, high-range, and retarding.

- Class II. Air-entraining Admixtures.

- Class III. Calcium Chloride.

In addition, fly ash may be added to portland cement concrete to, among other things, increase workability, reduce the heat of hydration, increase the ultimate strength, and increase the resistance of the mix to freeze-thaw, chloride, and sulfate damage. Mn/DOT's Specification 3115 governs the use of fly ash in portland cement concrete.

(3) Strength. Strength is undoubtedly a very important characteristic of the PCC used for pavement construction, and measures should be taken to ensure that the appropriate strength is obtained. The strength attained is dependent on a wide range of factors, chief among which are the composition of the concrete mix, the method of preparation of the mix, and the curing methods used.

There are two fundamental rules, which, when followed, will lead to high-strength concretes. The first is the achievement of the highest cement content in a unit volume of concrete for a given aggregate and with all other conditions remaining the same. The second is the attainment of the maximum density for a given unit volume of concrete. Implicit in these rules are the increase in the strength of concrete with a decrease in water/cement ratio and an increase in the cement/void ratio.

Compressive strength is the parameter used by the Department to control the strength of concrete. On the basis of compressive strength and the cement/void ratio, the concrete used by the Department is classified into eight grades. For the two types of concrete used by Mn/DOT - Types 1 and 3 - the minimum compressive strength requirements for specified cement/void ratios are given in Mn/DOT's Specification 2461. Further information on the strength requirements for concrete can be found in Mn/DOT's Concrete Manual.

(4) Quality. Strength is not the only important property of concrete. The workability, durability, volume change, air entrainment, and, as indicated earlier, the density of concrete have an enormous influence on good performance. The best possible concrete work can be obtained only if all of the properties are carefully monitored to ensure minimum standards are met. The specific details of the requirements with regards to workability, durability, volume change, air entrainment, and density can be found in Mn/DOT's Concrete Manual.

3-3.02.04 MISCELLANEOUS

As indicated in Section 3-3.01.02, lime and lime/fly ash are sometimes used as stabilizers in pavement construction. As in the case of cement-treated bases, the properties of the mixtures obtained depends on their intended use, which in turn determines how the mixtures are made.

Since Mn/DOT has no specification for lime and lime/fly ash soil mixtures, and does not use them in practice, there are no engineering property requirements to report for them.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), "Manual of Subsurface Investigation," 1988.

American Association of State Highway and Transportation Officials, "Guide for Design of Pavement Structures," 1986.

American Association of State Highway and Transportation Officials, Testing Standards, Washington, D.C.

American Geological Institute, Dictionary of Geological Terms, Revised Edition; Anchor Press/Doubleday, Garden City, New York, 1976.

American Geological Institute, Glossary of Geology, Third Addition, Bates and Jackson, Editors, Alexandria, VA, 1987.

Busch, R.M., Laboratory Manual in Physical Geology; AGI/NAGT, Second Edition, 1990.

Carpenter, S. H., "Highway Subdrainage Design by Microcomputer: DAMP—Drainage Analysis and Modeling Programs," Federal Highway Administration, FHWA-IP-90-012, August 1990.

Church, Horace K., Excavation Handbook, McGraw-Hill Book Co., New York, 1981

Deere, Don U., "Rock Quality Designation (RQD) after 20 Years, U.S. Army Corps of Engineers, Waterways Experiment Station, Contract Report No. GL-89-1, 1989.

Federal Highway Administration, FHWA-TS-80-224, "Highway Subdrainage Design", August 1980.

Federal Highway Administration, "Technical Guide Paper on Subsurface Pavement Drainage," Technical Paper 90-01, October 1990.

Flint, Richard F., Glacial and Quaternary Geology; John Wiley and Sons, Inc., New York, 1971.

Folk, Robert L., Petrology of Sedimentary Rocks, Hemphill Publishing, Austin Texas, 1980.

Halstead, W. J., "Use of Fly Ash in Concrete," National Cooperative Highway Research Program, Synthesis of Highway Practice 127, October 1986.

Hobbs, Howard C. and Joseph E. Goebel, Geologic Map of Minnesota, Quaternary Geology, Minnesota Geologic Survey, Map S-1, 1982.

Hobbs, Bruce E., Winthrop D. Means, and Paul F. Williams, An Outline of Structural Geology, John Wiley and Sons, Inc., New York, 1976.

Hudson, S. W., B. Frank McCullough, and R. Frank Carmichael III, "Surface Design and Rehabilitation Guidelines for Low-Volume Roads — Final Report," Federal Highway Administration, FHWA/TS-87-225, June 1987.

Hundman, Donald W., Petrology of Igneous and Metamorphic Rocks, International Series in the Earth and Planetary Sciences, McGraw-Hill Book Co., New York, 1972.

Hunt, Roy E., Geotechnical Engineering Investigation Manual, McGraw-Hill Book Co., New York, 1984.

International Society of Rock Mechanics, "Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses", International Journal of Rock Mechanics and Mining Sciences, Vol. 15, pp.319-368, 1978.

Jackson, Kern C., Textbook of Lithology; McGraw - Hill, 1970.

Kersten, Highway Research Board Special Report 2, "Frost Action in Soils," a symposium, 1952.

Mathis, D. M. "Permeable Base Design and Construction, Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, 1989.

Mossler, John H., Paleozoic Lithostratigraphic Nomenclature for Minnesota; Report of Invesitgations 36, Minnesota Geological Survey, St. Paul, 1987.

Ojakangas, Richard W. and Charles L. Matsch, Minnesota's Geology, University of Minnesota Press, 1982.

Oregon Department of Transportation, Soil and Rock Classification Manual, 1987.

Schwartz, George M., and George A. Thiel, Minnesota's Rocks and Waters: A Geological Story, Minnesota Geological Survey, Bulletin No. 37, University of Minnesota Press, 1963.

Sharp, Robert P., Living Glaciers: Understanding Glaciers and Glaciation, Press Syndicate, University of Cambridge, 1988.

Sims, P.K. and G.B. Morey, editors, Geology of Minnesota: A Centennial Volume, Minnesota Geologic Survey, 1973.

Smith, K. D., D. G. Peshkin, M. I. Darter, A. L. Mueller, and S. H. Carpenter, "Performance of Jointed Concrete Pavements, Volume I—Evaluation of Concrete Pavement Performance and Design Features," Federal Highway Administration, FHWA-RD-89-136, March 1990.

Transportation Research Board, "Lime Stabilization," State of the Art Report 5, 1987.

United States Army Corps of Engineers, Engineering Manual 1110-345-306, "Engineering and Design, Present Design for Frost Conditions."

United States Army Corps of Engineers, "Appendix VII: Permeability Tests," Laboratory Soils Testing, Engineering Manual EM1110-2-1906, November 1970.

United States Army Corps of Engineers, Waterways Experiment Station, Unified Soil Classification System, TM 3-357, 1953.

Williamson, D.A., "Uniform Rock Classification for Geotechnical Engineering Purposes", Transportation Research Record No. 783, Transportation Research Board, pp.9-14.2, 1980.

Chapter Four
INVESTIGATION

4-1.0 GENERAL

The investigation of subsurface conditions along a new highway alignment; and in the case of repair or maintenance projects, surficial and subsurface conditions along existing highways, is called a survey. The survey may be preliminary, such as a corridor study (although these are seldom performed any more, since most highway corridors have long since been established) or for pavement selection; or it may be more elaborate, such as the more frequently performed geotechnical and pavement surveys described in Sections 4-2.0 and 4-3.0, respectively. Surveys will be performed for all projects, except for routine or minor maintenance. The detail of the survey obviously depends on the complexity of the project.

4-1.01 PURPOSE OF SURVEYS

The purpose of any of the surveys performed by the Department is to define the in-situ soil, rock, ground water, and existing pavement/subgrade conditions, including the material types and their engineering character, to the extent necessary for the design of the most economical, uniform, and stable roadway.

4-1.02 TYPES OF SURVEYS

4-1.02.01 PRELIMINARY

Preliminary surveys are performed for either the selection of a route within a corridor and in preparation of an EIS for the project or to provide sufficient information for selection of a pavement type for cost studies after the route has been selected. The District Soils Engineer is responsible for scheduling these surveys.

1. Corridor Study. For the purpose of selecting a route within a corridor, some borings at critical locations may be desirable, but are not required as a matter of routine. Available information on such physical features as rock outcrops, swamps, and other items which might restrict the roadway alignment should be included. Generally, high altitude photos or one to 24,000 USGS Quad sheets are of adequate detail for reference. There is not much need for this type of survey at the present time.

2. Pavement Selection. As part of the pavement type selection process, borings generally five to 10 feet deep and on 500- to 1,000-foot centers, are performed. The frequency of the drilling depends upon the variability of the subgrade and the encountered terrain. Where such conditions are constant, the larger spacing recommended above could be used; on the other hand, where expected or encountered conditions are highly variable or erratic, the shorter spacing recommended above should be used. In some locations, even a more detailed drilling program may be justified. Such problem areas could include swamp crossings, deep cuts

or fills, or areas of observed seepage. Reference maps, and the accuracy of boring locations, will vary with this type of survey, from one to 24,000 USGS

Quad sheets to site maps with scales on the order of one inch equals 200 feet and five-foot contour intervals, as available. Aggregate sources should be identified as a part of the pavement selection survey.

4-1.02.02 FOUNDATION

Data for the design of bridges, large culverts (more than approximately 80 square feet), retaining walls, and buildings are obtained during a Foundation Survey. The Foundations Unit (Office of Materials Research & Engineering) is responsible for scheduling these surveys and will provide all drilling and sampling, laboratory testing, design, and recommendations. In addition, the Foundations Unit will provide piezometer and slope inclinometer installations in deep cut or fill sections, where detailed stability analyses need to be performed. The District Soils Engineer may also be involved in these studies, reading the piezometers and slope inclinometers, and performing other data gathering to assist the Foundations Unit.

4-1.02.03 GEOTECHNICAL AND PAVEMENT

Soil, rock, and ground water information and recommendations for grading, base, and surfacing are included in the geotechnical and pavement surveys. Knowing the proposed grade of the roadway, borings are spaced at intervals of approximately 100 feet, depending on expected and encountered conditions. The borings should extend to depths of at least five feet below the proposed grade in cuts; and five feet below the existing grade in fills, with a minimum of one boring per fill section extending down to a depth equivalent to the proposed fill height. The maps used for these surveys should be adequate to locate the borings accurately to 5± feet horizontally and 0.5± feet vertically. Borings should never be terminated in unusually soft or unsuitable soils or at a level above the final frost penetration depth. If such unusually soft or unsuitable soils are encountered, the Foundations Unit (Office of Materials Research & Engineering) should be notified so that they may schedule additional borings, sampling, and testing, as required.

Questions regarding bedrock (auger refusals, rock type, backslope design, excavation characteristics, etc.) and groundwater can be directed to the Geology Unit (Office of Materials Research & Engineering).

Information for these surveys can be obtained by destructive and non-destructive testing (NDT) means.

1. Destructive Methods. Destructive methods include borings, as discussed above; pavement cores; and test pits.

    a. Borings. Machine-advanced flight auger borings are used to obtain soil information. This equipment is maintained and operated by district-level personnel. In places inaccessible to the

drill rig, hand augers may be used. Obviously, it is best to use the drill rig wherever possible, because it is much easier to use and normally provides more consistent results. The encountered materials are visually described (Section 3-2.01.01), and disturbed grab samples are taken. These samples are taken either from the cuttings as the hole is advanced or, preferably, from the auger as it is withdrawn from the hole (Section 4-2.04.02). Following completion of the boring, the hole should be backfilled in accordance with current Department guidelines.

b. Pavement Cores. Cores cut through existing pavements can be used to measure the pavement and base-course thicknesses and, in the case of bituminous pavements, the relative amount of stripping can be observed. The coring operation also furnishes material samples for laboratory analysis. For example, the gradation of non-stabilized granular bases can be determined, as can moisture properties, if the base is not overly contaminated with coring water. A visual examination of a core taken at a PCC joint or crack will also provide information about the condition of the joint and will reveal any deterioration that has started at the joint or crack. Many other material properties of the surface layer (either portland cement concrete or asphalt) can be determined from laboratory tests performed on the extracted cores. The core holes are typically backfilled with either a hand-tamped cold mix patch or hand-batched concrete.

c. Test Pits. In areas that are inaccessible to the drill rig, backhoe-dug test pits may be used for shallow soil exploration. Test pits may also be used in pavement areas, in lieu of cores, to obtain a larger sample of the base and subbase aggregates. This is typically done for a study of the potential reuse of these materials or for other in-depth studies. The excavated test pit should be backfilled with the cuttings or other suitable fill material, compacted in relatively thin lifts; and, in the case of test pits dug through existing pavements, the roadway should be properly patched.

2. Nondestructive Methods. For exploration of pavements and subgrades, NDT methods may also be used. These include radar equipment, such as the ground penetrating type (GPR); static and dynamic deflection measuring devices, which include Falling Weight Deflectometers (FWD); and geophysics (for soil and rock evaluation), which include seismic and resistivity. In addition, a number of more specialized devices have been used in pavement applications. These include the Transient Dynamic Response (TDR) device, which has been used for detecting subsurface voids; the half-cell potential survey, to detect corrosion in reinforced pavements; spectral analysis of surface waves (SASW), which has been used to characterize the strength and thickness of surface layers; and wave propagation, which has also been used to identify layer properties.

a. Radar Equipment. Several companies market services that use a radar to identify subsurface properties. The principle is that a short radar pulse transmitted through a material and reflected back to a receiver can identify and locate material property changes that occur at layer interfaces. This equipment, mounted in vans for example, is able to survey pavements at speeds of up to 40 miles per hour. The data can then be analyzed for layer thicknesses, the presence of voids, and the presence of subsurface moisture. Ithas also been used to locate buried objects, such as utilities. Mn/DOT does not currently own this type of equipment.

b. Deflection Equipment. Deflection measuring equipment has been in use for many years. Originally, static deflection devices, such as the Benkelman beam, were used to measure the response of a pavement to a known applied load. Dynamic deflection devices have received more widespread use in recent years, as they have been found to more accurately and rapidly simulate the moving loads applied to a pavement. The responses from deflection measuring equipment have been used to estimate layer strengths and subgrade moduli for both asphalt concrete and portland cement concrete (PCC) pavements. In addition, in PCC pavements, the responses have been used to identify the location of voids at joints and assess the degree of load transfer across joints. More discussion on this topic is found in Section 4-3.0.

c. Geophysics. Of the several geophysical methods available for exploring subgrade materials, resistivity and seismic surveys are used by the Geology Unit. The resistivity method uses electrical currents to determine subsurface material characteristics, while seismic refraction/reflection uses seismic waves to measure vertical and lateral changes in soil and rock materials. Geophysical methods are non-destructive tests that are particularly useful when a drill rig cannot be utilized, e.g., in areas of rough terrain, crop lands (or other non-accessible areas), or where auger drilling meets "refusal" and it is uncertain whether the refusal is on a boulder layer or on bedrock.

d. Non-contact Evaluation Equipment. There are a number of different devices on the market and under development that have the ability to provide very useful survey information about a pavement without actually coming in contact with the pavement. Foremost in this group are a number of devices that obtain photographic records of the pavement surface that can be interpreted, either manually or automatically, to obtain distress information. Devices also exist that can obtain measurements of ruts, and of cross-slope and longitudinal variation of a pavement. All of the equipment in this grouping have the ability to collect data at or near traffic speeds.

e. Other Equipment. As noted above, other equipment exists which may be of interest in the evaluation of subsurface conditions. This equipment, which has been either sparsely used or is still under development, includes a device that can measure the presence of active corrosion in subsurface reinforcing, devices that measure the surface propagation of waves, and devices that measure the TDR of a pavement's surface to an applied load. These devices are also intended to identify material properties of relatively homogeneous pavement layers.

f. New Developments. Research is currently being funded to develop NDT equipment that will further the ability to rapidly and inexpensively evaluate the structural condition of pavements. This includes the ability to locate cracks in pavements and the presence of subsurface moisture. There is also a large amount of funding being devoted to furthering the capabilities of the non-contact evaluation equipment.

It is anticipated that the future will see increasing use of automated survey equipment that can collect data without traffic control.

4-1.03 ACCESS AUTHORIZATION

Before driving onto the site, it must be verified that authorization to enter the property has been granted by the owner and that leasers of the property, and others who might use the property regularly, have knowledge of such access authorization. The authorization to drive onto the site and drill borings should be obtained in writing prior to mobilization, for all locations except those on State-owned right-of-way and where very minor and non-controversial encroachment is required.

4-1.04 UTILITIES

Utilities, including gas, electric, water, sewer, and telephone, must be located prior to drilling. In lieu of contacting each utility company directly, the "Gopher State One Call" program, a centralized utility location service, must be used. To use this service, the exact project location and limits must be phoned in (St. Paul/Minneapolis area, 612-454-0002; other areas, 1-800-252-1166) at least 48 hours in advance. Whenever possible, and certainly in locations where numerous or critical utilities are located, it is still recommended that the Drilling Foreman meet utility representatives at the site for an orientation.

4-1.05 SAFETY

Each and every job we do, no matter how small or large, no matter how important that it be done quickly, will be more successful if it is accomplished in an environment of safety. When safety comes to mind, we immediately think of drilling operations, and that is the main thrust of this section, but we must also include other office and field-related job activities, travel between the job and

home, and the variety of personal activities with which we are involved. Safety is not just the responsibility of one person, but is everyone's job.

Most accidents can be prevented. The use of common sense, application of learned safety practices, and proper training will help achieve a safe drilling environment. Your health and safety, and that of your coworkers, depends on how well you practice safety. Each employee is responsible for their own personal safety, the safety of their fellow workers, and for using available safety devices prescribed for accident prevention. Refer to Mn/DOT's Safety and Health Guidelines for more complete coverage of safety issues.

4-1.05.01 TRAINING

All workers must be trained so that they are aware of their job duties and requirements, and safe working practices. This is particularly important with new employees. A new worker cannot be expected to be immediately familiar with all of the dangers or hazards associated with doing a job.

It is the Drilling Foreman's responsibility to take charge - in knowing, observing, and enforcing safe working practices. The new worker learns from the Drilling Foreman's leadership - both informally, through observation of the proper use and maintenance of equipment and protection devices; and formally, through participation in on-the-job safety meetings and other instructions on safety. Part of this training is applying the following steps:

- Tell the new worker what must be done.
- Warn of any known dangers or hazards.
- Show how to do the job in a safe manner.
- Watch closely as the work is done, to see if appropriate safety precautions are applied.
- Don't allow new employees to work alone until there is certainty of their ability to do so safely.

Whenever possible, the Drilling Foreman and coworkers should work in pairs or groups, continuously observing and improving each other's safety practices. This
approach is also important in the event that an accident resulting in personal injury or incapacitation of one of the workers occurs.

4-1.05.02 PERSONAL GEAR

Each employee is responsible for obtaining, using, and maintaining their own personal protective gear. In general, the requirements for this equipment are that it be comfortable and not unduly restrict the worker's ability to perform their job, while, at the same time, providing significant protection from a danger or hazard. The following gear is included.

1. Clothing and Footwear. Obviously the type of work being performed and prevailing conditions dictate the appropriate clothing. However, some generalizations can be made. Clothing should be seasonal to provide protection from the elements. Shirts and full-length pants must be worn, even in the summer months, because of the protection they provide from sun, weeds, and insects. Clothing and accessories, including jewelry, should not be loose or baggy, as such have a tendency to get uncontrollably caught in rotating or moving parts of drilling equipment and other machinery. Heavy cloth or leather gloves should be worn to protect

   the hands from cuts and abrasions from burrs and sharp edges during the handling of drilling tools, equipment, and supplies. Steel-toed shoes or boots, to protect the feet from injury due to falling objects, are required at all times for field crews engaged in drilling or support activities.

2. Reflectorized Safety Vests. Whenever an employee is working on the right-of-way, in a construction or a maintenance work area, or on the roadway where public traffic or other vehicles are moving, the employee is required to wear a reflectorized safety vest. These are of great value in increasing visibility to other vehicle operators. Reflectorized safety vests are available through the district stock rooms. They should be available on the drill rig and used by each and every member of the crew.

3. Safety Helmets or Hard Hats. Whenever drilling operations are underway, or when any other overhead dangers or hazards exist, hard hats should be worn. This includes at all times on the work site - during loading and unloading, setting up, tearing down, and when drilling. Headband and crown straps should be adjusted for a comfortable fit to encourage full-time use of the hard hat. Hard hats can also be fitted with a cotton liner to provide additional protection to the head and ears during cold weather. Hard hats and liners are available for each employee, through the district stock room.

4. Safety Glasses or Goggles. The rule to remember is that if a hard hat is required, then safety glasses or goggles should also be worn. They are required at all times on the work site - during loading and unloading, setting up, tearing down, and when drilling. They should also be worn to protect the eyes at any other time when there is a danger of flying dust or debris, such as when cleaning the equipment or chipping ice. Prescription safety glasses or plain-glass goggles are available for each employee, through the district stock room.

4-1.05.03 VEHICLES AND TOOLS

Vehicles and tools can be used safely, provided they are used with common sense, that learned safety precautions are applied, and that they are periodically inspected and maintained in a safe working condition. With regard to vehicular safety, all of the generally learned precautions apply. For example, when drilling or support equipment are being moved, care should be exercised to keep from

getting in the way, particularly when the vehicles are backing up and the driver's vision is obstructed. When traveling in the vehicles, safety seat and shoulder belts must be worn at all times.

As a part of the Department's approach to safety and also to increase the life and usability of the Department's equipment, each driver is responsible for maintenance, including proper operation, care, and appearance. As a minimum, the following care of the vehicles is required:

- Correct lubrication;
- Checking and maintaining oil levels;
- Proper care of tires;
- Checking and keeping the cooling system, fan belts, battery, and aircleaner in good working order;
- Checking for and tightening loose bolts and parts; and
- Cleanliness and appearance.

Equipment is to be serviced whenever practical and as consistently as possible, at the District Maintenance Shop.

4-1.05.04 SITE HOUSEKEEPING

The general cleanliness of a site greatly contributes to safe operation. The site and equipment should be kept neat and orderly, with all tools and supplies assigned a safe and accessible storage spot. Work areas should be free of excess mud, ice, spilled lubricants, or other debris which could cause workers to slip, trip, or otherwise injure themselves.

4-1.05.05 NATURAL OR MAN-MADE HAZARDS

There are many dangers or hazards which can affect the safe operation of drilling equipment. Some of these are discussed below.

1. Utility Lines. Both overhead and underground utilities can be hazards to a safe drilling operation. Telephone and telegraph lines, video (cable), and electrical transmission lines may be located overhead. The minimum safe distance between the drill's mast and telephone and telegraph lines is 15 feet. The minimum safe distance between the drill's mast and electric transmission lines or video (cable) is dependent on the power being carried in the line; however, in the absence of other specific information, a distance of at least 35 feet should be maintained. Extra care must be taken when drilling around overhead utilities so that the workers do not become complacent relative to the presence of the utilities. It is surprisingly easy to forget, after a long hard day of drilling, that the utility lines are still there.

Underground utility lines include water, sewer, telephone and telegraph, video (cable), gas, and electric; and can be particularly dangerous to the safety of the crew and equipment, because they can't be seen. See Section 4-1.04 for instructions on using the "Gopher State One" program to locate these hazards. All probing beneath the earth's surface should be done a minimum of 10 feet beyond the identified location of these utilities.

2. Electrical Storms. To prevent injury from lightning strikes, drilling operations should not be done when there is the presence of or a threat of an electrical storm. Whenever possible, the mast should be lowered until the storm passes. The drill crew should move away from the drill equipment and into a safe area.

3. Inclement Weather. During rain, snow, severe heat or cold - when it is just too unbearable or impractical to drill - other activities should be conducted. In extremely hot weather, the possibility of heat stress or heat stroke exists. In extremely cold weather, there is the danger of frostbite or hypothermia.

   During cold weather, a salamander or L.P. gas heater may be used, adding to the safety precautions that must be exercised. With either type of heater, special care must be taken in transport, setup, and fueling.

4. Over Water or Ice. When working over water, such as in a swamp crossing or on ice, each member of the crew should wear a life vest or preserver.

   Before moving any equipment over or setting up on ice be sure that the ice thickness and condition are adequate for support. The Department's experience indicates that a minimum of 16 inches of good ice is required to support the average loaded truck. If the ice is soft, a minimum of 24 inches of ice would be required. If the ice is honeycombed or has undercurrents, no attempt should be made to traverse the ice with equipment. The Drilling Foreman is responsible for the safety of such movements over ice.

5. Toxic or Hazardous Substances. If an area is suspected of being contaminated with toxic or hazardous substances, the District Soils Engineer should not attempt drilling. Instead, a request for such drilling should be directed to the Environmental Services Section (Office of Technical Support). They will then schedule the drilling, which will be done by the Foundations Unit (Office of Materials Research & Engineering); or a consultant will be hired, depending on the current work load and/or anticipated hazard level.

   If a previously unidentified toxic or hazardous substance is encountered during what was thought to be a routine exploration, the drilling should be immediately stopped and the Environmental Services Unit contacted for instructions.

4-1.05.06 TRAFFIC CONTROL

Extra care must be taken when working, or when parking equipment, on the shoulder of or in an existing roadway. It is the Drilling Foreman's responsibility to implement the use of flagmen, barricades, warning lights, arrow boards, message boards, and other safety precautions; and to coordinate their use with the local authorities, state, county, or city. Particular consideration should be given to the safety of the traveling public, as well as to the drill crew.

Work which affects vehicular or pedestrian traffic should be provided with traffic control that is in accordance with the current edition of Appendix B, "Traffic Control for Short Term Street or Highway Work Zones." The Appendix is a supplement to Part VI of the Minnesota Manual on Uniform Traffic Control Devices (MMUTCD). This Appendix is intended to show application of the basic principles to be observed when performing short term work on any street or highway.

4-2.0 GEOTECHNICAL SURVEY

4-2.01 PURPOSE

The purpose of the geotechnical survey is to determine the nature and physical properties of the foundation soils, identify construction concerns, and locate sources of construction materials for input into the design of a roadway.

4-2.02 OFFICE STUDIES

The initial steps in completing a geotechnical survey are done in the office, prior to going into the field. First, the project is classified, e.g., as a thin overlay, resurface, recondition, reconstruction, major reconstruction, or new construction project. This will provide an indication of the extent and complexity of the required Design Recommendations Report, with new construction obviously requiring the most study. Next, a review of already available information needs to be made. Anticipated geologic conditions in the project area, discussions with others relative to personal experience in the area, and review of construction history for the existing or nearby projects are included in this review. Finally, after reviewing all of the following documents and information, planning of the field operations can begin.

4-2.02.01 PRELIMINARY PLANS

The proposed route and grade are a part of the preliminary plans. By studying these plans, many of the conditions that could potentially cause problems, and which should be planned for, can be identified. Included are swamp crossings, major cut and fill sections, and the presence of existing buildings or features. District Soils Engineers should be included in early project scoping and preliminary location/layout stages so significant obstacles and problems such as slides, unstable ground, groundwater, rock, etc. can be avoided or planned for.

4-2.02.02 MAPS

Any available maps will be useful in determining the extent to which construction will influence or be influenced by the physical site conditions. Particularly useful are topographic and geologic maps, which can be used as guides in planning the investigation and defining areas of concern for the site reconnaissance. Agricultural maps are likely to be of the least value, because they typically deal only with the surficial deposits and do not provide much insight to the engineering properties of encountered materials.

4-2.02.03 PREVIOUS WORK

Studies and construction plans completed for the existing or nearby projects can be useful in identifying problem areas. In particular, previous investigations (Design Recommendations Reports) and existing construction records (including the Construction Log), which give a history of the roadway, should be useful in planning the investigation.

4-2.02.04 MAINTENANCE INPUT

A valuable, but often overlooked source of information is the past maintenance records. An interview with the Maintenance Supervisor and review of maintenance documents can provide site information/concerns, including past performance, history, and frequency and type of repairs.

4-2.02.05 ENVIRONMENTAL CONCERNS

Any environmental-related information, relative to what might be encountered during the geotechnical survey and its impact on design, should be reviewed.

Included are the possible presence of old underground storage tanks, hazardous or toxic spills, etc.

4-2.02.06 AGGREGATE SOURCES

There are several aids available to identify potential new and existing sources, characteristics, and the extent of aggregate deposits. Gravel pit condition surveys are regularly performed at the district level and filed with the Aggregate Unit (Office of Materials Research & Engineering). This unit maintains a statewide master file of aggregate source information consisting of aggregate source maps ("pit sheets") and a computer file known as the Aggregate Source Information System (ASIS). It is described in detail, along with information on leasing procedures, in the following paragraphs.

The District Soils Engineer, in cooperation with the Aggregate Unit, should identify new and existing potential aggregate sources to determine their characteristics and extent. New sources are routinely located and tested, and tested areas of existing sources may be already extended by the Aggregate Unit. The Aggregate Unit's Geologist routinely conducts prospecting statewide, using aerial photo interpretation and geologic field reconnaissance. This activity often results in the initial contact with the property owner. The "Gravel Exploration and Leasing" information sheet is then given to each property owner who is contacted for preliminary testing. This form (which is shown in Appendix A1) is available to the district and may be used in a similar manner by the District Soils Engineer.

Bedrock quarries as sources of aggregate material are always privately owned and never leased by the State. Information on quarries and rock quality can most easily be obtained from the Geology Unit (Office of Materials Research & Engineering).

New aggregate prospects, as well as existing aggregate sources which require additional or updated information, are drilled and sampled by the Aggregate Unit, using their 10-inch diameter auger rig. Preliminary testing and sampling can often be done by the district. To improve gradation results, it is recommended that six-inch, or larger, diameter augers be used, instead of the "standard" 4-1/2-inch diameter augers. The Aggregate Unit is ideally equipped for both preliminary and final testing of aggregate sources. Requests for such assistance should be addressed to the Aggregate Engineer.

If gradation and quality results from the preliminary testing are acceptable, either the District Soils Engineer or the Aggregate Engineer should submit the "Aggregate Source Leasing Request" to the District Right-of-Way Office for possible leasing. This form (which is shown in Appendix A2) requests that both the District Soils Engineer and the Aggregate Engineer be notified when action (leasing of an aggregate source or unsuccessful lease attempt) has been taken by the Right-of-Way Office. Most leases negotiated with property owners are written on Mn/DOT Form 25681, entitled "Non-exclusive Material Pit Lease." This form is shown in Appendix A3. Such a non-exclusive lease does not require a lump sum payment by Mn/DOT, nor does it prevent the owner from selling aggregate to others while the Department's lease is in effect. This type of lease is usually more acceptable to the property owner than an exclusive lease, which reserves the materials for the exclusive use of the leaseholder, e.g., Mn/DOT, during the term of the lease. On the other hand, almost all private road contractors obtain exclusive leases from property owners, thus ensuring a known supply of aggregate for their own use.

After an aggregate lease has been obtained, more thorough sampling using the Aggregate Unit's 10-inch diameter auger is scheduled. Some coarse deposits may require trenching or other exploration methods. Generally, the spacing of the borings should be on a 100±-foot grid, with closer spacing in erratic deposits or possibly greater spacing in uniform deposits. A sample representing the material should be taken from each boring at each substantial change in the appearance of the material, with at least one sample every 10 feet. Where the material extends below the water table, separate above and below water table samples should be taken and the water level recorded. Samples may be tested at either the district or Office of Materials Research & Engineering Laboratory. As with all augured borings, samples below about 40 feet are not considered to reliably represent the in-situ materials.

After completion of this testing, the "Request for Gravel Pit Survey" (which is shown in Appendix A4) is initiated by either the District Soils Engineer or the Aggregate Engineer and forwarded to the District Survey Office. Reference is made to Section 5-5.10, "Pits," and Section 5-6.08, "Gravel Pit Sheets," in Mn/DOT's Surveying and Mapping Manual for a detailed explanation of both pit surveys and the resulting "pit sheets" or maps. These are subsequently produced cooperatively by both district and Aggregate Unit personnel.

Most aggregate source surveys are now done by electronic methods, and the resultant pit sheets are products of modern CAD system equipment which is operated by both district and Aggregate Unit personnel. The Aggregate Unit generates final pit sheets and condition survey updates on a CAD workstation, and the resulting information may be networked to the appropriate districts. Hard copies of new and revised pit sheets are also mailed to district offices on a routine basis. Following completion of a pit sheet, an analysis of the deposit and quality and quantity computations are made by the Aggregate Unit and entered into ASIS, a computerized data base available to all Department offices.

Mn/DOT developed ASIS for the purpose of storing and retrieving aggregate source information. It is implemented in METAFILE on an IBM-PC microcomputer. It consists of a master file of some 7,000 Mn/DOT-owned and

leased and commercial aggregate sources (gravel pits) and rock quarries (separate from gravel pits, which are never referred to as quarries). A unique identifier, called a "source number" (replacing the old "gravel pit number") is assigned by the Right-of-Way Property Management Unit (Office of Right-of-Way and Surveys) to each source. Each record contains data on location, status, owner/operator, date of lease expiration, date of last revision, date of last condition survey, and quality and quantity information. Several types of output reports have been predefined in METAFILE, and user access is through a series of menus for both update and retrieval. The majority of sources identified in the ASIS database represent privately owned aggregate deposits which have, at one time or another, been leased by the Department, but which may or may not be in actual use at any given time. Also, most of these numbered sources, irrespective of those identified as quarries or commercial sources, were assigned a source number at the time that Mn/DOT's Right-of-Way personnel first negotiated a lease.

Two versions of ASIS are currently being supported - one used by the Aggregate Unit and the other available to district personnel and other users. The version used by the Aggregate Unit contains aggregate source information for the entire state, as well as selected out-of-state pits. The district database includes all sources within the district and in all bordering counties. Updated data discs are sent from the Aggregate Unit to the districts on a quarterly basis.

The ASIS version made available to the districts allows their users to make changes in the existing records but does not allow addition of totally new records. When districts enter updated information into their copies of the database, the Aggregate Unit must be notified by submitting a completed copy of the "ASIS Data Entry Form" (which is shown in Appendix A5) and/or an updated printout with the changes highlighted. This will ensure that future updated discs sent out to the districts will contain accurate and current information. The "ASIS Users Guide" is a handy reference for the new ASIS user. This Guide can be found in appendix A6.

Gravel pits that are recommended as "possible aggregate sources" either by the District Soils Engineer or the Aggregate Unit and which, therefore are listed in the project plans, should be field checked by district personnel approximately four months prior to the project letting date. If conditions have changed, the pit should be resurveyed, and the new information should be promptly forwarded to the Aggregate Unit for updating the pit sheet.

Locations of aggregate sources contained in ASIS are shown by symbols on Mn/DOT county map sheets, which are maintained and distributed by the Aggregate Unit. The "status" field in ASIS refers to one of five source symbols shown on the county maps. The status categories and map symbols are discussed below.

A = Active. Indicates an active file in the Aggregate Unit office, but has no correlation with whether or not the actual source or gravel pit is producing aggregate at the present time. In fact, such a source number (and symbol on the map) may only indicate an aggregate deposit that at one time was leased by the Department and that the Aggregate Unit has tested, but from which no material has ever been excavated.

I = Inactive. Indicates an inactive file in the Aggregate Unit office, where the actual source of the aggregate is either depleted or at least unavailable for further use by the Department. If future circumstances make such sources available, the status may be changed to Active.

N = No Data. Indicates a location, which, while assigned a number in ASIS, has never been drilled and sampled by the Aggregate Unit and for which no test data is available at Mn/DOT.

Q = Quarry. Indicates a bedrock quarry. Most of the quarries presently listed in ASIS and identified by symbol on the county maps are found in southeastern Minnesota's limestone/dolostone deposits. Igneous and metamorphic bedrock quarries in other parts of the state may also be added when they are adequately identified and assigned source numbers.

C = Commercial Aggregate. Indicates a known commercial source of aggregate which has never been leased by the Department, but which has been assigned a source number in ASIS to facilitate tracking of test results if and when the source is used by the Department in the future or when used by counties. These sources are often used for concrete aggregate and are also identified in the Concrete Office's COPES - PIT database at the Office of Materials Research & Engineering (Section V-A in the "ASIS Users Guide").

The final map symbol, which does not have a separate status category in ASIS, is "Mn/DOT." These aggregate sources are owned by the Department and number about 450, throughout the state. Department-owned sources that contain usable reserves of aggregate, especially where private sources are unavailable or in short supply, should be recommended and listed in project plans. Private road contractors can use them only if they are listed in the plans prior to the bid letting. Requests for use of Department-owned pits are occasionally made by private road contractors for projects they have been awarded, but where the pit was not previously listed in the project plans. In these cases, such "after the fact" use is not permitted, because it could be construed by others as providing unfair advantage to the successful bidder.

In the interest of conserving higher type aggregates and ensuring "best use" of Mn/DOT-owned materials, department policy prohibits the use of Class 5 and better aggregates, from Mn/DOT-owned sources, for common borrow or granular borrow. If a Mn/DOT-owned source is shown in the plans because it is recommended as a possible source in the Design Recommendation Report, the above restriction still applies (Section 1602 of Mn/DOT's Specifications).

It is important to keep in mind that a privately owned, leased, aggregate source should only appear in project plans as a possible source if the term of the lease extends through the letting date of the project and reasonable time remains for use on the project. Sources with expired leases may also be listed, with the provision that leases be renewed before inclusion of the pit in the plans. This procedure has merit, because it is often impractical to maintain current leases on all usable pits, on the chance that the pit might be used at some time in the

future. However, problems do occur as a result of this practice. To minimize these problems, the District Soils Engineer should notify the District Right-of-Way Engineer of the need for lease renewal at the time the Design Recommendations Report is being written. The District Right-of-Way Office has the sole responsibility for gravel pit leasing and lease renewal.

Another way to ensure absolute availability of a gravel source, in lieu of outright purchase by the Department, is by the negotiation of an exclusive lease. Typically, these have been negotiated for five or 10 year periods, with an initial lump sum payment to the owner. Subsequent use of material from these aggregate sources, during the term of the lease, is then the same as if the Department owned the source.

Aggregate source information is supplied by the Aggregate Unit whenever a District Engineer requests a Surface Type Determination for a new project. This information is furnished on Mn/DOT Form TP-02471-02 (7-83) and is primarily for the purpose of preparing cost estimates by the Estimating Unit (Office of Technical Support). The sources listed on this form do not necessarily have current leases, and many are commercial sources that have never been leased by Mn/DOT. Therefore, DO NOT routinely list these sources as "possible sources" in the Design Recommendation Letter unless you know that a current lease exists, or will be forthcoming from Right-of-Way.

"Natural Materials Sources" (Section 1602 of Mn/DOT's Specifications) contains additional valuable information on aggregate source acquisition, use, and rehabilitation.

4-2.02.07 BORROW SOURCES

The District Soils Engineer reviews the proposed alignment and grade, working with the designer at the district level, to determine the need for, and the required quantity of, borrow material. Identification of past borrow operations, as well as a general knowledge of geologic conditions along the project alignment, can aid in identifying potential borrow sources.

4-2.02.08 EXISTING TOPOGRAPHY, VEGETATION, AND DRAINAGE

Valuable information, including some potential problem areas that could require special consideration, can often be identified by studying the topography, vegetation, and drainage. Examples are steep slopes, which could require stability studies; rock outcrops, which could indicate anticipated subsurface conditions; and thick underbrush or wet areas, which could require underdrains or other special treatment.

4-2.02.09 EXISTING STRUCTURES

The performance and need for replacement or repair of existing privately owned structures, particularly adjacent to deep cuts or fills, should be assessed and carefully recorded. Similarly, there may be a need to replace or extend existing roadway structures, such as the extension of culverts for shoulder/slope widening projects.

4-2.03 BORING PLAN

Boring locations should be planned by the District Soils Engineer utilizing all of the information gathered in the office studies described in the previous section and in accordance with the following guidelines.

4-2.03.01 SOILS

Subgrade and/or foundation materials are assessed through the use of disturbed borings, undisturbed borings (for foundations), test pits, or hand augers. Borings should be deep enough to develop the engineering data required for analysis and should penetrate major soil horizons, frost depth, and frost-susceptible materials. Borings should be taken to a depth of at least five feet below the proposed grade line in cut areas and at least five feet below natural ground in fill sections. At least one boring in each fill section should extend to a depth equal to the height of the proposed fill. Generally, borings are extended to depths of five to 10 feet and are located at 100-foot intervals, but consideration should be given to geologic conditions and planned alignment and grade. On divided highways, the borings on the two roadways may be staggered.

Where non-uniform subgrade soils or water conditions are encountered, boring intervals should be decreased. The importance of recognizing non-uniform conditions is that they are often to blame for poor pavement performance and need to be specifically addressed by the District Soils Engineer or District Design Engineer in developing design recommendations.

4-2.03.02 ROCK

Boring intervals should be decreased and additional borings should be taken on cross sections, as needed, when there is evidence of bedrock, or if refusal occurs above the planned bottom of boring. The number of borings required depends on the anticipated rock variability and length of cut. When there is evidence of the possibility of bedrock above the proposed grade line, rock coring will be required. Requests for geological work, geophysical work, and/or rock coring should be made through the Geology Unit, with copy to Foundations Unit (Office of Materials Research & Engineering).

4-2.03.03 GROUND WATER

Ground water levels vary with changes in season, precipitation, infiltration, and other factors. The level and rate of inflow of the ground water should be determined during and for a period of time after drilling (preferably 24 hours), so that both the need for long-term observation methods can be evaluated and the effect of ground water on the roadway design can be considered. When water is encountered in a boring, it should be reported as a "water level" or "water encountered" unless the specific occurrence of the water is known, i.e., water table, perched, artesian.

Both short- and long-term methods can be used to observe the ground water level. Short-term methods are relatively quick and easy, and include observing drilling tools and cuttings during the drilling operations for water contact and

measuring the depth from the ground surface to the water level in the boring after the boring is finished. These observations are more reliable in pervious soils, such as sands and gravels, than in clayey soils of low permeability. In clayey soils, the ground water may not have had time to stabilize and/or the sides of the borehole may have become smeared, either of which gives a false impression that the ground water table is deeper than it actually is.

More reliable methods of determining ground water levels include referencing historical ground water levels in the area and long-term measurements, such as the installation of piezometers (small diameter wells). If long-term measurements appear to be necessary, the Foundations Unit (Office of Materials Research & Engineering) should be contacted.

(Note that installation and abandonment of wells, peizometers and bore holes penetrating groundwater is regulated by state laws and regulations, and department policy, and shall be persured only by qualified persons and equipment in compliance with all applicable rules.

4-2.03.04 SWAMP AREAS

In swamp areas, to provide an adequate picture of the subsurface conditions within the swamp, boring intervals should be decreased and additional borings should be taken on sections perpendicular to the centerline. A minimum of three borings is required for each section, when relatively uniform swamp bottoms are encountered. The borings are usually taken at the centerline and on each side of the roadway, half way between the shoulder P.I. and the toe of the slope. At least one boring should extend a minimum of 15 feet below the apparent swamp bottom, to provide adequate evidence against a false bottom. Resistance soundings, consisting of advancing the augers without sampling and recording the level where resistance is felt (assumed bottom of swamp), may be used to supplement boring information.

Scheduling of work in swampy areas is important. Access may be more easily obtained during winter months when the swampy area is frozen. If it is determined that excavation of the swamp will result in injury to persons or damage to adjacent facilities, or if a floated embankment is desired, the Foundations Unit (Office of Materials Research & Engineering) should be contacted to obtain undisturbed samples, install piezometers, and/or perform stability studies.

4-2.03.05 DEEP OR SIDEHILL CUTS

Care should be taken in determining the soil and water conditions present whenever deep or sidehill cuts of appreciable magnitude are proposed. In such areas, boring intervals should be reduced, with borings being taken at the edge of the roadway, as well as along the centerline. Where general instability could create a problem, borings should be placed on sections perpendicular to the centerline on at least the uphill side of the cut. Where buildings or structures are located adjacent to the crest of slope, the Foundations Unit (Office of Materials Research & Engineering) should be notified so that undisturbed sampling, piezometer installation, and/or stability studies can be performed.

4-2.03.06 SETTLEMENT/FROST HEAVE/SLIDE AREAS

Ground movements should be monitored through the use of field surveys, installation of slope inclinometers, etc. To determine differential transverse and longitudinal heaving, level surveys should be performed in both winter and summer, with the results then compared to each other. It may be advisable to set frost-free bench marks for winter and summer reference.

Although the Foundations Unit (Office of Materials Research & Engineering) has the responsibility to analyze the effects of ground movements, the District Soils Engineer plans the survey work, coordinates it with the Foundations Unit and other offices within the district, and may assist in obtaining field measurements.

4-2.03.07 AGGREGATE SOURCES

Areas proposed as aggregate sources should be explored by sufficient borings, planned to determine the quantity and quality of the aggregate available. Generally, borings are spaced on a 100-foot grid pattern, with closer spacing in erratic deposits and possibly wider spacing in uniform deposits. The Aggregate Unit (Office of Materials Research & Engineering) may be contacted to perform the initial investigation or when more thorough sampling is required after a lease is obtained. Reference is made to Section 4-2.02 and the Appendix A6, which describes the Department's ASIS system.

4-2.03.08 BORROW SOURCES

Likely areas intended for use as borrow sources should be explored by sufficient borings to determine the quantity and quality of borrow materials and the level of the ground water table. Borings should be extended to the depth of planned excavation plus three feet and spaced on a 100-foot grid pattern, or closer in the case of non-uniform deposits. The Foundations Unit (Office of Materials Research & Engineering) may be contacted to obtain undisturbed samples if accurate shrink/swell factors are desired.

4-2.03.09 STRUCTURE FOUNDATIONS

It is the responsibility of the District Soils Engineer or District Design Engineer to notify the Foundations Unit (Office of Materials Research & Engineering) of the need to perform a Foundation Survey (Section 4-1.02.02).

Small culverts, of less than approximately 80 square feet in size, may be investigated by the district. In these situations, a minimum of three borings should be planned on a section along the culvert alignment. The borings should be extended to provide subsurface and ground water information to a depth below the proposed culvert footing equal to at least twice the width of the footing.

4-2.04 SAMPLING PLAN

4-2.04.01 PURPOSE AND TYPE OF SAMPLES

Undisturbed samples, i.e., samples of an intact specimen of material that are minimally altered from their in-situ condition, are required to test for those properties that are controlled by the overall mass, such as strength and permeability. Disturbed samples, i.e., samples that are broken up or remolded, can be used for determining properties that are controlled by the individual components of the material, such as the grain-size distribution and Atterberg limits. The following paragraphs discuss these samples, and the responsibility for obtaining them.

1. Disturbed Samples. Disturbed samples are taken with augers, by the district, in general accordance with AASHTO T 203. The augers are rotated and advanced into the soil the desired distance. They are then withdrawn (pulled dead) from the hole and the soil is removed for examination and testing. Samples may also simply be obtained from the auger cuttings.

   For testing purposes, minimum sample quantities needed for the required tests are given in Table 4-2.1.

Table 4-2.1. Required sample quantities.

Test	Quantity
Gradation for granular material	20 - 30 lb.
Gradation for cohesive material	10 lb.
Gradation and moisture-density for cohesive material	25 - 30 lb.
Fertility	10 lb.
R value determination	60 lb.
pH for surface water	1 pint in clean glass jar
pH for soil	1 lb. in clean plastic or glass container

2. Undisturbed Samples. Undisturbed soil samples and rock cores required for foundation design are obtained by the Foundations Unit (Office of Materials Research & Engineering).

   Pavement Cores are used for measurement of the pavement/ base thickness and evaluation of the condition of the existing roadway. They are obtained by the district.

4-2.04.02 SAMPLING FREQUENCY AND DEPTH

1. Borings. When possible, it is advantageous to screw the augers into the ground slowly and then pull them out without rotation (pulled dead), allowing for a better visualization and logging of changes/transitions in the stratigraphy. Representative samples of each material type that is significant to the project design and construction should be saved for classification, R value determination, and turf establishment testing. It is common to obtain one soil sample for subsequent classification testing, at every five feet penetrated and at changes in material type. More frequent sampling may be dictated by unusual soil conditions.

   Samples for classification and R value should be obtained from borrow sites if the material is to be used in the upper four feet of the subgrade. For investigations of potential aggregate sources, a representative sample of the material should be taken from each hole for each substantial change in the appearance of the material, with at least one sample for each 10 feet of penetration. Where the material extends below the ground water table, separate samples from above and below the water table should be taken. Samples from auger-bored holes deeper than about 40 feet are not considered reliable because of the disturbance and displacement of materials as they rise on the auger.

   Minimum sampling rates required for various tests are described below.

   a. R Value. Table 5-3.1 establishes guidelines for the frequency of sampling for the determination of R values as a function of the soil classification.

   b. Fertility. Where the topsoil is to be removed and replaced to provide planting soil, samples should be taken at the approximate rate of one per mile from the full depth of the topsoil to be removed. Additional samples are required if there is a major change in soil type.

      If the topsoil is not being replaced, samples should be taken from all horizons that will be exposed and expected to support some type of turf. In this case, the sampling rate should be one per horizon per mile.

   c. pH. pH values are obtained for turf establishment recommendations and culvert type determination. Water samples for pH determination should be obtained from all stream, river, or waterway crossings which might require a culvert installation. Soil pH samples should be obtained as needed.

2. Test Pits. Random representative samples of the materials encountered during test pit excavation should be retained for subsequent classification and R value testing.

4-2.05 FORMAT FOR GEOTECHNICAL SURVEY FIELD NOTES

Visual observations, the sequence of materials encountered in the borings, and the results of field tests should all be carefully and accurately recorded. Most organizations use a "boring log," "record of subsurface exploration," or some similarly titled form for this purpose. The Department does not have a mandated form, but strongly recommends that each District Soils Engineer keep such data in a uniform, organized, and retrievable manner.

The record of each boring in the field book should be capable of standing alone, complete with all of the following information, entered in the order given:

- Project identification and number,
- Boring location, typically by roadway station and offset left or right of centerline,
- Method of drilling and sampling (e.g., flight augers with grab samples),
- Date of start and completion,
- Names of drill crew members,
- Surface elevation,
- Description (Section 3-2.04) and depth of materials encountered,
- Sample locations,
- Ground water information when encountered and at appropriate recorded times after drilling, and
- Other pertinent information and/or comments.

The accuracy of the recorded information is important because it becomes the basis for subsequent design recommendations. For example, the level of silt seams may impact frost design, wet or saturated layers may impact dewatering or drainage requirements, and shear planes or other discontinuities may impact a slope stability problem.

4-2.06 TESTS

Classification testing is routinely performed at the district level. All testing of undisturbed samples to determine engineering properties (including R value) is performed by the Office of Materials Research & Engineering. Turf establishment testing (fertility) is performed for the Office of Materials Research & Engineering by the University of Minnesota.

The following sections briefly describe many of the tests that may be performed at the district level or by the Office of Materials Research & Engineering on soil samples obtained for input into the design process. These descriptions are provided to give the District Soils Engineer adequate instruction, or references,

to perform classification testing or to provide a general feel for the type and magnitude of testing performed by the Office of Materials Research & Engineering. General reference is made to Mn/DOT's Grading and Base Manual and/or Lab Manual, where appropriate, for detailed descriptions of the following tests. In the case of differences, preference should be given to the above-referenced Mn/DOT manuals.

4-2.06.01 MECHANICAL ANALYSIS

A sieve analysis of the coarser portion of the sample and a hydrometer analysis of the fine-grained portion of a sample combine to make up a mechanical analysis.

1. Sieve. A sieve analysis is used to determine the grain-size distribution of the portion of a soil larger than the No. 200 sieve. The method of performing a sieve analysis is presented in full detail in Section 5-692.215 of Mn/DOT's Grading and Base Manual and Section 1200 of Mn/DOT's Laboratory Manual. This test is based on AASHTO T 27.

2. Hydrometer. A hydrometer analysis is used to determine the grain-size distribution of the portion of a soil smaller than the No. 200 sieve. A brief description of the test procedure follows; for complete instructions, refer to Section 1302 of Mn/DOT's Laboratory Manual. This test is based on AASHTO T 88.

   Approximately 100 grams of soil passing the No. 10 sieve (or 50 grams of air-dried soil passing the No. 40 sieve) is required. For sandy materials, 100 grams should be used. For silty or clayey materials, 50 grams should be used. This material is then treated with 125 milliliters of sodium hexametaphosphate, a dispersing agent, and left alone to mix for 12 hours. After the 12-hour time period has elapsed, the material is mechanically dispersed for 60 seconds using a mechanical stirring device. After dispersion, the material is placed in a glass container, and distilled water is added until a volume of 1,000 milliliters is attained. The container is then shaken vigorously for one minute, at which time it is gently placed on a level table. Readings are then obtained by using a hydrometer at intervals of 2, 4, 8, 15 minutes, and so on. If temperature changes can occur, the temperature of the mixture should also be taken at each interval.

   The data is then reduced and recorded on a grain-size distribution chart which relates the percent passing (or retained) and the grain size.

4-2.06.02 ATTERBERG LIMITS

The Liquid Limit (LL) is defined as the water content of the soil at which a standard groove cut in the remolded soil sample will close, over a distance of one-half inch, at 25 blows of the LL cup falling 10 millimeters on a hard rubber base. A moisture content sample is then taken from the soil sample, where the groove closed, to determine the LL. For further information, see Section 1303 of Mn/DOT's Laboratory Manual. This test is based on AASHTO T 89.

The Plastic Limit (PL) is defined as the water content at which a thread of soil *just* crumbles when it is carefully rolled out to a diameter of one-eighth inch. For further information see Section 1304 of Mn/DOT's Laboratory Manual. This test is based on AASHTO T 90.

The Plasticity Index (PI) is the range of water contents over which a soil is plastic. It is determined by Equation 4-2.1.

$$PI = LL - PL \qquad \text{Eq. 4-2.1}$$

4-2.06.03 ORGANIC CONTENT

The organic content of a soil sample should be computed by AASHTO T 267 if the total percentage of organics is desired. If the suitability of the soil material for supporting growth is desired, then AASHTO T 194 should be used. The total percentage of organics method is briefly described below.

First, place 10 to 40 grams of oven dried (230° ± 9° F) soil, passing the No. 10 sieve, into tared crucibles or porcelain dishes and weigh. Then, place the sample and container in an oven/furnace for six hours at 833° ± 18° F. After the container has cooled, reweigh the dried sample and container. Finally, using Equation 4-2.2, calculate the organic content.

$$\textit{Percent Organic Matter} = \frac{A - B}{A - C} X\ 100 \qquad \text{Eq. 4-2.2}$$

where

A = weight of container and oven dried soil before ignition, gm.;
B = weight of container and oven dried soil after ignition, gm.; and
C = weight of container, gm.

4-2.06.04 IN-PLACE MOISTURE AND DENSITY

1. Moisture. The moisture content of a soil sample can be determined by either the oven method or the calcium carbide gas pressure (speedy moisture) method. These methods are covered in full detail in Section 5-692.250 of Mn/DOT's Grading and Base Manual. They are based on AASHTO T 217 (speedy moisture) and T 265 (drying oven).

2. Density. Many methods are available for determining the density of a soil mass. Among them are the sand cone, drive cylinder, rubber balloon, block sample, and nuclear methods. Initially, the wet weight, the volume, the moisture content, and the dry weight of the soil sample must be determined for each of these methods. From this information, the wet and dry density can be determined.

   Specific instruction on the method normally used by the Department - the sand cone test - can be found in Section 5-692.242 of Mn/DOT's Grading

and Base Manual. The bases for these tests are AASHTO T 191 (sand cone), AASHTO T 204 (drive cylinder), AASHTO T 205 (rubber balloon), AASHTO T 233 (block sample), and AASHTO T 238 (nuclear method).

4-2.06.05 STRENGTH

The following strength tests are performed at the Office of Materials Research & Engineering.

1. R Value. To determine the R value, four separate samples of the soil are prepared, bracketing the optimum moisture content of the soil as determined by the Proctor test.

   After allowing the samples to stand overnight, each sample is then compacted in four equal lifts into a four-inch diameter mold to a height of 2-1/2 inches. The lifts are compacted by a mechanical ram, the first three of these lifts receiving 10 tamps at 50 pounds per square inch and the final lift receiving 100 tamps at 100 to 150 pounds per square inch, depending on the soil.

   A phosphor-bronze disk and filter paper is then placed on the specimen and, using a compression testing machine, a load is applied at a rate of 2,000 pound feet per minute until a specified indication of water in the sample has been exudated. A pounds per square inch reading is taken at the exudation end point, the target reading being 240 pounds per square inch. It is therefore, essential that the four sample specimens "bracket" a reading of 240 pounds per square inch.

   The specimens are then placed in the expansion-pressure device, water is added to the top of the sample and allowed to soak for a period of 16 to 20 hours. The amount of expansion is measured to the nearest 0.0001 inch, and expansion pressure (in pounds per square inch) is then calculated.

   Next, the resistance value is determined by applying a five pounds per square inch horizontal pressure to the specimen and a vertical load applied to achieve a vertical displacement of 0.05 inches per minute. The horizontal pressure at a vertical load of 2,000 pounds is recorded and the loading stopped. The vertical load is reduced to 1,000 pounds, and the horizontal pressure adjusted to five pounds per square inch. Next, the stabilometer pump handle is turned at a constant rate of two turns per second, and the number of turns required to raise the horizontal pressure from five to 100 pounds per square inch is recorded as the turns displacement. The resistance, or R value, is determined from Equation 4-2.3.

$$R = 100 - \left[\frac{100}{\frac{2.5}{D}} * \left(\frac{160}{P_h} - 1\right) + 1\right] \qquad \text{Eq. 4-2.3}$$

where

R	=	resistance value,
D	=	turns displacement, and
P_h	=	horizontal pressure, psi.

If the height of the specimen is not exactly 2.5 inches, it must be corrected, using the available height-correction chart. Specific testing information can be found in AASHTO T 190.

Table 5-3.2 in Section 5-3.05.02, Bituminous Pavement Design, provides R value estimates by soil classification. These values may be used for design purposes for smaller type projects.

2. California Bearing Ratio. To determine the California Bearing Ratio (CBR), three separate samples should be compacted in three layers each, by tamping with a 5-1/2-pound hammer. The compactive energy (number of blows) should be varied for each sample so that their dry density ranges between about 95 and 100 percent of the sample's previously determined maximum dry density. A dial indicator is then set on the sample, and surcharge weights approximately equivalent to the design overburden weight, but not less than 10 pounds, are added. The sample is then immersed in water and allowed to soak for four days. A final reading is obtained from the dial indicator and the percent swell calculated using Equation 4-2.4.

$$Percent\ swell = \frac{\Delta\ length}{4.584}\ X\ 100 \qquad \text{Eq. 4-2.4}$$

The sample is then removed from the water and allowed to drain for 15 minutes. Thereafter, the penetration piston is seated with a 10-pound load, and surcharge weights equal to those used during the soaking are placed on the piston. Load is than applied to the three-square inch piston at a steady rate of 0.05 inch per minute, and readings are obtained at 0.025, 0.050, 0.075, 0.100, 0.150, 0.200, 0.300, 0.400, and 0.500 inch of penetration. The data collected is then graphed as a stress (load/area)-strain (penetration/length) curve. This data is corrected for shape and the CBR values calculated for 0.10 and 0.20 inches of penetration by Equation 4-2.5. If the CBR at 0.20 inches of penetration yields the greatest value, the test should be re-conducted.

$$CBR = \frac{Corrected\ load\ value}{standard\ load}\ X\ 100 \qquad \text{Eq. 4-2.5}$$

The standard load values are 1,000 and 1,500 pounds per square inch, respectively. The three test results are plotted as CBR versus dry

density, and the design value is chosen based on the dry density required in the field. For detailed instructions refer to AASHTO T 193.

3. Resilient Modulus. To conduct the resilient modulus (M_r) test, numerous load cycles are applied to samples compacted in a triaxial cell and confined to simulate field conditions. A vertical load is applied, and the resultant deformation of the sample is measured. The test can be run under varying confining pressures, with the sample subjected to different moisture contents to obtain the M_r as a function of stress level and moisture conditions.

The deviator stresses and cell pressures used are different, depending on whether a cohesive or granular material is being tested. Initially, a series of 1,000 or 1,200 loads are applied for cohesive or non-cohesive samples, respectively. Once these "conditioning loads" have been applied, actual testing begins.

For cohesive samples, 1,600 additional load cycles with various deviator stresses and cell pressures are conducted, with measurements of the recovered deformations after the 200th load for each combination. For non-cohesive samples, 5,400 additional load cycles are used.

The M_r is defined as the ratio of the repeated axial deviator stress σ_d to the recoverable axial strain ϵ_r.

$$M_r = \frac{\sigma_d}{\epsilon_r} \qquad \text{Eq. 4-2.6}$$

M_r = resilient modulus;
σ_d = repeated axial deviator stress ($\sigma_1 - \sigma_3$), psi
ϵ_r = recoverable axial strain.

The material model for:

(a) Fine-Grain Soils

$$M_r = K_1 \sigma_d^{-K_2} \qquad \text{Eq. 4-2.7(a)}$$

σ_d = repeated deviator stress, psi
K_1, K_2 = material constants

(b) Coarse-Grain Soil

$$M_r = K_1 \Theta^{K_2} \qquad \text{Eq. 4-2.7(b)}$$

Θ = Sum of principal stresses ($\sigma_1 + 2\sigma_3$)
K_1, K_2 = material constants

The stress sensitivity of fine grain and coarse grain soils is described by Eqs. 4-2.7(a) and 4-2.7(b), respectively. For fine-grained cohesive soils,

the resilient modulus decreases with increasing stress while granular soils will stiffen with increasing stress.

For detailed instructions on conducting the resilient modulus for soils refer to AASHTO T-274.

The current test procedure is under review and revision and a new test procedure is anticipated. In the meantime, M_r is being used in the Strategic Highway Research Program (SHRP) and is expected to gain increasingly widespread acceptance as a means of characterizing pavement materials.

4. Cohesion and Angle of Internal Friction. A soil's cohesion (c) and angle of internal friction (ϕ) may be determined by unconfined compression, triaxial, or direct shear tests. Each of these methods is discussed in the following paragraphs.

a. Unconfined Compression. To run an unconfined compression test, as described in detail in AASHTO T 208, the sample must be undisturbed and have a minimum diameter of 1.3 inches. The height to diameter ratio should be between two and three. The sample may be tested using either controlled strain or controlled stress procedures. For controlled strain tests, the sample is loaded with a strain rate of between one-half and two percent per minute. The strain rate should be estimated to provide for failure in less than 10 minutes. Load and deformation readings should be taken every 30 seconds. Load readings are taken until 20 percent strain is reached or until the load readings begin to decrease.

For controlled stress tests, the failure load is estimated, based on previous experience or by using some type of penetration device, such as a pocket penetrometer. Loads are then placed on the sample in increments of one-tenth to one-fifteenth of the failure load. Readings of strain are taken 30 seconds after each load is applied. This is continued until 20 percent strain is reached, or until failure occurs. If more than 15 or less than 10 load increments are required, then the magnitude of the loadings should be adjusted. After the test is complete, the moisture content of the sample should be determined and a stress versus strain plot prepared. The stress and strain are determined by Equations 4-2.8, 4-2.9, and 4-2.10.

$$\epsilon = \frac{\Delta L}{L_o} \qquad \text{Eq. 4-2.8}$$

where

ϵ = axial strain;
L = length of sample, in.; and
L_o = initial length, in.

$$A = \frac{A_o}{1 - \epsilon} \qquad \text{Eq. 4-2.9}$$

where

A	=	area of sample, in^2;
A_o	=	initial area of sample, in^2; and
ϵ	=	axial strain.

$$\sigma_c = \frac{P}{A} \qquad \text{Eq. 4-2.10}$$

where

σ_c	=	compression stress, psi;
P	=	load, lb.; and
A	=	area of sample, in^2.

b. Triaxial. Four different types of triaxial compression tests can be performed, depending upon the consolidation and drainage conditions allowed during the test. They are the unconsolidated undrained test (UU or Q), the consolidated undrained test without pore pressure measurement (CU or R), the consolidated undrained test with pore pressure measurement (CU- or R-BAR), and the consolidated drained test (CD or S). The choice of which test to use should be determined by the anticipated field conditions, with the laboratory sample modeling the "real-world" situation. Tests run by the Office of Materials Research & Engineering are generally consolidated undrained tests with pore pressure measurement to determine the effective, long-term, soil strengths.

For all triaxial tests, a minimum of three samples should be tested. One sample should be tested at the overburden pressure. One of the others should be tested at a lower pressure, and the third should be tested at a higher pressure. The samples should have a minimum diameter of 1.3 inches and a height to diameter ratio between two and three.

For a Q test, the sample is confined and no volume change due to consolidation is allowed in the sample. Load is then applied to the sample by means of controlled strain with a strain rate of 0.3 to 1.0 percent per minute, with a suggested rate of 0.5 percent per minute. Load and deformation readings are recorded until the load ceases to increase with strain or until 20 percent strain is reached. After the test is completed, the moisture content of

the sample should be determined and a stress versus strain plot prepared for each test. The stress and strain are determined by Equations 4-2.8, 4-2.9, and 4-2.10.

For an R or R-BAR test, the sample is confined and volume change due to consolidation is allowed and recorded. Once the sample has consolidated under the applied confining pressure, loading may begin. The load should be applied at a rate which allows for pore pressures to remain equal throughout the sample. No volume change should be allowed during loading. Load and deformation readings should be taken at intervals to accurately define the stress-strain curve. If an R-BAR test is being conducted, the pore pressure should be recorded along with the load and deformation. Readings should be taken until failure is obtained or until 20 percent strain is reached. After the test is completed, the moisture content of the sample should be determined and a stress versus strain plot prepared for each test. The strain values are determined by Equations 4-2.8 and 4-2.9. Equation 4-2.10 is used to determine the stress for an R test,

$$\sigma_c = \frac{P}{A} - U \qquad \text{Eq. 4-2.11}$$

while Equation 4-2.11 is used for an R-BAR test.
where

σ_c	=	compression stress, psi;
P	=	load, lb.;
A	=	area of sample, in^2; and
U	=	pore pressure, psi.

For an S test, the sample is confined and volume change due to consolidation is allowed and recorded. Once the sample has consolidated under the applied confining pressure, loading may begin. The load should be applied slowly enough that no pore pressure is developed. Volume change during loading is recorded, along with the load and deformation. Load and deformation readings should be taken at intervals to accurately define the stress-strain curve and should be taken until failure is obtained or until 20 percent strain is reached. After the test is completed, the moisture content of the sample should be determined and a stress versus strain plot prepared for each test. The stress and strain values are determined by Equations 4-2.8, 4-2.9, and 4-2.10.

c. Direct Shear. An additional method of determining a soil's cohesion and angle of internal friction is by performing a direct shear test, as described in AASHTO T 236. A direct shear test is essentially a consolidated drained test, i.e., no pore pressure is allowed to develop in the sample during shearing.

As in the triaxial compression tests described above, a minimum of three samples should be tested. One sample should have a normal stress equal to the overburden pressure. One of the others should be tested at a lower pressure, and the third should be tested at a higher pressure. The samples should be at least two inches in diameter (or width, if rectangular samples are used) and have a thickness of at least one-half inch or six times the maximum grain diameter. The minimum diameter to thickness, or width to thickness, ratio should be two to one. After the samples are contained in the shear box, the normal load should be applied. Consolidation readings are taken as in a one-dimensional consolidation test. When consolidation is complete, the time to 50 percent consolidation may be determined from the consolidation data. The time to failure is then determined by Equation 4-2.12.

$$\textit{time to failure} = 50t_{50} \qquad \text{Eq. 4-2.12}$$

where

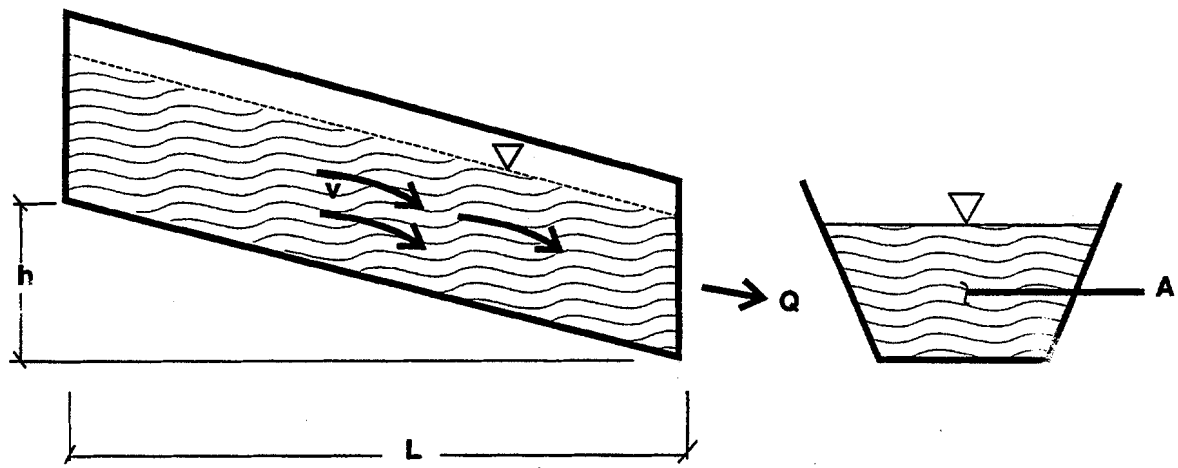
t_{50} = time to 50 percent consolidation.

The rate of shearing should then be estimated by dividing the estimated shear deformation at failure by the computed time to failure. Load and deformation readings should be taken at intervals to accurately define the stress-strain curve and should be taken until failure is obtained or until 10 percent strain is reached. After the test is completed, the moisture content of the sample should be determined and a stress versus strain plot prepared for each test.

4-2.06.06 PERMEABILITY

Before studying permeability, a clear understanding of the terms relating to the flow of water is necessary. Figure 4-2.1 shows those terms discussed in this section.

Figure 4-2.1. Terms relating to the flow of water.



A = Area of sample
H = Hydraulic head
L = Length fo sample
Q = Volume of flow
V = Velocity

Two methods for determining the permeability of soils are as follows:

1. Constant Head. A constant head permeability test, as described in detail in AASHTO T 215, is used to determine the permeability of granular materials. The sample for testing is selected and compacted into the permeability mold. (The density of compaction affects the permeability.) It is then saturated under vacuum to assure that there is no air in the sample. Water is then allowed to flow through the sample from the constant head tank to a collector tank. Water is continually added to the constant head tank to maintain the water level.

   Once a stable head condition is obtained, readings of time, head, quantity of flow, and water temperature are recorded. The head is then increased by five millimeters and the test is repeated. This is continued until the turbulent flow boundary is reached. This is determined as the head when the velocity (v) and hydraulic gradient (i) do not have a linear relation, where v is defined in Equation 4-2.13 and i is defined in Equation 4-2.14.

$$v = \frac{Q}{At} \qquad \text{Eq. 4-2.13}$$

where

v	=	velocity, in./sec.;
Q	=	volume of flow, in^3;
A	=	area of sample, in^2; and
t	=	time, sec.

$$i = \frac{h}{L} \qquad \text{Eq. 4-2.14}$$

where

i	=	hydraulic gradient;
h	=	hydraulic head, in.; and
L	=	length of sample, in.

The coefficient of permeability is then determined using Equation 4-2.15. The permeability is corrected for the water temperature by multiplying the calculated permeability by the ratio of the viscosity of water at test temperature to the viscosity of water at 68° F (20° C).

$$k = \frac{QL}{Ath} \qquad \text{Eq. 4-2.15}$$

where

k	=	coefficient of permeability, in./sec.;
Q	=	volume of flow, in^3;
L	=	length of sample, in.;
A	=	area of sample, in^2;
t	=	time, seconds; and
h	=	hydraulic head, in.

2. Falling Head. A falling head permeability test can be used to determine the permeability of fine-grained soils. The sample should be compacted and saturated as above for the constant head test. The constant head tank from the previous test should be replaced with a burette. The difference in water level from the burette to the collector tank is measured and recorded as h_1. Water is then allowed to flow out of the burette and into the collector tank. Once a predetermined change has occurred, the head is measured again and recorded as h_2. The time required for the change in head and the temperature of the test water should also be recorded. The coefficient of permeability is then determined using Equation 4-2.16. The permeability is corrected for the differences in water temperature as in the constant head test.

$$k = 2.303 \frac{aL}{At} \log \frac{h_1}{h_2} \qquad \text{Eq. 4-2.16}$$

where

k	=	coefficient of permeability, in./sec.;
a	=	area of burette, in^2;
L	=	length of sample, in.;
A	=	area of sample, in^2;
t	=	time, sec.;
h_1	=	hydraulic $head_1$, ft. or in.; and
h_2	=	hydraulic $head_2$, ft. or in.

4-2.06.07 COMPRESSIBILITY

One-dimensional consolidation tests, run on either undisturbed or recompacted samples of plastic soils, are performed to determine the material's compressibility. These tests are performed by the Foundations Unit in the Office of Materials Research & Engineering .

The major component of compressibility is primary consolidation, the process of gradually transferring the applied load from the water in the soil to the soil structure. That is, the application of a load to a saturated soil causes the immediate reaction of the load being carried by the water in the soil. Then, as the water has had time to dissipate, the load is carried by both the water in the soil and the soil structure until 100 percent of the water drainage occurs under the stress that has been applied. Then the load is carried entirely by the soil structure. The rate at which the primary consolidation occurs depends on the permeability of the soil. In general, the time to primary consolidation is fast for granular soils and slow for plastic soils.

After the primary consolidation is complete, the soil undergoes a realignment and adjustment of the soil particles. This is called secondary consolidation and is usually much less than the primary consolidation, except in marls and peats and in some organic soils. There are some people that talk of a third type of consolidation, tertiary consolidation. This occurs as a result of water being squeezed out of the plant fibers or soil structure and is primarily of concern when dealing with peats or marls.

Samples to be tested should be trimmed and placed in the consolidation ring in such a manner as to avoid disturbance and moisture loss. The consolidation properties of the soil sample are determined under a series of loads. The exact loads used may be varied depending on the anticipated field conditions. The following typical loads are provided for reference: 100, 250, 500, 1,000, 2,000, 4,000, etc. pounds per square foot. The thickness of the sample should be recorded prior to the placement of a load and again at the following times after placing the load: 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, and 30 minutes, 1, 2, 4, 8, 12, 24, 48, etc. hours. Readings should be plotted as thickness versus log time. The next load may be applied when the linear secondary portion of the curve is well defined.

After the load cycles are completed, the thickness versus log time graphs are used to determine the deformation for zero percent consolidation, for 100 percent consolidation, and the time to 50 percent consolidation for each load

cycle. The procedure to find each of the items is called the Casagrande construction. The coefficient of consolidation (c_v) should also be computed for each load interval by Equation 4-2.17. The initial void ratio, water content, density, and degree of saturation are also calculated after the test is completed and the sample is dried. For complete procedures, see AASHTO T 216.

$$c_v = \frac{0.05H^2}{t_{50}} \qquad \text{Eq. 4-2.17}$$

where

c_v	=	coefficient of consolidation;
H	=	length of longest drainage path, in.; and
t_{50}	=	time to 50 percent consolidation, days or sec.

4-2.07 SOILS PROFILE

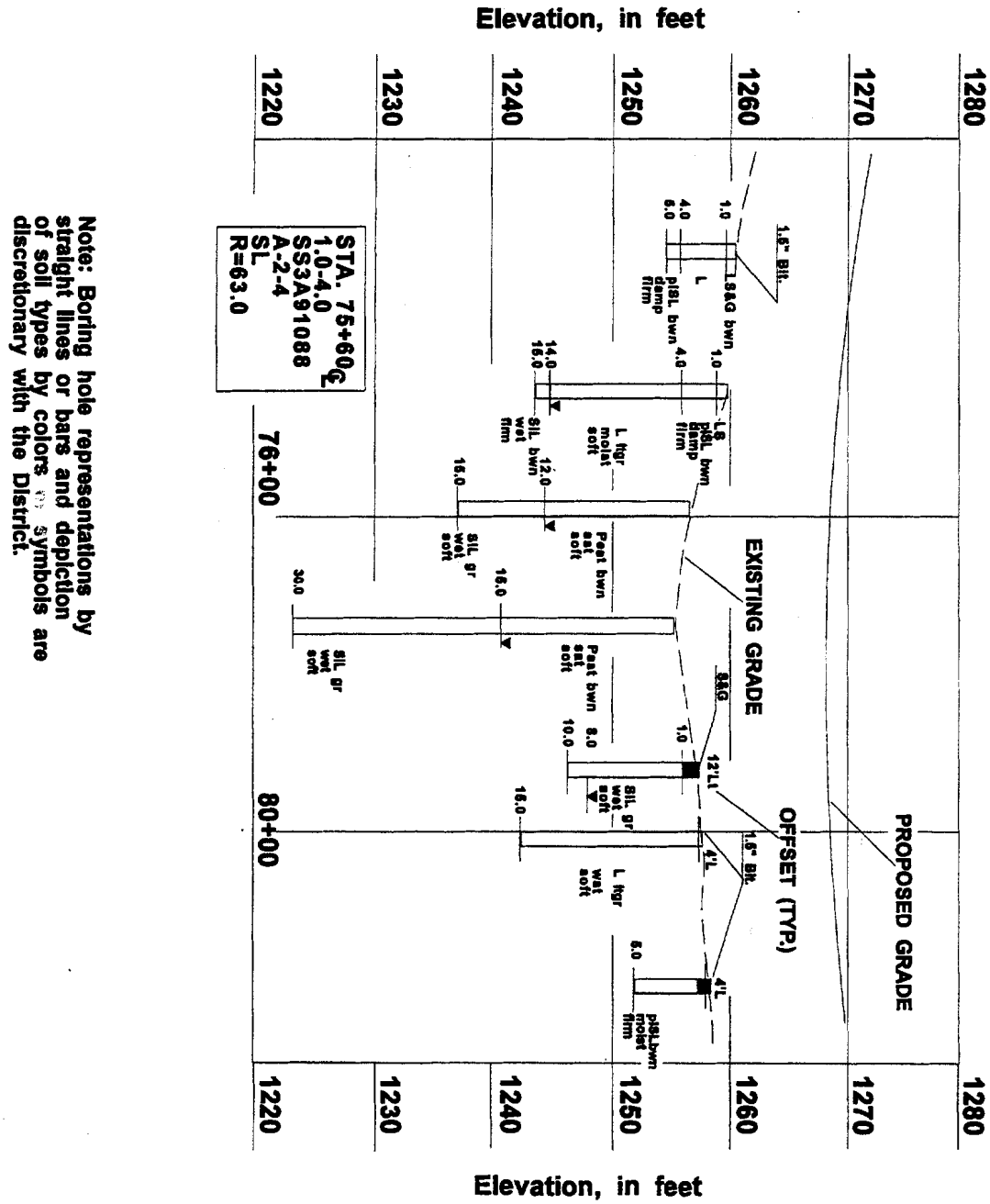
To better visualize the physical relationship between the existing soils and pavement conditions and the new roadway, a soils profile, a drawn cross-section or visual presentation of the information obtained during the performed surveys, should be prepared. It should be drawn at an appropriate scale, such as one inch equals five to 10 feet vertically and one inch equals 20 to 100 feet horizontally, depending on the detail and complexity of the project. Plan/profile presentations should be used whenever possible.

The following should be included on the soils profile:

- Project identification;
- Existing and proposed grades, and/or existing groundlines.
- Boring locations, including station and offset;
- Existing pavement and subgrade conditions, including interpreted stratigraphy;
- Ground water; and
- Pertinent test results.

Figure 4-2.2 is an example of a properly prepared soils profile.

Figure 4-2.2. Example soils profile.



A soils profile should be routinely submitted as part of the Design Recommendations Report on projects where there is significant soils work. For projects which do not typically require much soils information, such as overlays, a soils profile would not be necessary.

4-2.08 BORING ABANDONMENT

All borings should be appropriately abandoned (backfilled) after drilling. Abandonment procedures depend on State rules, presence of water, soil/rock type, and drilling method. This work must always conform to current Department guidelines. See Appendix F.

4-2.09 SUMMARY

As discussed in this section, all information relative to the existing subgrade needs to be considered as background by the District Soils Engineer in preparing design recommendations for a new roadway. Results from previous office studies (Section 4-2.02), newly obtained borings and samples (Sections 4-2.03 and 4-2.04), and testing (Section 4-2.06) should all be used in the design stages.

4-3.0 PAVEMENT SURVEY

The primary purpose of performing a pavement survey is to obtain data, either on a project or network level, that can be used to monitor pavement performance over time and which will provide input to pavement management functions. Minnesota has had parts of a Pavement Management System (PMS) since 1965, when a ride roughness and surface distress rating procedure was initiated to rank potential pavement candidates for resurfacing. A pavement rating (called a Pavement Quality Index and described subsequently) was then set up. The most important activity of the pavement management program was to monitor and provide feedback on pavement performance. The performance data obtained serves as the basis for several Department activities, such as programming and budgeting of candidate projects, as well as the design of the rehabilitation measures deemed necessary to keep the entire network of roadways above an acceptable level of serviceability.

The process currently followed by Mn/DOT in the survey of its pavements, as part of the PMS, is described below.

4-3.01 PURPOSE

Pavement condition surveys are performed to assess and document the current pavement condition. Although many different types of data can be collected in the field, it is generally recommended that efforts concentrate on the type, extent, and

severity of distress, which can then be used to determine the optimal timing for rehabilitation actions and the level of rehabilitation needed.

The ability to predict future condition from an assessment of current condition is critical for scheduling future rehabilitation needs. Maintaining a historical record of pavement condition, over time, allows past pavement deterioration patterns to be evaluated and used to forecast future performance trends. Historical records also provide feedback on the effectiveness of past pavement design strategies. Such feedback should be used to improve future design and rehabilitation strategies.

The type and extent of pavement survey data collected depends primarily on the projected use of the data. If it is to be used as baseline data to determine overall network condition and/or needs, it is considered to be network-level data. Typically, at the network level, a statistically representative portion of the pavement network is surveyed to determine general condition levels.

A pavement survey may also be conducted to determine the rehabilitation needs of a specific section which may be targeted for repair. In that case, a greater level of detail is typically required, influencing the type of equipment used in the survey procedure and the percentage of the pavement section inspected.

Data collection efforts must also take into account the traffic levels anticipated on each roadway segment. Pavement surveys that require slow speeds or stopped equipment along busy roadways create traffic control and safety

problems for the inspection crew and the travelling public. Recent technology is advancing the state of the practice through the development of automated survey equipment that can operate at or near traffic speeds.

It is the intent of these sections to provide information regarding the types of pavement data currently being collected by the Department, as well as to discuss the procedures that should be followed to collect and evaluate this data.

4-3.02 OFFICE STUDIES

Although data collection efforts are typically focused on the collection and evaluation of field data, it should be recognized that a great deal of information can be obtained within the Department's offices. This information is an important component of the pavement survey process and serves as the basis for building an inventory of relevant pavement-related data. Information for each pavement section in the network may include location, length, width, subgrade properties, base and subbase materials and properties, surface properties, as-built design, traffic, historical data on maintenance or rehabilitation actions, or any other information that has a bearing on the ability of the pavement section to perform its given function. Various types of data that may be collected from office studies, and where the data may be obtained, are discussed in the following sections.

4-3.02.01 PREVIOUS DESIGN RECOMMENDATIONS REPORTS

Investigations which were prepared for previous design projects are often a good source of information regarding in-situ properties. These data can provide background information to assess the characteristics of the existing pavement layers and make knowledgeable decisions about appropriate rehabilitation design actions. It may be contained in various Mn/DOT office memoranda, including Foundation Investigation and Recommendations Reports or Recommendations for Grading, Base, and Surfacing Reports that are sent within or between departments.

4-3.02.02 IN-PLACE PAVEMENT TYPICAL SECTIONS

Approximate pavement thickness information is important in determining feasible rehabilitation strategies. Clearance heights for bridges, for example, must be considered before an overlay can be designed. If the maximum tolerance is reached, a portion of the existing surface must be removed prior to the application of an overlay. The thickness of the existing layers is an important factor in determining the feasibility of removing existing material and building overlays. The presence of fabrics, rubber and/or other additives, in the pavement structure should be identified before removal takes place, as they may have an adverse effect on the removal process, and they will definitely impact any recycling efforts. Copies of the corrected final plans from district files or Plan Files in the Central Office are a good source for in-place pavement data. In addition, if any previous geotechnical surveys or other investigative studies included the coring of pavements, approximate pavement layer thicknesses would probably have been obtained. Existing pavement management database files, such as Roadway History, may also be a source of this type of information.

At the time of data collection and evaluation, an effort should be made to investigate whether the particular roadway segment under consideration for any types of proposed rehabilitation, resurfacing, or reconstruction includes any formal or informal research sections. If there are research sections, the Physical Research Unit of the Office of Materials Research & Engineering should be informed of the nature of the proposed work so that further recommendations, analysis, evaluation and conclusions can be made or drawn prior to initiating any design details and/or construction operations.

4-3.02.03 BITUMINOUS CONSTRUCTION DATA

Existing construction data files provide the engineer with information concerning the actual construction of bituminous pavement sections. Engineering design requirements, as well as quality control test results, are available to provide information such as mix types, aggregate sources, mixture properties, in-place densities, and roadway histories.

1. Mix Types. Information on the type of mix that was used in the construction of a pavement surface should be included. Where applicable, mix types used in underlying layers may also be provided. Any special mix properties or additives that may have been used during construction should be noted.

2. Aggregate Sources. Aggregate quality and gradation can also have a large impact on the durability and performance of a pavement surface. Typically, construction files list this type of information. Comparisons of pavement sections constructed with different aggregate sources would assist in developing mix properties which best ensure that the pavements perform as they are intended.

3. Mixture Properties. Mixture property tests are available for asphalt content, asphalt type, and aggregate gradation. In addition, data are available on mixture air void contents on many projects built after 1986. Mixture air void control is critical in increasing pavement life.

4. In-place Density. In-place density data are available on projects with compaction control by the Specified density or Control strip (Nuclear) methods. Good compaction is necessary to assure predicted pavement life.

5. Roadway Histories. Information is available on pavement cross sections including subgrade type, base type and thickness, and pavement thickness. Traffic data and maintenance information regarding the pavement section are available through the appropriate Mn/DOT office.

4-3.02.04 CONCRETE CONSTRUCTION DATA

As described in the previous section, construction data files are also an important source of information for the evaluation of concrete pavements. Important factors which influence the performance of concrete pavements include mix properties (cement type, water/cement ratio, cement void ratio,

slump, entrained air, strength, etc.), aggregate types, aggregate gradations and top size, aggregate sources, thickness, joint spacing, joint sealant type and reservoir dimensions, dowel bar diameter, tie bars, and steel reinforcement. The construction method and equipment used for placement are also important considerations.

4-3.02.05 AGGREGATE/STABILIZED BASE CONSTRUCTION DATA

Base material properties are as important as the surface properties. Aggregate sources and gradations and the type of stabilizing material will affect the performance of the pavement section and influence the design of rehabilitation treatments. As discussed in the two previous sections, construction data files can provide a great deal of this type of information.

4-3.02.06 PAVEMENT MANAGEMENT DATA

Pavement Management Systems (PMSs) require data which can be used to trigger a decision as to when rehabilitation actions will be needed, as well as what level of repair is necessary. This information is used to prepare budgetary planning documents, to ensure that adequate levels of funding are available into the future, or to illustrate the impacts of anticipated funding levels on overall pavement condition. The following types of information are needed to sustain a PMS.

1. Pavement Quality Index. Evaluating the existing condition of pavements is the foremost need of a PMS. The results of an evaluation of the pavements in a network, such as Minnesota's pavement network, have to be expressed in some common form to allow a comparison of the serviceability of the different pavements. This comparison is done by the Department through the use of a Pavement Quality Index (PQI). The PQI is defined by Equation 4-3.1 as follows.

$$PQI = \sqrt{SR \times PSR} \qquad \text{Eq. 4-3.1}$$

where

SR = Surface Rating and
PSR = Present Serviceability Rating.

Following is a description of the distress survey that goes into the determination of the PQI in Minnesota.

a. Surface Rating. Mn/DOT utilizes the survey procedure described in the "Pavement Management Surface Rating Manual" to collect pavement distress information. The surface rating system can be used for bituminous, concrete, and bituminous-overlaid concrete pavements. Data is collected by the Pavetech van, which video tapes the pavement surface and the actual survey is performed at the workstation.

Data are collected annually on about 50 percent of the system. The sample size is 500 feet in each direction every mile.

At the present time, the annual survey does not require the manual computation of the Surface Rating (SR) for either bituminous or concrete pavements. The procedure, however, is described in the Pavement Management Surface Rating Manual. Following the procedures described, a numerical rating ranging from 0.0 to 4.0 is determined, with 4.0 being the rating for a pavement with no distress. It is discussed in greater detail in Section 4-3.03.01.

The automated Pavetech van provides many advantages to a highway agency. Primarily, these include the safety and efficiency of collecting data at or near travelling speeds, the consistency of data

obtained by photography of deterioration as it develops, and the resulting historical visual record.

b. Present Serviceability Rating. This value is calculated by correlating the International Roughness Index (IRI), which is measured with the Pavetech Van or South Dakota Profiler, into the Present Serviceability Rating (PSR). This correlation was developed by the use of a large panel of raters. The PSR is a numerical rating ranging from 0.0 to 5.0, with 5.0 being the rating for a perfect ride. It gives an indication of the rideability of a pavement from the perspective of the travelling public. It is discussed in greater detail in Section 4-3.03.02.

2. Construction As-built. Newly constructed pavements should also be evaluated in terms of serviceability rating standards prior to acceptance by the Department. Several ongoing studies are currently developing ways of relating certain critical pavement properties at construction (smoothness, strength, thickness, mixture variables) to their impact on the long-term performance of the pavement. Incentives and disincentives can then be distributed to the contractor, depending on whether the as-constructed values exceed or fall short of design values. All incentives and disincentives should be tied to the relative impact of the variable on the performance capabilities of the pavement.

3. COPES. The <u>CO</u>ncrete <u>P</u>avement <u>E</u>valuation <u>S</u>ystem (COPES), developed under NCHRP Project 1-19, was intended to be a means of storing a large amount of data to improve the knowledge of concrete pavement design, construction, and maintenance. As the study developed, six state-level demonstrations were conducted, including one in Minnesota. The COPES database is still being used in Minnesota, although modified and now in a FOCUS PC database format, and includes design data.

A review of the sample design data shows that the COPES database includes data elements in six areas. The first area, uniform section

information, includes network inventory data, such as district, surface type, previous surface, date of last construction, county, number of lanes, slab thickness, and whether or not the highway is divided.

Very comprehensive joint information is included in the second area of the design data. In addition to transverse joint spacing, skewness, presence of dowels, and detailed dowel design, the construction method used for transverse and longitudinal joints is included, as is tie bar design, joint sealant, joint dimensions, and shoulder/lane joint information.

The third area includes information relating to the reinforcing steel used in the concrete pavement. Included is the type of rebar, method of rebar placement, rebar yield strength, and transverse and longitudinal bar diameter and spacing.

Concrete data is featured in the fourth area of the design database. This area contains mix design data, compressive and flexural strength test results, material types, paving procedure, curing method, and finishing techniques.

The last two areas contain information on the base/subgrade, shoulder, and drainage. Data elements include base type, subgrade type, stabilized or unstabilized base thickness, shoulder surface type, shoulder width and thickness, base type, and type/location of any drainage elements.

4. Soils File. Existing soils files can be used to supply much of the needed information regarding the subgrade on which the pavement sections have been built. The COPES database permits storage of an extensive amount of information regarding subgrade material characteristics. Data relating to AASHTO soil classification; subgrade strength tests; swell potential; frost susceptibility; optimum, lab, dry density and moisture content; measured in-situ dry density and moisture content; plasticity index; and liquid limit can all be obtained from materials reports.

5. Friction Number. Friction testing must be performed at frequent intervals, in order to ensure the greatest potential for safety. Early friction or skid tests were conducted by locking the wheels of an automobile and measuring the distance traveled while sliding to a stop. On today's high volume roadways this method of testing is no longer feasible.

   Throughout the 1960s and 1970s, most highway agencies measured skid through the use of a locked-wheel trailer. This device features a primary unit which carries recording equipment and, in most instances, the water supply for wetting the pavement. A trailer, equipped with transducers to measure either the force required to pull the locked wheel, or the torque required to lock the wheel, follows behind. Both the speed of the vehicle and the trailer weight are typically constant. The computer, within the primary unit, integrates the force/torque-time relationship and

calculates a skid number, which can be loosely defined as the coefficient of friction, or "friction factor," multiplied by 100.

The most modern equipment features a fifth wheel on the primary unit instead of a trailer. This modification limits the calibration steps required to checking only the speedometer and the force/torque transducer.

6. Deflection Data. In order to predict the remaining life of pavements for pavement management and to identify feasible rehabilitation alternatives, reasonable assessments of the current strength of each of the pavement layers must be obtained. Non-destructive testing (NDT) has become the standard for determining such material property characteristics.

To use pavement deflection data to estimate strength, a known load is applied to the pavement surface and the resultant deflections under the load, and at controlled distances from the load, are measured. The values obtained are used to backcalculate estimates of layer moduli and load-carrying capacity.

There are many different types of NDT equipment in use today. Types of loading include static loading conditions, vibration loading, and instantaneous impulse loading, the latter of which has been found to more closely simulate the effect of a moving wheel load. Mn/DOT currently owns and operates several impulse loading testing vehicles, more commonly known as Falling Weight Deflectometers (FWD).

The FWD is a trailer-mounted device which drops a known load onto a load plate on the pavement surface and records the deflection of the pavement at various distances from the point of impact. The height that the weight is dropped can be varied to produce the loading required by the specific pavement type. Doubling the height of the fall quadruples the energy of the drop at impact.

To conduct a test, the FWD must be stationary while the test is actually being performed. In cases where it is not possible to obtain a lane closure otherwise, NDT can be performed at night when traffic volumes are at a minimum (although Mn/DOT does not currently test at night). Deflection measurements are automatically recorded and transferred directly to the on-board computer in the support van. Tests are usually performed at approximately 500-foot intervals, in the outer wheel path of the bituminous pavement sections being analyzed. Concrete pavements are typically tested along the transverse and longitudinal joints, in one corner, and in the center of the slab.

At the network level, NDT data can be used to identify potential sites for pavement rehabilitation, to determine relative priorities among projects, or to determine the differences in pavement behavior, which are caused by such factors as climate, traffic, or material types. In most cases, network-level analysis is used to rank sections as being stronger, or weaker, than other pavements of a similar type, in order to help determine priorities among project sections.

At the project level, NDT results are used to design maintenance and rehabilitation strategies, so a larger number of tests are conducted. The exact number of tests required depends on the level of reliability desired in the final design. For these types of projects, homogeneous sections are usually identified and classified as analysis units. NDT results may be used to determine the boundaries of these sections.

Additional discussion on FWD deflection measurements can be found in Section 4-3.03.04.

The measured test results are then used as the basis for a comprehensive analysis to determine the actual properties of the pavement layer materials. Several analysis methods are available for the back calculation of modulus values from deflection data. These include layered elastic and finite element computer programs, each of which has its own advantages and disadvantages.

Several layered and linear elastic, computer programs have been developed for pavement design. They are CHEVRON, ELSYM-5, and BISAR. BISAR is a proprietary program which must be obtained by each user from the Shell Oil Company. The other two programs are in the public domain. Several back calculation programs which utilize linear elastic techniques are MODULUS, CHEVDEF, and EVERCALC. These programs utilize the CHEVRON method. Another back calculation program, ELSDEF, utilizes the ELSYM-5 program. In addition to these programs are BOUSDEF and ELMOD which use the method of equivalent thicknesses and Boussinesq theory to perform backcalculations of FWD data.

Linear elastic analysis is based on five basic assumptions. The first is known as Hooke's Law, i.e., stress is proportional to strain in each pavement layer, and the proportionality constant is the modulus. Second, each layer is assumed to be homogeneous and isotropic. In other words, no cracks, voids, or open spaces should be present. Third, the analysis assumes that the pavement extends infinitely in both the horizontal and vertical directions. The fourth assumption is that the pavement surface is free from any stress or strain outside the loaded area of the NDT device. And, finally, the fifth assumption is that the vertical and shearing stresses and displacements are continuous across the interface of the pavement layers.

Finite element computer programs are also available to analyze pavement data; however, the selection of appropriate input data for their non-linear analysis requires experience and much engineering judgment. In addition, because of the considerable amount of calculations that are performed, very few versions are currently available for the personal computer.

Mn/DOT has several of these layered elastic computer programs available to perform the backcalculation of modulus values for flexible pavement systems. These programs include MODULUS, BOUSDEF,

EVERCALC, and ELMOD. The ELMOD program is a proprietary software package developed by Dynatest, the manufacturer of the FWD apparatus used by Mn/DOT.

For concrete pavements, ILL-BACK, ELMOD, ELCON and EVERCALC are examples of available software that can be used to back calculate the slab elastic modulus and subgrade modulus. The soil support k value can be indirectly determined. Manual solutions can also be used, in conjunction with equations or graphs that have been developed elsewhere.

Once the analysis is completed, the results of the back calculation analysis are converted into useful values for pavement design. Seasonal variations for asphaltic concrete surfaces are taken into account during the analysis, as are any correction factors that need to be applied to adjust for subgrade strength due to wetter conditions at certain times of the year. Typically, the asphaltic concrete Modulus of Elasticity, portland cement concrete (PCC) Dynamic Modulus, PCC Modulus of Rupture (determined from the dynamic modulus), and static modulus of subgrade reaction (determined from the subgrade resilient modulus, M_r) are used in any further analysis and design.

7. Traffic. Refer to Section 4-4.0.

4-3.03 FIELD RECONNAISSANCE

In order for the collected survey data to be valuable for pavement management purposes, correct operating procedures must be followed to ensure accuracy. The objective of this section is to discuss the field procedures which should be followed for the successful collection of the pavement data.

4-3.03.01 PAVEMENT DISTRESS

The procedures that should be followed for the Pavement Management Surface Rating Evaluation are discussed thoroughly in the "Pavement Management Surface Rating Manual." The road is videotaped by the Pavetech van, which is a survey vehicle which captures the video from four different cameras at highway speed. The actual survey is performed at the Pavetech workstation by a trained operator.

The Manual recommends that for each pavement segment being rated, the selection of areas to survey should be based on the following guidelines.

- Ratings are to be taken at the locations where the number of lanes or the surface type changes, and at other intermediate locations as determined by the district. These locations are referred to as "D" records. In addition, ratings should be taken at each reference post between D records, unless the reference post is less than 0.3 miles from a D record. Ratings at reference posts are designated as "M" records in the PMS.

- On two-lane roadways, the first 500 feet at each record location should be rated, in the increasing reference post direction.

- For all roadways other than two-lane roadways, ratings are taken for 500 feet in each direction at each record location. In the increasing direction, the rating is taken for the first 500 feet in the outside lane. In the decreasing direction, this would be accomplished by starting at a record location and rating forward toward the next record location 500 feet, in the outside lane. The values obtained between any two records are always recorded on the form at the lowest record location, no matter which direction of travel.

4-3.03.02 SERVICEABILITY

Pavement serviceability measurements provide an indication of the acceptability of a highway pavement to fulfill its intended function. Developed during the AASHO Road Test, the Present Serviceability Rating (PSR) is a subjective rating, which designates the condition of the pavement at any instant in time. The PSR is determined by a panel of individuals who rate the pavement on a scale from 0.0, which indicates an "impassible" road, to 5.0, which indicates a "perfect" pavement. When the rating is correlated with objective measurements made on the pavement surface (thereby eliminating the need for a rating panel), the resultant value is generally referred to as the Present Serviceability Index (PSI).

Mn/DOT has, however, chosen to refer to the PSR as the correlation of the objective measurements to the subjective rating. The objective measurement is performed by the Pavetech or the South Dakota Profiler, which produces an International Roughness Index (IRI) value. This IRI is then converted into the PSR by two correlation equations, one for concrete and one for bituminous.

4-3.03.03 FRICTION NUMBER

Friction numbers can be greatly influenced by a wide variety of factors, including whether the pavement is wet, muddy, or icy; the pavement surface texture and roughness; and tire speed, temperature, and inflation. Skid resistance is measured using a locked-wheel skid tester, as specified by ASTM E 274. The test should be performed at speeds approximating 40 miles per hour. The coefficient of friction, which describes the amount of friction between the vehicle tire and pavement surface, is multiplied by 100 to determine the skid number.

4-3.03.04 DEFLECTION MEASUREMENTS

Mn/DOT owns and operates several Dynatest Falling Weight Deflectometers. As discussed in Section 4-3.02.06, the amount of deflection testing required depends almost entirely on the purpose for which the data is being collected. If the testing is being done to get an overall assessment of the load-carrying capacity of the pavements within the state, or if NDT is being performed to prioritize rehabilitation sections, a much less rigorous testing frequency can be utilized than one in which a rehabilitation design is being developed.

The FWD is an impulse loading device which exerts a force similar in magnitude and duration to that of a moving wheel load. It does this by dropping a weight onto a spring connected to a load plate of known dimensions that sits on the pavement. By varying the weight of the load, and the height from which the load is dropped, the magnitude of the load can be changed. The resulting pavement deflection is measured by seven seismic deflection transducers, one of which is located at the center of the loading plate, with the remaining sensors spaced in a straight line away from the load. It is expected that in the future additional sensors will be added to the Dynatest Falling Weight Deflectometer and the backcalculation methods will be modified to reflect this change. Mn/DOT presently uses seven sensors spaced at variable distances from 0 to 2000 millimeters.

Testing patterns vary, but typically involve a 100- to 200-foot spacing for project-level testing and a one-tenth mile spacing for network-level testing. For bituminous pavements, testing is generally performed within the outer wheel path of the outer lane. For concrete pavements, test locations are at the center of the slab for determination of the layer modulus values, at the corner for void detection, and at joint locations for determining load transfer efficiencies across the joints. At least three different load levels are required to evaluate any loss of support beneath slab corners. Mn/DOT generally uses drop heights which simulate 6,000-, 9,000-, and 12,000-pound loads. For other routine testing, a simulated 9,000-pound wheel load is all that is necessary. Additional load levels will aid in the evaluation of the layer moduli.

The strength characteristics of asphaltic concrete pavements are temperature dependent, meaning that they change as temperatures change. At higher temperatures, the modulus of an asphaltic concrete mix tends to decrease. As the temperature cools, the mixture becomes stiffer, resulting in higher moduli values. Because of this, deflection measurements from asphaltic concrete pavements must be normalized to a standard temperature. This requires the recording of pavement and air temperatures at the time the testing is taking place.

Concrete pavement testing is also affected by temperature due to warping and curl of the slab.

Subgrade strength generally varies with the time of year when the testing is performed. These variations are greatly attributed to the amount of moisture prevalent within the soil during a particular time of year or season.

Testing during the winter months, when subgrades could be frozen, should be avoided. Conservative test results will be obtained by performing the FWD testing during the spring months when the subgrade is thawing and typically at its weakest.

4-3.03.05 DRAINAGE

The presence of water within the pavement structure is a major contributor to pavement deterioration. It is extremely important, therefore, to evaluate the effectiveness of the pavement drainage system in evaluating pavement

performance with time, as well as in the identification of feasible rehabilitation options.

There are several drainage components which must be evaluated in a comprehensive pavement evaluation. These include cross slopes, the condition of the drainage outlets, the presence of any pumping through cracks and/or joints, faulting at joints or cracks, and the presence of standing water in ditches. The following areas should be checked as a part of a pavement evaluation, in order to determine the condition of the drainage mechanisms within the pavement structure.

1. Pavement Surface. Pavement distress is typically caused by either a load-related mechanism, a construction- or material-related defect, or some type of moisture-related mechanism. The presence of certain distresses, commonly identified as part of the Pavement Management Surface Rating Survey, are important indicators of the presence of water within a pavement structure.

   The presence of any pumped material along the side of concrete pavement joints or cracks is a strong indicator that there is a moist subgrade that is pumping its way out as loads are applied to the pavement. With time, this will lead to the formation of a void under the slab that will not be able to withstand the continual loads being applied to it, resulting in an advanced rate of pavement failure. Other distresses, such as stripping in AC pavements and D-cracking in PCC pavements, are also strong indicators that water is present in the pavement.

   An effective surface drainage study should consist of the notation of any signs of excessive water in the pavement structure. The presence of the above-mentioned distresses should be noted, and further pavement evaluation studies should be conducted prior to an overlay design, to identify the extent of the problem. Cross slopes should also be noted, to determine the effectiveness with which water is drained off of the pavement surface into ditches.

2. Ditches. In addition to the pavement surface, the ditch condition should also be noted, for its condition can provide a good indication of the effectiveness of a drainage system. If a ditch is clogged with weeds, debris, or some other type of material deposit, water cannot flow as it was originally designed. The presence of standing water in a ditch also indicates that the ditch is not fulfilling its intended function. The distance from the pavement surface to the bottom of the ditch is also important, as it is an indication of the drainage distance and the overall capacity of the drainage system.

   The slope and size of the ditch are also important. If ditch slopes are flat, or if a reverse slope exists, water will tend to soak into the subgrade soil and saturate it. This limits the ability of subsurface moisture to drain away from the structure, again eventually softening the subgrade and reducing the load-carrying capacity of the pavement.

3. Culverts. The condition of culverts should be inspected and maintained regularly. Large flow periods often carry brush, branches, and other large objects that can clog the entrance to a culvert, thereby raising the headwater elevation until flooding results. As a way of evaluating the culvert condition, the presence of any standing water or debris should be noted. In addition, the condition of the grates and lead-in ditches should be noted.

4. Subsurface Drains. The condition and operation of any in-place subsurface drain on the project (pavement edge drain, subcut drain, old tile drain, ground water control, etc.) should be evaluated. Improper functioning of such drains can lead to the conditions described in paragraph 1, above. Look for plugged, crushed, or totally absent discharge points; settlement/distress along shoulders over drain locations; presence of white carbonate or rusty iron deposits at discharges; water standing on or staining of bituminous shoulders; or any other signs of improper operation.

4-3.03.06 MAINTENANCE INPUT

Another valuable source of information pertaining to pavement performance is maintenance records that identify the type and extent of maintenance that has been performed on a given pavement section. Notes taken by maintenance personnel at the time of routine maintenance are often more up to date than the pavement condition information stored in the PMS. Historical maintenance data, typically obtained from a maintenance management system, can also provide an insight into the ability of the pavement design to carry out its intended function.

4-3.04 TESTS

Material characterization is another important type of information to assess the overall condition of a pavement network. These test results are used to supplement NDT data and provide information on the quality of a specific material, in order to assist in the selection of feasible rehabilitation strategies. Especially at the project level, the following tests provide valuable information to the rehabilitation designer.

4-3.04.01 EXTRACTIONS

Extractions of the asphalt materials in the pavement layer allow assessment of certain material properties. These include the consistency of the asphalt binders, the ductility, and the amount of asphalt present in the mix.

4-3.04.02 GRADATIONS

It is important to know the current gradation of the base material in order to help explain overall pavement performance. Primarily, gradation tests will assess the stability of the base and its ability to drain water, which in turn have a large impact on performance.

4-3.04.03 MOISTURE AND DENSITY

Tests which provide an indication of the moisture content and density of the material are also an important part of a pavement evaluation. These tests can provide an indication of material deficiencies that will need to be addressed prior to the application of a rehabilitation treatment.

4-3.04.04 STRENGTH TESTING

Material strength testing is an important supplement to the results of NDT. The R value or M_r provide a good indication of the ability of the pavement to withstand anticipated loads.

4-3.05 SUMMARY

As discussed in this section, it is necessary to determine key information about a pavement so that a comprehensive pavement evaluation can be conducted. Based on this evaluation, a rehabilitation designer can assure that the cause and extent of the deterioration will be assessed and prevented from reoccurring in the future. Results from previous office studies (Section 4-3.02), field studies (Section 4-3.03), and testing (Section 4-3.04) should all be used in the pavement management process.

4-4.0 TRAFFIC ANALYSIS

4-4.01 PURPOSE

Traffic analysis is an essential part of project feasibility studies, project selection, and project path analysis; in determining the type and sizing for facilities, interchanges, and pavement structures; and in studying the relationship between past traffic and anticipated pavement life. Since the scope of this manual encompasses only the geotechnical and pavement design aspects of Mn/DOT's varied program, the following sections will discuss sources of traffic data and the process of forecasting cumulative Equivalent Single Axle Loads (ESALs). Please contact the Traffic Forecast and Analysis Section for information regarding additional uses of traffic data and methods to produce traffic forecasts for purposes not covered in this manual.

4-4.02 OFFICE STUDIES

Most traffic analysis begins with an effort to quantify and understand current and past traffic volumes, characteristics, and patterns; as they pertain to a specific project location. Traffic data has been routinely collected by Mn/DOT and county and municipal agencies for internal use and for reporting to external agencies (i.e., traffic data is sent to the FHWA for inclusion into the Highway Performance Monitoring System - HPMS). Annual and bi-annual summary reports of traffic data are available through the Traffic Forecast and Analysis Section. Traffic and ESAL forecasts, or special traffic studies, can be requested from the same section when additional information is required for a specific project or research inquiry. Short descriptions of the various types of traffic data, data collection equipment, reports, and the ESAL forecasting process are contained in the following sections.

4-4.02.01 CONTINUOUSLY COLLECTED TRAFFIC AND WEIGHT DATA

Over 160 continuous, traffic volume, collection sites are scattered throughout the state and are located, primarily, on the trunk highway system. Most of the continuous sites are equipped with inductance loops in the pavement and utility cabinets along the right-of-way, containing circuitry that powers the systems, converts the signals from the pavement sensors into vehicle data, and permits the transfer of lane and time period specific data via telemetry. Inductance loop sites are typically referred to as Automatic Traffic Recorders (ATRs). An increasing number of continuous sites are being added, but these sites are equipped with additional permanent weighing-in-motion (WIM) devices that permit the recording of axle weights, classification of vehicles, vehicle speed, and vehicle length, for each lane and for predetermined time periods. The sites of the second kind are typically referred to as WIM sites and represent Mn/DOT's continued support of the Strategic Highway Research Program in cooperation with AASHTO. In the future, inductance loops and other sensors that have been installed as parts of centralized, traffic control systems may double as ATRs, further enhancing knowledge of traffic volume characteristics.

Data from the continuous sites occasionally contain gaps caused by malfunctions. Estimates for the missing data are usually made by analysts using data from equivalent time periods at the same location.

Data from many of the continuous sites currently provides the basis for determining the statistical variability of counts of shorter duration and for determining time based factors to adjust short counts to estimate Annual Average Daily Traffic (AADT). A more detailed description of the short duration count and adjustment program can be found later in this section. As more WIM sites are installed throughout the state, analysts will incorporate more continuous vehicle classification data into new systems designed to adjust short duration vehicle class counts.

Axle weight data obtained at continuous WIM sites has contributed greatly to our understanding of how ESAL factors can vary through time and between locations. Additionally, vehicles are weighed without delay or knowledge of the weighing, thereby eliminating bias that could be attributed to the measurement process itself. Due to the variability introduced by the dynamics of trucks traveling at highway speeds and by the inherent limitations of scale designs, calibrated WIM installations have, at best, observed errors of approximately ten percent for individual axles and approximately four percent for gross weights (according to field studies comparing highly precise static weights of five-axle tractor, semi-trailer trucks and correlated weights from WIM). Initial scale calibration is attained when the average axle weights of heavy test vehicles, as measured by the WIM equipment, equal known axle weights.

Various reports are available for continuous traffic data depending upon the instrumentation at each site and method used in polling the data. An analyst can obtain hourly data for any specific hour or aggregation of hours within a calendar year if research warrants such detail. Routinely generated reports have included the "ATR Data Summary" and the "Weighing-In-Motion Data" reports. Both reports contain AADT, average monthly volume information, and day of the week average volume information. Weighing-In-Motion reports also contain volume and ESAL factor information by specific vehicle classification distinctions, but they do not contain "Nth" highest hourly volume and directional distribution information as the ATR Data Summary reports do.

4-4.02.02 SHORT DURATION COUNTS AND EXPANSION PROGRAMS

Over 45,000 short duration counts are taken at selected points on the trunk highway, county state aid highway, county road, municipal state aid, city street, and township road systems. These counts, together with short duration count expansion programs, provide the basis for the creation of AADT and Heavy Commercial Average Annual Daily Traffic (HCADT) flow maps, and/or documentation of trends in traffic for certain types of roadways. While other short duration counts are made by the Department (i.e., 12-hour turn movement counts), only those short counts that are integral to the production of AADT and HCADT flow maps and trends are discussed in this section. AADT flow maps are produced every two years for the trunk highway system and every four years for the county road system. In the Minneapolis/St.Paul metropolitan area, county road AADTs are also available every two years. HCADT flow maps are available

for the trunk highway system only (HCADT represents commercial vehicles with six tires or more).

Short duration sampling has traditionally been limited by human and equipment considerations. Time periods of 16 and 48 hours remain the most practical time periods for manual and portable systems, respectively. Short duration sample sites are chosen by analysts from the Traffic Forecast and Analysis Section and selected count requests are sent to District Office personnel for counting between the months of April and October of each year.

The overwhelming majority of short duration, traffic volume data collection sites are counted with portable pneumatic-tube counters for a period of 48 hours. Results from each count are sent to clerks and analysts working for the Traffic Forecast and Analysis Section where the counts are recorded, adjusted to AADT, and transcribed onto work maps in preparation for flow map creation. In adjusting a pneumatic-tube count to estimate an AADT at that location, factors are applied to correct for the day of the week, month of the year, and vehicle classification mix. ATRs, singly or in groupings, based on time/volume similarities, provide the data from which year-specific, time-related, adjustment factors are calculated.

The vehicle classification count program provides the basis for axle correction factors that convert the total number of "axle hits" recorded by the pneumatic-tube counting equipment to an estimated number of vehicles (see Table 4-4.1 for a partial key to vehicle classification). Axle correction factors were first applied to trunk highway pneumatic-tube counts in 1986. No axle correction factors are applied to county road counts when AADTs are calculated on those systems.

Table 4-4.1. Vehicle classification.

Type Number	Illustrated Example	Vehicle Description
1		Passenger Cars
2		Panel and Pickups (under 1 ton)
3		Single Unit - 2 axle, 4-tire
4		Single Unit - 2 axle, 6-tire
5		Single Unit - 3 axle and 4 axle
6		Tractor semitrailer Combination - 3 axle
7		Tractor Semitrailer Combination - 4 axle
8		Tractor Semitrailer Combination - 5 axle
9		Tractor Semitrailer Combination - 6 axle
10		Trucks with Trailers and buses

Vehicle classification counts are taken at over 1,000 locations in the state during a six-year cycle. Approximately 95 of these locations are sampled every two years, with approximately half of those sites being visited twice. A count duration of 16 hours (two eight-hour shifts of 6 a.m. to 2 p.m. and 2 p.m. to 10 p.m.) is a standard sampling period for the vehicle classification count program. Adjustments are made to the edited, raw, vehicle classification data using factors calculated from continuous vehicle classification data so that the effects of weekends and the eight remaining hours in a 24-hour day are accounted for. Adjusted vehicle classification data is next increased or decreased to match an official AADT from completed traffic flow maps. Reports containing raw and adjusted vehicle classification count data are available each year from Traffic Forecast and Analysis Section personnel.

4-4.03 CUMULATIVE EQUIVALENT SINGLE AXLE LOAD FORECAST PROCESS

4-4.03.01 PROCEDURE

The process of forecasting cumulative Equivalent Single Axle Loads (ESAL) for use in pavement design and selection is one which contains four major parts. This process provides a common framework for all ESAL forecasts in the state. District traffic forecasters, the Traffic Forecast and Analysis Section, and consultants working for Mn/DOT, can all participate in the production of ESAL forecasts, depending upon a particular project's schedule, complexity, and staff commitments. The four parts of this process are:

- Determination of two-way Average Annual Daily Traffic (AADT),
- Determination of two-way vehicle classification volumes,
- Determination of flexible and rigid ESAL factors, and
- Determination of the design-lane distribution of ESALs.

An electronic spreadsheet called MNESALS, that can help an analyst produce a cumulative ESAL forecast which is constructed using all four parts of the ESAL forecasting process, is available from personnel in the Traffic Forecast and Analysis Section. The spreadsheet operates on IBM DOS-based personal computers and uses LOTUS 1-2-3, version 1.0 or higher. Please refer to this spreadsheet and its user's manual for details regarding specific forecast calculations.

Simply following the process described in the upcoming section does not, however, guarantee that the forecasts are adequate. The analyst must choose <u>appropriate</u> AADT and vehicle classification volumes, lane distribution factors, and ESAL factors for the current year, for the year the highway opens (the base year), and for 20 years after opening (the design year). As with any other aspect of the roadway design process there will always be a degree of uncertainty associated with the analyst's estimates and projections. The Traffic Forecast and Analysis Section staff will offer assistance to analysts performing ESAL forecasts and approve forecasts that are procedurally and methodologically sound.

4-4.03.02 DETERMINATION OF TWO-WAY AADT

The first part of the ESAL forecasting process provides the analyst with a basic constraint for the current year and each of the forecast years. Regular traffic counts and AADT estimation throughout the state have provided the analyst with a reasonable historical record of highway use. Since step two of the ESAL forecasting process requires that AADT be separated into different vehicle types, the determination of current and forecast AADT sets relative limits on particular vehicle classification volumes.

The Analyst performing an ESAL forecast must first evaluate AADT and/or Heavy Commercial Average Daily Traffic (HCADT) forecasts that may already have been approved during the Preliminary Design phase of a project's development. If no AADT forecasts exist for a project, the analyst determining AADT can employ any or all of the following AADT forecasting methods, depending on the location, scale, complexity and importance of the forecast. The analyst may obtain detailed descriptions of these methods and data collection techniques associated with each method from Traffic Forecast and Analysis Section personnel.

- Inferred growth rate;
- Project AADT as a function of population, employment, or other independent variables;
- Least squares projection using historic AADT. (Trunk highway AADTs prior to 1986 generally require axle correction factor adjustment.);
- Analysis of trends and patterns within a travel corridor; and
- Use of computer generated travel demand estimates.

Mn/DOT traffic flow maps (available in Room B-20 of the Transportation Building, St. Paul) provide current estimates of AADT for the trunk highways, county road systems, and the municipal state aid system. If necessary, the Analyst can request additional AADT estimates by contacting the Traffic Forecast and Analysis Section staff.

4-4.03.03 DETERMINATION OF TWO-WAY VEHICLE CLASSIFICATION VOLUMES

The second step of the ESAL forecasting process provides the analyst with volume estimates for each classification of vehicle for the current, base, and design years. The number of vehicle classifications depends upon the number of uniquely defined, average ESAL factors. In Minnesota there are eight sets of average ESAL factors that are generally used in ESAL forecasts. Number of axles and body configuration usually provide the basis for defining vehicle classifications when quantifying ESAL factors.

Occasionally, the analyst will choose to deviate from the eight vehicle classifications when additional average axle weight and axle configuration data is available. By making the distinction between fully loaded trucks in one vehicle

classification category (maximum weight per axle permitted) and the trucks remaining within the same category, the analyst can substitute two classification categories for the original category in an ESAL forecast. In this way, the analyst can use knowledge of local trucking characteristics and haul direction to substantiate the use of project-specific ESAL factors.

Since each road segment in the state has a distinct mix of vehicle types, this step of the ESAL forecasting process requires that the analyst evaluate existing vehicle classification data for the highway project segments, or segments close to the project. Vehicle classification site listings and maps are available from personnel in the Traffic Forecast and Analysis Section. If no data exists on or near the project, the analyst should request that additional vehicle classification data be collected. If data cannot possibly be collected, the analyst may refer to Table 4-4.2 for values that may be used to estimate vehicle classification volumes.

Table 4-4.2. Vehicle classification volumes.
Revised February 1, 1997

VEHICLE CLASS PERCENT OF AADT *Analysis of 1983-1989 Data*							
Vehicle Classes	Trunk Highways Greater Minnesota			Trunk Highways Mpls/St. Paul Seven County Area			Rural CSAH or County Road
	Average	Min/Max	95% of data sites	Average	Min/Max	95% of data sites	Average
Autos, pickups	89			93			94.1
2 ax, 6 tire SU	2.7	.4 / 9.3	< 4.9	2.2	.8 / 7.6	< 3.6	2.6
3 + ax SU	1.5	0 / 28.7	< 4.1	1.0	0 / 8.9	< 2.8	1.7
3 ax TST	.1	0 / 1.1	< .3	.2	0 / 1.2	< .4	0.0
4 ax TST	.2	0 / 1.6	< .6	.2	0 / 1.1	< .6	0.1
5 + ax TST	6.1	0 / 31.0	< 15.5	3.2	0 / 22.6	< 10.6	0.5
Buses, Trucks w/Trailers	.4	0 / 3.9	< 1.2	.3	0 / 2.7	< .9	1.0
Twin Trailers	.1	0 / 1.0	< .3	.0	0 / .4	< .2	0.0
Total Number of Sites	N = 837			N = 239			1977 County Road Study

SU = Single Unit Trucks
TST = Tractor Semi-Trailer
ax = axle

In forecasts involving new alignments, the Analyst occasionally needs additional data that defines truck classification and travel patterns. Additional data requests should be made to staff in the Traffic Forecast and Analysis Section to ensure that acceptable data collection methods and adjustment procedures are used.

After determining current vehicle classification volumes for a site on a proposed highway project, the analyst needs to determine how the volumes for each classification may change in the future. If historical data from the vehicle classification data record suggests that each vehicle classification volume as a percent of AADT has remained relatively stable through time, then the analyst can usually assume that future vehicle classification volumes can be estimated by multiplying each vehicle classification percent of current AADT by the AADT for the future year. Considering a different example, if historical vehicle classification data or anticipated land development along a proposed highway project suggests that passenger vehicles will increase or decrease proportionately faster than truck classifications, then the analyst can project truck classification volumes differently when compared to the AADT projections, depending on the assumptions guiding the analysis. Whatever method is used for projecting future vehicle classification volumes, the analyst must document the assumptions that are part of the forecasts.

4-4.03.04 DETERMINATION OF FLEXIBLE AND RIGID ESAL FACTORS

The third part of the ESAL forecasting process involves the use of average vehicle ESAL factors (formerly known as Sigma N-18 factors). These factors represent daily, axle-based, ESAL factors, summed by vehicle within a vehicle classification and averaged within a year, at one or more weighing-in-motion (WIM) data collection sites. Flexible pavement ESAL factors (also known as BESALs) for each axle are calculated using a structural number of five, and a terminal serviceability of 2.5. Rigid pavement factors (also known as CESALs) are calculated using D=8, and a terminal serviceability of 2.5. The 1986 American Association of State Highway and Transportation Officials (AASHTO) "Guide for Design of Pavement Structures," Appendix D, contains ESAL factors for additional axle configurations, structural numbers, slab thicknesses, and terminal serviceability values.

Most WIM sites are located on roadways with ten-ton, single-axle, weight limits with no springtime restrictions. TH 99 has a seven-ton, springtime axle weight restriction. As can be seen in Table 4-4.3, average ESAL factors for each vehicle classification can vary significantly depending upon where the data is collected. Unless detailed axle weight, commodity and truck body type, and haul direction data is available, the analyst should use the default ESAL factors from Table 4-4.3.

Table 4-4.3. Average ESAL factors by vehicle classification from WIM.
Revised February 1, 1997

Flexible ESAL Factors for the Design-Lane						
		TH 99	I-494	I-94	TH 2	I-94
Vehicle Class	Default	a	b	c	d	MNROAD
Cars, Pick-ups	.0007					c
2 ax 6 tire SU	.25	.26	.13	.22	.26	.25
**3+ ax SU	.58	.51	.63	.44	.71	.61
3 ax TST	.39	.26	.21	.55	.36	.59
4 ax TST	.51	.57	.35	.47	.50	.60
5+ax TST	1.13	.92	1.14	1.00	1.74	.99
*(6 ax TST)	.78	.42	.74	.64	.69	.69
T w/tr, buses	.57					
MTC buses	1.25					
Twin Trailers	2.40	.49	.77	2.16	1.90	3.15
Rigid ESAL factors for the Design-Lane						
Cars, pick-ups	.0007	a	b	c	d	c
2 ax 6 tire SU	.24	.25	.12	.21	.25	.24
3+ ax SU	.84	.73	.93	.62	1.09	.90
3 ax TST	.37	.25	.20	.53	.34	.56
4 ax TST	.53	.60	.35	.49	.52	.63
5+ax TST	1.89	1.52	1.90	1.66	2.94	1.64
*(6 ax TST)	.80	.51	.89	.77	.83	.83
T w/tr, buses	.74					
MTC buses	1.23					
Twin Trailers	2.32	.47	.73	2.09	1.84	3.06

Vehicle Classes:
SU = Single Unit Trucks
TST - Tractor Semi-trailer
ax = axle
WIM = Weighing-in-Motion (fixed sites)
T w/tr = Trucks with trailers
MTC = Urban transit buses

Data Taken From Years:
a = 1985, 1990
b = 1982-1985, 1990
c = 1990
d = 1984, 1985, 1990

* - This vehicle class is not usually considered separately in an ESAL forecast.
**- Use 0.91 if the route is a sugar beet hauling route.

The default ESAL factors were determined after analysis of data taken from the lane at each WIM site that records the highest total ESALs. Analysis of historic ESAL factors from the WIM sites supports the premise that ESAL factors for individual vehicle classifications are relatively stable in Minnesota. However, roadways that serve points of transfer between modes of transport and port facilities can experience fluctuating average ESAL factors for trucks from year to year depending upon the transport mode and the port that best serves the market for particular commodities. Additionally, since laws regulating axle load limits for trucks using trunk highways are occasionally relaxed, 20-year cumulative ESAL forecasts are increased by 12 percent in anticipation of future relaxations.

4-4.03.05 DETERMINATION OF THE DESIGN LANE DISTRIBUTION OF ESALS

The fourth part of the ESAL forecasting process requires the Analyst to quantify how much of a single direction's ESALs are carried in a single lane during an average day. Based on data from vehicle classification sites, lane distribution studies, and WIM data analysis, the Traffic Forecast and Analysis Section recommends that the analyst performing an ESAL forecast use a design lane factor of 1, 0.9, or 0.7 for one, two, or three lanes in a single direction, respectively. When using two-way traffic volumes in the forecasts, use 0.5, 0.45, or 0.35 for two-, four-, or six-lane, two-way facilities, respectively.

Please remember that factors and values found in these sections are subject to change each year. If you have requested copies of tables or spreadsheets in the past, please note the date associated with the tables and spreadsheets to determine whether you have the most current versions.

4-4.03.06 SAMPLE CALCULATION

The following calculation, shown in Table 4-4.4, assumes that all values have been judged to be accurate and reasonable. Moreover, the procedure shown should be followed only if the analyst does not have access to the MNESALS electronic spreadsheet available from personnel in the Traffic Forecast and Analysis Section. The spreadsheet will produce accurate calculations for a cumulative 20-year ESAL. The following process approximates the calculation performed in the spreadsheet.

Table 4-4.4. Sample ESAL calculation.
Revised February 1, 1997

Example 20-Year Design Lane Cumulative ESAL Calculation (Use only if you do not have access to the MNESALS electronic spreadsheet)							
Vehicle Classes	Base Year AADT (Two-Way)		Flexible ESAL Factors		Base Year ADL	Design Year AADT (Two-Way)	Design Year ADL
Cars and Pick-ups	1207	x	.0007	=	.8	1690	1.2
2 ax, 6 tire SU	98	x	.25	=	24.5	137	34.2
3+ ax SU	34	x	.58	=	19.7	48	27.8
3 ax TST	6	x	.39	=	2.3	8	3.1
4 ax TST	8	x	.51	=	4.1	11	5.6
5+ ax TST	120	x	1.13	=	135.6	168	189.8
Buses, trucks w/trailers	25	x	.57	=	14.2	35	20.0
Twin Trailers	2	x	2.40	=	4.8	3	7.2
TOTAL	1500				206	2100	288.9

ADL = Average Daily Load

(Base Year ADL + Design Year ADL) / 2 = (206 + 288.9) / 2 = 247 (rounded)
Number of days in 20 years x 247 = 7300 x 247 = 1,804,335
Design Lane Factor (if 4 lane) x 1,804,335 = .45 x 1,804,335 = 811,951
Load limit increase factor x 811,951 = 1.12 x 811,951 = 909,385
Cumulative 20-Year Design Lane Flexible ESAL (rounded) = 909,000

Procedures for calculating a cumulative 20-year Design Lane Rigid ESAL are the same as above except rigid ESAL factors are used in place of the flexible ESAL factors.

REFERENCES

American Association of State Highway and Transportation Officials, Testing Standards, Washington, D.C.

Central Mine Equipment Company, "Drilling Safety Guide," St. Louis, MO.

Department of the Interior, Bureau of Reclamation, "Driller's Safety Manual," U.S. Government Printing Office, Denver, 1982.

Minnesota Manual on Uniform Traffic Control Devices (MMUTCD) Appendix B, "Traffic Control for Short Term Street or Highway Work Zones," March 1990.

United States Army, FM 5-541, "Military Soils Engineering," May 1986.

GEOTECHNICAL AND PAVEMENT MANUAL

Part 2

MINNESOTA DEPARTMENT OF TRANSPORTATION

Chapter Five
DESIGN

5-1.0 DESIGN CONSIDERATIONS

The activities involved in the provision of pavements can broadly be divided into three groups. First, there is the geotechnical design of the natural roadbed soil to obtain a suitable foundation for the pavement. This is followed by the design, construction, and upkeep of a pavement structure built on top of the foundation. Last, but not least, is the design and provision of drainage facilities to direct surface and subsurface flows of water away from the pavement structure, to prevent saturation of the pavement. These functions, in essence, encompass the activities of an engineer after the decision has been made to provide a pavement at a given location, suitable for travel at an acceptable level of service throughout its design life. These three aspects of the provision of a pavement are discussed in detail in the subsequent sections of this chapter.

Prior to the decision to actually proceed with the design of a pavement, however, a number of pertinent factors, which have an influence on the design process, have to be addressed. These range from a consideration of the source of funding for the project to the special guidelines that have to be followed, depending on the urban or rural location of the project. To address such factors, knowledge of the functions and responsibilities of the Department, and of the activities of the various Federal, State, and local agencies which may have an influence on the design process, is paramount.

Chapter 1 of Mn/DOT's Road Design Manual provides an overview of the different offices of the Department, their activities, the working relationships between the different groups, and the resources available from them. Similar information is given on the Federal, State, and local agencies. Guidelines are also provided on the development process for highway projects, including information on the preconstruction role of Mn/DOT. Given the increasingly complex nature of the design process due to environmental concerns, government regulations, and the difficult engineering, funding, and management decisions that have to be made, this information is very useful to the engineer.

A background on the design controls and criteria for highways in Minnesota can also be found in Chapter 2 of Mn/DOT's Road Design Manual. This includes information on the classification systems used by the Department in the application of non-highway design criteria, the establishment of jurisdictional authority, and the determination of the available sources of funds for projects. Tier standards adopted by Mn/DOT for two-lane and multi-lane rural highways, to provide the most appropriate dispensation of funds for the upkeep of the highest mileage of roadway structures at a high level of service, are also discussed. A knowledge of this information is necessary for the design processes discussed in this chapter to be in accordance with the goals of the Department.

5-2.0 GEOTECHNICAL DESIGN

5-2.01 EXCAVATION AND EMBANKMENT CONSTRUCTION

It is the responsibility of the District Soils or Materials Engineer in conjunction with the Design Engineer to assure to the maximum extent possible that the finished grade elevation is kept at least five feet above the water table (at least equal to the depth of frost penetration) to minimize frost heaving and thaw weakening of the subgrade. For silty soils the elevation should be significantly higher. It is also the responsibility of the District Soils or Materials Engineer to evaluate and provide recommendations for excavation and the placement of embankment materials. The existing soils or materials and their preparation, including subgrade correction, embankment placement, and protection of the completed embankment, all need to be considered and included in the recommendations. Each of these topics is discussed in the following sections.

In general, embankment materials and construction should comply with Mn/DOT's Specification 2105.

5-2.01.01 SOILS EVALUATION

It is the responsibility of the District Soils Engineer to evaluate the soils, classifying and designating those materials which are to be used within the various portions of an embankment. The following paragraphs discuss the types of soils which may be encountered along the proposed alignment and which need to be evaluated.

1. Suitable/Unsuitable Materials. Materials classified as "suitable" for embankment fills generally consist of granular, select granular, and common materials with an R value of at least the design value. Soils which would normally be suitable, but are less desirable, may be classified unsuitable for a particular project, if an abundance of better material is available.

   Topsoil, highly organic materials, debris, and other generally unstable soils are always classified "unsuitable," but may be utilized in certain portions of an embankment when not enough suitable material is available (Section 5-2.02.03).

2. Excavated Soils. Excavated soils from cut sections should be used to the fullest extent practical and so far as the excavated material is designated suitable with respect to the requirements of the project. See Mn/DOT's Specification 2105.3D.

3. Salvaged Materials. Existing aggregate and topsoil should be salvaged and reused in new construction where possible. See Mn/DOT's Specification 2105.2C.

4. Borrow. Borrow consists of material required for embankment construction that would not otherwise be obtained from roadway cut sections. It is the responsibility of the District Soils or Materials Engineer

to list, locate, test, and survey locations suitable for the required borrow (Section 4-2.03.08). Borrow material should be classified as select granular (Mn/DOT's Specification 3149), common, or topsoil (Mn/DOT's Specification 3877).

The District Design Engineer is responsible for determining the required quantities of materials for embankment construction, using shrink/swell factors provided by the District Soils Engineer. Shrink/swell factors are determined by locating the source of materials, either from cut sections or borrow areas, as described above; and then estimating the volume change of these materials from their undisturbed condition to their reworked and compacted condition. This is referred to as the shrink (if the compacted density is more than the in-situ density) or swell (if the compacted density is less than the in-situ density) of the material.

A shrinkage factor of 100 percent means that a given material will occupy the same volume when placed and compacted in the roadway that it did in the ground prior to excavation. A factor greater than 100 means more borrow or excavation material will be needed to build the planned embankment. A factor less than 100 percent indicates that the natural material will swell.

Other factors besides compaction of the excavated material will affect the final shrinkage factor. The primary complication is the consolidation of the underlying foundation soils by construction equipment. This can be on the order of one-half to one foot in tilled fields, which can have a dramatic effect on grading jobs involving shallow cuts and fills; on the other hand, old road cores and deep cuts may be quite dense. The shrinkage factor then is the sum of the compaction factor plus other factors. If varying conditions are encountered, more than one shrinkage factor will be required.

The District Soils Engineer should indicate in the Design Recommendations Report the basis for the shrinkage factors used. The preferred method of determination is to obtain undisturbed soil samples of the in-place soils and measure the in-situ density and moisture content. The undisturbed soil samples can be taken by means of sampling the surface soils or in shallow excavations. In the case of plastic soils, undisturbed thin-wall tube samples can be obtained at depth by District or Foundation Unit drill rigs. Densities of deep granular soils can only be estimated by means of split tube blow counts or ease of drilling.

These in-place values should then be compared against the Proctor density/moisture values to arrive at a compaction factor. Although Mn/DOT's specifications may call for 95 or 100 percent of T99 (Method C) density, it must be realized that the construction target density will be some value above the specification density due to normal statistical scatter. Investigation 183 data indicate that plastic fills are generally placed about three percent above specification density, while granular fills are generally about four percent higher.

After calculation of the compaction factor is made, then the other factors should be added to arrive at an estimated shrinkage factor (Shrinkage Factor = Compaction Factor and Other Factors). As indicated above, the primary complicating factor is the consolidation of the underlying foundation soils by construction equipment.

Shrinkage factors have, in the past, included the disposal of unsuitable material. Such a practice should be discontinued. A separate calculation based on available soils borings should be made of the amount and limits of unsuitable materials (primarily organic soils) which cannot be utilized on the job. Likewise, estimates of swamp shrinkage should be made separately, due to complicating factors such as vertical subsidence and lateral compression. Guidance in this area may be obtained from the Foundations Unit (Office of Materials Research & Engineering).

Where rock cuts are involved, the Geology Unit (Office of Materials Research & Engineering) should be contacted for an estimate of swell. Generally, rock shrinkage factors will be less than 100 percent.

Depending on the project location and availability of borrow or disposal sites, it may be better to skew the final shrinkage factor to one side or another. Where excess or deficient material must be truck hauled long distances, as in urban areas, special attention must be placed on the assignment of a shrinkage factor.

In the case of smaller projects, or if adequate exploratory resources are not available, it may be necessary to make an estimate of the shrinkage factor using other sources as well as past experience. A primary source of in-place moist density data is the U.S. Soil Conservation Service. A computer listing of the engineering properties of some 600 or so soils found in Minnesota, at depths to five feet, has been furnished to each District Materials Engineer. These data can then be compared against Mn/DOT's Proctor data to arrive at a compaction factor.

Another source is the analysis of data from an MHD study performed by W.W. Dreveskracht. A regression analysis of some 70 compaction versus depth of excavation calculations indicated the following compaction factors:

Depth (feet)	Compaction Factor (%)
1	122
2	116
5	108
10	102
15	98
20	96
25	94
30	92

where r^2, an indication of the scatter in the data, is 0.48 and Standard Error is 8.6. The values given are compaction factors only and do not include other factors, such as equipment shrinkage, which must be considered to arrive at an estimated shrinkage factor.

Due to the number of variables involved, especially natural soil properties and construction operations, as well as normal statistical variations, it must be realized that there will always be some uncertainty in the selection of a design shrinkage factor. That uncertainty can be reduced, however, by obtaining adequate data and performing careful analysis before a design shrinkage factor is selected. District construction, maintenance, and design personnel should be contacted as to their experiences with similar projects and soils.

Past Mn/DOT experience indicates that field conditions (e.g., depth, overburden and material type) impact the shrinkage or swell as shown in Table 5-2.1.

Table 5-2.1. Approximate shrinkage factors for depth, overburden and material type.

Material and Application	Shrinkage Factor (%)
Rock	
Sandstone	90 - 100
Limestone, granite, basalt, etc.	70 - 90
Shale	90 - 110
Soil	
Deep cuts and high fills	100 - 130
Normal cuts and fills	130 - 140
Ditch cuts and shallow fills	135 - 150
Shoulder grading	140 - 150
Light shoulder grading	150 - 165

Also see Table 5-2.13 for shrinkage factors for swamp backfill.

Depending on the type of material, the length of haul, and other factors, an additional adjustment in the required volume of backfill should be made to account for accidental loss or waste. Typical values may be on the order of about five percent, or less.

5-2.01.02 SOILS PREPARATION

Preparation of soils for foundation support and embankment placement typically involves reworking or enhancing the existing materials, promoting uniformity of the material's properties, and eliminating such problems as pockets of high moisture content or unstable or unsuitable subgrade soils.

1. Compaction. Compaction involves increasing the soil density by reducing the air voids (Figure 3-2.4) using artificial means, such as rolling or tamping, and is carried out to improve the engineering properties of a material utilized as fill. Engineering properties influenced by compaction include strength, compressibility, permeability, and volume change characteristics.

   For a given compactive effort, an optimum moisture content exists which results in the greatest degree of compaction. Therefore, in order to achieve the greatest density, it is advantageous to maintain the moisture content of the compacted soil close to its optimum during compaction. This will allow the greatest degree of compaction with the applied amount of compactive effort.

2. Lime Treatment. The engineering characteristics of soils may be altered by the application of lime. Lime is typically utilized as a drying agent with wet soils. In addition to its drying qualities, lime reacts with clay in the soil to produce decreased plasticity, shrinkage, and swell; improved workability; and increased strength and stability; but it may increase frost susceptibility of the soil, as well as pavement roughness and cracking.

   Construction with lime involves scarification of the existing subgrade, lime spreading and mixing, and compaction. Lime should be applied to the scarified surface for the full width across grade. Mixing with either rotary mixers or disc harrows should immediately follow to prevent loss due to wind. Lime should not be spread during windy conditions, due to excessive dusting. Thorough mixing is essential to distribute the lime uniformly, to proper depth and width, throughout the soil. After satisfactory moisture and soil conditions are achieved, the material can be compacted as specified.

   Additional information may be obtained from Section 5-2.01.04 and Mn/DOT's Grading and Base Manual, Section 5-692.521.

3. Geotextiles. Geotextiles can effectively be used for separation/stabilization in the bottom of subcuts, below culverts, etc., where soft soils make placement of the next lift difficult or density hard to achieve. Geotextiles should not be used as a substitute for portions of the required granular material. Mn/DOT's Specification 3733, Type V

is often referenced for this application. Geotextiles have also been used in embankment reinforcement to steepen slopes, and to improve bearing capacity and spread the loaded area in swamp crossings. These uses/designs require consultations with the Foundations Unit (Office of Materials Research & Engineering). Additional information may be obtained from Section 5-4.03.02 and Mn/DOT's Road Design Manual, Section 8-5.02.09.

5-2.01.03 SUBGRADE CORRECTION

Subcuts and culvert installations may require minor excavations and treatment of the subgrade that will support the roadway.

1. Subcuts. Subcuts are made to ensure uniformity of material and stability in the upper portion of the roadbed. Specifically, they are utilized to eliminate differential or pocketed high-moisture conditions, unstable materials, frost-heave potential, and non-uniform subgrade conditions. Typical subcut depths range from two to four feet with one foot minimum. Runouts are three to four feet. All subcuts should be backfilled with suitable grading materials.

   It is the responsibility of the District Soils Engineer to specify the depth of the subcuts. To determine the appropriate depth of subcut, first the depth of unsuitable subgrade materials, perhaps the calculated frost depth (Section 3-2.03.03), is determined.

   In areas of the roadbed that generate frost heaves, or may have the potential to produce a heave, the subcut depth should extend to below the frost line, with 20(H) to 1(V) tapers at each end. These transitional tapers are provided to prevent abrupt differential movement caused by different soil types (granular vs non-granular). The subcut should be backfilled with Select Granular Material (Mn/DOT's Specification 3149.2B). Perforated pipe should be placed in the bottom of the subcut according to the design standards (Fig. 5-4.2(e)) to assure that water does not collect as in a bath tub. If ground water levels are expected to intrude into the subcut area, special perforated pipe drain systems should be designed and installed. Assistance with such designs can be requested from the Geology Unit.

   If the subgrade beneath a roadway running under a bridge consists of plastic soils, the shadow cast by the bridge may produce a frost heave. The treatment for this condition is as described above with granular backfill and should extend at least 150 feet, plus tapers, on each side of the bridge.

   Refer to Mn/DOT's Road Design Manual, Figures 4-2.01A and B, for subcut sections. Similar treatments may be performed in sidehill cuts in non-granular soils.

   In general, when making subcuts or excavations, the following transitions should be provided:

- Where connecting new surfacing, adjacent to in-place pavements to be widened, cut vertically to the bottom of the in-place surfacing or to the bottom of the new surfacing design, whichever is deeper; then at a 1(H) to 1(V) slope to the bottom of the recommended subgrade excavation.

- Where connecting to in-place roadways at the termini of proposed new construction, cut vertically to the bottom of the in-place surfacing or to the bottom of the new surfacing design, whichever is deeper; then at a 20(H) to 1(V) taper to the bottom of the recommended subgrade excavation.

- Where matching into in-place crossroads, cut vertically to the bottom of the in-place surfacing or to the bottom of the new surfacing design, whichever is deeper; then at a 4(H) to 1(V) slope to the bottom of the recommended subgrade excavation.

- Provide 20(H) to 1(V) tapers when changing subcut depths. Tapers between plastic and granular soils should be constructed so that the granular soil overlays the plastic soil.

As a precautionary measure, traffic lanes to be used during construction must be delineated to keep vehicles a safe distance away from the adjacent excavation. The delineation should coincide with points established by projecting a 2(H) to 1(V) or flatter slope between the edge of the traffic surface and the near edge of the bottom of the excavation.

2. Culverts. Generally culvert excavations, particularly in rural settings, are open cut, being laid back without bracing or shoring. It is advisable to slope the excavation away from the culvert at slopes from 4(H) to 1(V) or flatter. However, the actual excavation slope should be determined during construction by the Contractor, depending on the in-situ soil properties and/or safety factors. The excavation slope lines indicated in the plans for pay quantity determinations are not necessarily adequate slopes from a safety standpoint. It is the contractor's responsibility to determine, from the soil conditions encountered at the site, the proper slope necessary for safe construction practices.

New culverts in plastic and non-uniform soils should be placed on a minimum two feet of aggregate bedding. The excavation should be backfilled with granular material. A geotextile (Mn/DOT's Specification 3733, Type V) is often used below the aggregate bedding for separation/stabilization if soft and/or unstable soils underlay the bedding. Again, tapers with 20(H) to 1(V) slopes between material changes should be constructed to help reduce the differential heave at the roadway surface caused by frost penetration into the soil around the pipe. For additions to existing culverts, the excavation should be backfilled with soils that substantially match the in-place soils, relative to textural classification, layering, moisture, and density, if there are no signs of excessive heave in the existing culvert. If excessive heave is present, a complete granular treatment with 20(H) to 1(V) tapers

between different material types is necessary. See Mn/DOT's Road Design Manual, Figures 8-6.02 A through E, for typical sections for treatment of culverts in plastic soils.

Lining of existing culverts may be an option to the installation of new culverts. Such linings consist of smaller diameter pipes inserted within the existing culvert, with the resulting annular space being grouted shut; or the use of similar proprietary treatments currently on the market. The obvious advantage of such treatment is that the need for excavation is greatly reduced, or eliminated entirely, which may be particularly advantageous in an urban setting. The main caution when considering such an alternative is being certain that adequate flow capacity remains, following the lining of the culvert.

Design and construction of excavations within urban settings requires the same considerations as those in rural areas, except where space limitations and underground utilities exist. When space limitations prohibit the use of open-cut trenches in the proximity of bridge approaches, culverts, and utilities, the design of shoring, bracing, and/or dewatering may have to be considered. The Foundations Unit or Geology Unit (Office of Materials Research & Engineering) should be advised and consulted in such situations.

5-2.01.04 PLACEMENT OF EMBANKMENT AND BACKFILL MATERIALS

Prior to proceeding with any construction operations, all necessary clearing and grubbing should be completed.

1. Uniformity, Layers, Transitions. Each layer of roadbed should be constructed of uniform material. If available soils are so varied that uniform soils cannot be placed practically, the material should be blended and mixed to produce uniformity and to obtain a uniform moisture content and density. In transition areas where granular embankments or backfill join plastic soil embankments or backfill, 20(H) to 1(V) tapers should be used between the change in materials to prevent abrupt soil differentials and placed so that the granular overlays the plastic soil.

   Embankment layers should be placed according to Mn/DOT's Specification 2105.3E. Embankment material should be placed in relatively uniform layers, approximately parallel to the profile grade. Layers in the upper three feet of the roadbed should not be more than eight inches in loose thickness, while those below the upper three feet should not be more than 12 inches in loose thickness, except as follows:

   a. For soft and unstable foundation soils that will not support construction equipment, the embankment may be constructed as a single layer, to the lowest elevation at which construction equipment can operate without causing the underlying soils to intrude into the upper eight inches of the embankment. In no

case should the top of this grade be less than three feet below the subgrade.

b. For placement of granular material (not more than 20 percent passing the No. 200 sieve) in the upper three feet of the embankment, lift thickness may be increased to not more than 12 inches, as long as proper compaction can be achieved.

c. Predominantly stone, broken concrete, or rock fragment embankment material of sizes that cannot be compacted can be placed in lifts, not exceeding a thickness of two feet, up to an elevation one foot below subgrade. Such material larger than three inches in the greatest dimension should not be placed within the upper six inches of the roadbed embankment. Materials should not exceed six inches in greatest dimension when placed within the upper three feet of embankment or backfill.

Each such lift should be leveled prior to placing the next layer thereon, using suitable equipment operated in a manner that will provide even distribution of the larger rock or broken concrete and fill the voids with the spalls and finer material, so as to form a compact mass.

d. Embankment materials adjacent to structures should be placed in lifts not more than eight inches in thickness for a distance of 50 feet on each side of pipes, 48 inches or less in diameter; and 100 feet on each side of other structures. The placement shall be for the full height, from the embankment foundation to the top elevation of the structure.

e. Layer construction is not required for channel fills, spoil banks, and berms that do not provide foundation support for structural features.

The upper two to four feet of the embankment/fill should be comprised of selected earth material or granular material. Granular materials, if used, should be placed in the uppermost portions of the embankment. Within 12 inches of the subgrade surface, granular materials should not be capped with non-granular materials.

At no time should embankment material be placed frozen or on soil that is frozen to a depth greater than four inches. No frozen material exceeding six inches in the greatest dimension should be permitted in the embankment. Frozen material less than six inches in the greatest dimension can only be placed outside of a 1(H) to 1(V) slope down and outward from the grading subgrade PI.

2. Compaction. Evaluation of embankment/fill material or base course compaction should comply with Mn/DOT's Specifications.

The Design Recommendation letter prepared by the District Soils Engineer should include recommendations for the method used to measure or evaluate the effectiveness of the compactive effort the contractor is using to place embankment/fill soils and aggregate base materials. Mn/DOT Spec. 2105, Excavation and Embankment, lists two methods to measure effectiveness of compactive effort, Specified Density and Quality Compaction (formerly termed Ordinary compaction); Mn/DOT Spec. 2211, Aggregate Base, lists three methods, Specified Density, Quality Compaction and Control Strip. Descriptions of these methods can be found in the Specifications and in the Grading and Base Manual.

Although these methods are equally effective in evaluating the results of a Contractor's efforts, there are circumstances when one method will be more practical than another.

For example, for a grading project, Specified Density MAY be more appropriate if there is a limited number of inspectors; if the planned embankment construction is over 50,000 cubic yards; or if the work involves a "large" number of culvert installations.

Quality Compaction MAY be more appropriate if the embankment construction is less than 50,000 cubic yards; if the inspectors available are experienced in all aspects of embankment construction and are numerous enough to constantly observe all of the contractor's operations that require Quality Compaction.

For the Aggregate Base part of a project Specified Density MAY be most appropriate if there is a limited number of inspectors available; if the project is considered "large" (over 3 miles long); and if the distance from the District headquarters is MORE than 60 miles or one hour travel time.

Quality Compaction is usually appropriate if the project is less than 3 miles long and the inspectors available are numerous enough to constantly observe all of the contractor's operations.

Control Strip MAY be appropriate if the number of available inspectors is limited; if the project is considered "large" (over 3 miles long); and if the distance from the District headquarters is LESS than 60 miles or one hour travel time.

For Aggregate Surfacing (Mn/DOT Specification 2118) there is no compaction requirement unless the recommended surface thickness exceeds TWO (2) inches, in which case the aggregate is compacted and evaluated in accordance with the requirements of Mn/DOT Specification 2211.3, Quality Compaction.

Where the Specified Density Method is used, it is the project field inspector's responsibility to sample and test the materials to be used as fill; to determine their maximum dry densities and optimum moisture contents; and to perform density and moisture tests on the compacted embankment, to assure that specified density requirements have been attained. Materials placed within the upper three feet of the embankment, and those adjacent to structures, should be compacted to a dry density of not less than 100 percent of their maximum dry density. Materials placed below the upper three feet of the embankment, and not adjacent to structures, should be compacted to a dry density not less than 95 percent of their maximum dry density.

Quality compaction (visual inspection) may be specified for materials placed outside a 1-1/2(H) to 1(V) slope down and outward from the grading shoulder P.I. on fills over 30 feet in height, or outside of a 1(H) to 1(V) slope down and outward from the grading shoulder P.I. on fills of 30 feet or less in height. The embankment/fill should be compacted until there is no further evidence of consolidation.

Mechanical compaction is not required on portions of the embankment that are constructed with predominantly stone or rock fragments or placed in conjunction with topsoil covering or roadside grading. Density control should not apply to waste materials or non-rock material (topsoil, etc.) utilized for incidental drainage or landscaping outside the roadbed embankment. However, it is the responsibility of the project field inspector to approve that the placed material is a compact mass that is acceptably consolidated.

3. Moisture Control. Compaction moisture control should comply with Mn/DOT's Specification 2105.3F. For materials placed with specified density control, the moisture content of the embankment material should be not more than 115 percent of its optimum moisture content when 95 percent of the maximum density is specified, and should be within the range of 65 to 102 percent of its optimum moisture content when 100 percent of the maximum density is specified.

   Special moisture restrictions should be employed for problem soils. For expansive and lacustrine soils, the compaction moisture content should be within the range of 90 to 115 percent of the optimum moisture content for material placed below the upper three feet of the fill, and within the range of 90 to 102 percent within the upper three feet. The compaction moisture content for red drift soils should be within the range of 65 to 95 percent of its optimum moisture content.

4. Use of Lime. The decision on the use, type, and amount of lime is the responsibility of the District Soils or Materials Engineer. The amount of lime used depends upon the moisture content of the soil being dried, the plasticity, and/or the desired engineering properties to be attained.

   Quicklime and hydrated lime products are commonly used. Quicklime, or calcium oxide, is highly reactive, compared to hydrated lime.

Hydrated lime is quicklime that has had part of its water demand satisfied by adding water in the slaking process. For drying of soils, in most cases, less than one percent hydrated lime is sufficient, but in no case should more than two percent be used. Quicklime may be substituted for hydrated lime on the basis of one percent quicklime being equal to 1.3 percent hydrated lime. Additional information may be obtained from Mn/DOT's Grading and Base Manual, Section 5-692.521.

5. Controlled Rate/Surcharge. Compressible silts, clays, and organic deposits may be improved by surcharging or pre-loading. The pre-load fill is applied in controlled lifts to the area to be treated and allowed to remain in place while foundation soils consolidate sufficiently to increase their strength or reduce their compressibility. After sufficient time for the required compression has occurred, either the pre-load is removed or additional fill may be placed. Pre-loading has proven to be a relatively inexpensive method of improving deep compressible deposits.

   Monitoring of embankment settlements and movements is done through the installation of settlement plates (Figure 3-2.10), hub lines, and control points. Installation and location of settlement plates on, and hub lines and control points outside of, the embankment should be coordinated between the District Soils Engineer and the Foundations Unit (Office of Materials Research & Engineering).

6. Test Rolling. Evaluation of the load carrying capacity of a prepared subgrade with heavy rollers prior to the placement of the base course is called test rolling. The Department normally employs test rolling only on main lines of projects at least one mile in length. Areas which pump or rut excessively should be excavated and recompacted to provide adequate bearing material for the pavement. Test rolling should comply with Mn/DOT's Specification 2111.

5-2.01.05 EROSION CONTROL AND TURF ESTABLISHMENT

During construction, temporary erosion controls should be used in conjunction with permanent measures, with all forms of erosion control being placed as soon as permissible. Temporary measures are of particular concern with steep rolling topography, erosive subsoils (i.e., loamy sands, silt loams, and silty clay loams), and where most of the drainage from the construction limits may enter into streams, lakes, etc. Temporary measures consist of bale ditch checks, earth diversions, bale diversions, temporary drains, and sediment traps/basins. Additional details of these controls can be obtained in Mn/DOT's Road Design Manual, Section 8-5.0.

Erosion is controlled, on a permanent basis, through the placement of sod or the establishment of turf. In most situations erosion control sod should be provided on typical highway projects, even though a portion of the project may be in an urban setting. Erosion control sod is of good quality and usually blends in quite well with established lawns. Lawn and boulevard sod should be restricted to rest area lawns and building site lawns.

The following Table 5-2.2 summarizes the available types of sod and their usage.

Table 5-2.2. Sod types and usage.

Sod Type	Predicted Use (%)	Quality	Usage
Lawn and boulevard	5 - 15	Premium	High maintained areas, such as rest areas and lawns.
Erosion control	75 - 95	Average to high	Areas with low maintenance, such as ditch bottoms, bridge end slopes, lawns, and behind curbs.
Pasture	0 - 5	Low to average, harvested from local areas where commercial sod may not be available	Very harsh roadsides and/or very sandy soils.

When turf is to be established, it should be placed as soon as construction permits. This reduces the length of time areas are left unprotected and the area exposed is reduced. It is the responsibility of the District Soils Engineer to recommend types of turf establishment in the project Design Recommendation Report. The components of successful turf establishment are discussed below.

1. Topsoil. The most critical element for successful turf establishment is the amount and quality of topsoil at the site. Therefore, it is extremely important to salvage as much of the in-place topsoil as possible. All available topsoil should be salvaged and stockpiled during construction and then re-spread over the disturbed areas.

   a. General Roadsides. The Department's goal for topsoiling roadsides is to provide six inches of topsoil over all disturbed areas. If there is a shortage of suitable topsoil, four inches may be provided. In no circumstances should less than three inches be recommended. If there is an insufficient quantity of salvageable topsoil to provide even three inches, then a topsoil borrow item should be recommended.

   b. Rest Areas/Building Sites. For lawns on all rest areas and building sites, a minimum of six inches of topsoil should be provided. The Department's goal is to provide six inches of topsoil over the entire site. However, three to four inches can be recommended for the outlying areas (beams, ramps, drain fields, and exercise areas) if a shortage exists.

   c. Excess Topsoil. If there is an excess of topsoil on a proposed project, then the depth of replaced topsoil should be increased as much as possible. No less than six inches should be placed on

areas to be seeded. Inslopes may be built up with topsoil, and additional topsoil may be stockpiled for future use by maintenance.

d. Use of Peat as Topsoil. Because of shrinkage and stability problems, peat is generally not recommended for use as a topsoil medium. However, the District Soils Engineer should make a detailed review of the type of organic soils on the project and deal with each individual project or portion of a project. For instance, organic soils classified as muck, organic silt loams, and even peat loams may be used as topsoil. In some cases, sand could be mixed with peat loams to make a suitable growing medium. Semi-fibrous to fibrous peats should not be used to topsoil slopes. Fibrous peats have high shrinkage and low stability characteristics.

2. Lime. Lime aids in successful turf establishment on acidic type soils, which are generally found in the central, north central, and northeastern portions of the state. Usually, these soils are forest type soils and/or sands with a high degree of leachability. The degree of acidity is measured by the difference from a pH less than 7.0. For any particular project, the determination of lime requirements is made from soil test results and the information in Tables 5-2.3, 5-2.4, and 5-2.5, which follow. Areas 1 and 2 are delineated in Figure 5-2.1. The SMP buffer index test is conducted on mineral soils with less than 6.0 pH.

Table 5-2.3. Lime recommended for mineral soils when soil/water test is conducted.

Soil-water pH	Application (tons/acre) Area 1	Area 2
6.4 and higher	0.0	0.0
6.3	2.0	0.0
6.2	2.0	0.0
6.1	2.0	0.0
6.0	3.0	2.0

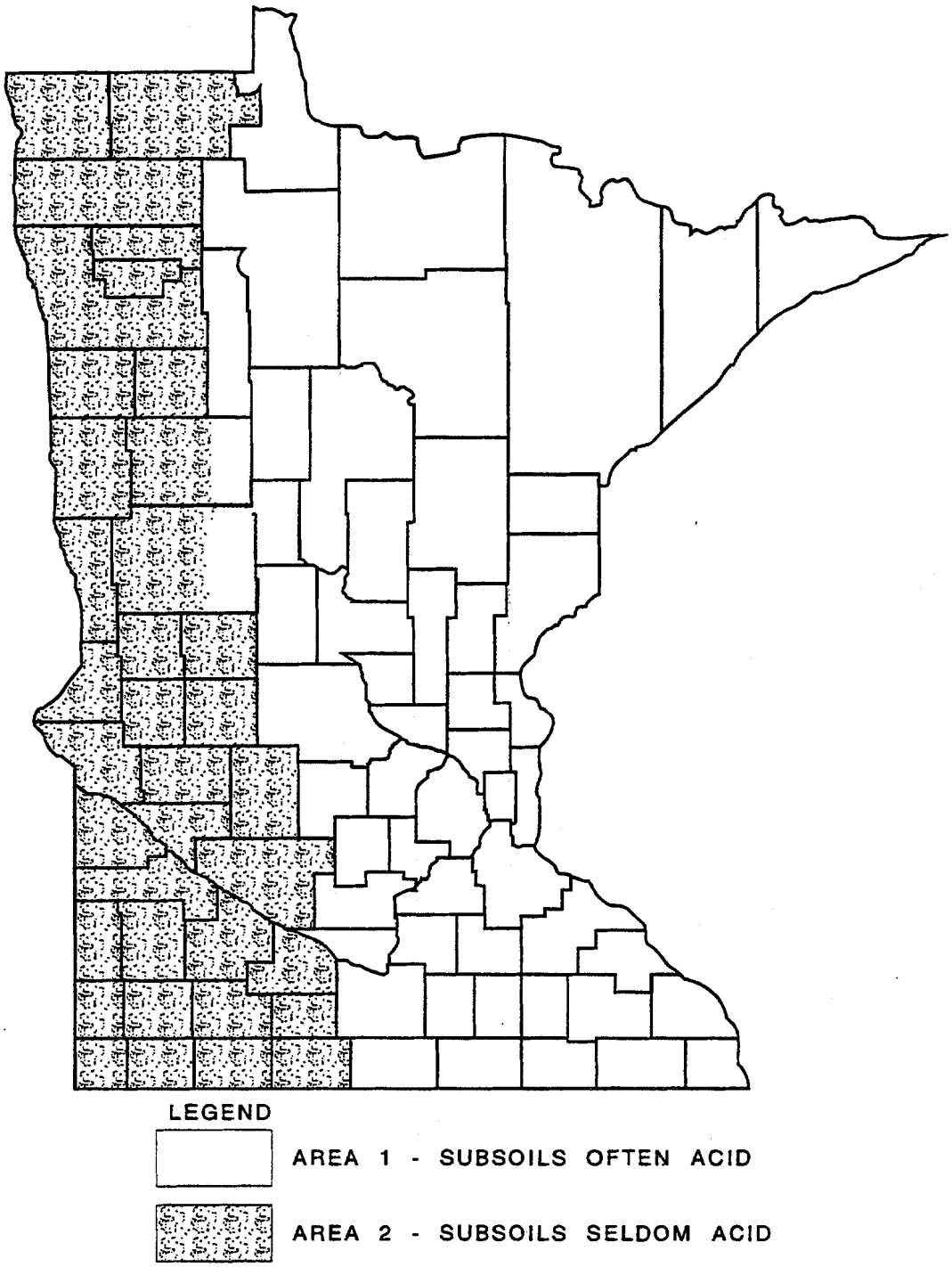
Table 5-2.4. Lime recommended for mineral soils when SMP test is conducted.

SMP Buffer Index	Application (tons/acre) Area 1	Area 2
6.8	3.0	2.0
6.7	3.0	2.0
6.6	3.0	2.0
6.5	3.5	2.0
6.4	3.5	2.5
6.3	3.5	2.5
6.2	4.0	3.0
6.1	4.0	3.0
6.0	4.0	3.5
5.9	4.0	3.5
5.8	5.0	4.0
5.7	5.0	4.0
5.6	5.0	4.5

Table 5-2.5. Lime recommended for organic soils.

Soil-water pH	Application (tons/acre)
5.5 and higher	0.0
5.4	2.0
5.3	2.0
5.2	2.0
5.1	2.0
5.0	2.0
4.9	3.0
4.8	3.0
4.7	4.0
4.6	4.0
4.5 or less	5.0

Figure 5-2.1. Map used to determine lime needs



3. Fertilizer

The determination of fertilizer requirements is made from soil test results. The amount of fertilizer to apply is dependent on the type of turf established, or seed mixture used, and the fertility level of the in-place

soils. In order to make fertilizer determinations, Tables 5-2.6, 5-2.7, and 5-2.8 should be utilized with Figures 5-2.2 and 5-2.3.

Table 5-2.6. Nitrogen required for various seed mixtures.

	Nitrogen Required (lb./acre)	
Organic Content of Soil (%)	Mixtures 400 - 700	Mixtures 100 - 300 and 800 - 1,000
0 - 5	70 - 80	30
6 - 10	50 - 60	20
> 10	40	20

Figure 5-2.2. Determination of phosphorous.

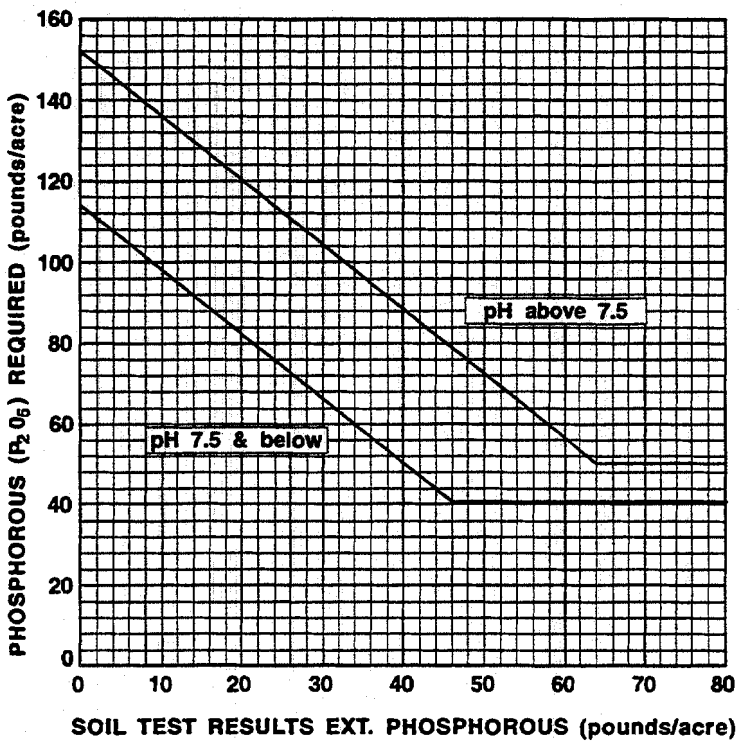


Table 5-2.8. "Shotgun" fertilizer analysis and rates.

Analysis	Rate (lb./acre)	Seed Mixtures	Soil Conditions
12-12-12	500	500 - 700	Average to favorable, medium, organic, dark colored, loams and heavier soils, average depths.
12-12-12	700	500 - 700	Lighter soils, forest soils, thin reddish soils, lighter colored, tan to gray.
10-20-20	450	400 - 700	Favorable soils, silty soils, darker colored salt contaminated soils.
8-32-16	400	800, 900	Alkaline soils, prairie-type soils found in southwestern Minnesota and through the Red River valley.
12-24-12	400	500, 700	Alkaline soils, prairie-type soils found in southwestern Minnesota and through the Red River valley when grass mixture is desired.
6-24-24	500	800, 900,	Heavier soils, tan to dark colored, or when B horizon soils are to be seeded.
20-10-10	400	650, 600,	Favorable soils when lawn-mixtures are seeded.
6-24-24	200	100 - 300	All soil types when native grasses are seeded.

Following is an example of proper fertilizer selection:

Example. Determine fertilizer analysis and rate for a soil with an organic content of 3%, extractable phosphorous of 14 lb./acre, exchangeable potassium of 110 lb./acre, and a pH of 6.8. Seed Mixture 500 is to be used.

From Table 5-2.6, nitrogen required = 70 - 80 lb./acre.
From Figure 5-2.2, phosphorous required = 88 lb./acre.
From Figure 5-2.3, potassium required = 110 lb./acre.

Table 5-2.7. Analysis of available fertilizers.

N P_2O_5 K_2O Ratio	Analysis	N P_2O_5 K_2O Ratio	Analysis
1:2:1	12-24-12	0:1:2	0-20-40
1:2:2	10-20-20	2:2:1	20-20-10
			16-16- 8
2:6:3	8-24-12		
		1:4:1	8-32- 8
1:4:4	6-24-24		
	5-20-20	3:2:4	15-10-20
1:1:1	10-10-10	2:2:3	14-14-21
	12-12-12		12-12-18
	14-14-14		
	15-15-15	3:3:4	15-15-20
	16-16-16		12-12-16
	17-17-17		
		2:3:3	12-18-18
1:4:2	8-32-16		14-21-21
	7-28-14		
	6-24-12	1:1:2	10-10-20
1:2:4	6-12-24	2:1:5	10- 5-25
1:3:9	4-12-36	2:1:2	24-12-24
2:1:1	20-10-10	9:23:0	18-46- 0
	16- 8- 8		
		1:1:0	25-25- 0
3:1:1	15- 5- 5		
	24- 8- 8	1:3:3	8-24-24
		- - -	9-23-30

Table 5-2.8. "Shotgun" fertilizer analysis and rates.

Analysis	Rate (lb./acre)	Seed Mixtures	Soil Conditions
12-12-12	500	500 - 700	Average to favorable, medium, organic, dark colored, loams and heavier soils, average depths.
12-12-12	700	500 - 700	Lighter soils, forest soils, thin reddish soils, lighter colored, tan to gray.
10-20-20	450	400 - 700	Favorable soils, silty soils, darker colored salt contaminated soils.
8-32-16	400	800, 900	Alkaline soils, prairie-type soils found in southwestern Minnesota and through the Red River valley.
12-24-12	400	500, 700	Alkaline soils, prairie-type soils found in southwestern Minnesota and through the Red River valley when grass mixture is desired.
6-24-24	500	800, 900,	Heavier soils, tan to dark colored, or when B horizon soils are to be seeded.
20-10-10	400	650, 600,	Favorable soils when lawn-mixtures are seeded.
6-24-24	200	100 - 300	All soil types when native grasses are seeded.

Following is an example of proper fertilizer selection:

Example. Determine fertilizer analysis and rate for a soil with an organic content of 3%, extractable phosphorous of 14 lb./acre, exchangeable potassium of 110 lb./acre, and a pH of 6.8. Seed Mixture 500 is to be used.

From Table 5-2.6, nitrogen required = 70 - 80 lb./acre.
From Figure 5-2.2, phosphorous required = 88 lb./acre.
From Figure 5-2.3, potassium required = 110 lb./acre.

The above nutrients are in an approximate 2:2:3 ratio. Thus, from Table 5-2.7, select 12-12-18 analysis. Next, determine an application rate by dividing the required nutrients by the percent active ingredient in the fertilizer, i.e., for nitrogen, 80 ÷ 0.12 = 666; for phosphorous, 88 ÷ 0.12 = 733; and, for potassium, 110 ÷ 0.18 = 611. Thus, an application rate of 650 lb./acre appears appropriate.

Use 12-12-18 analysis at an application rate of 650 lb./acre.

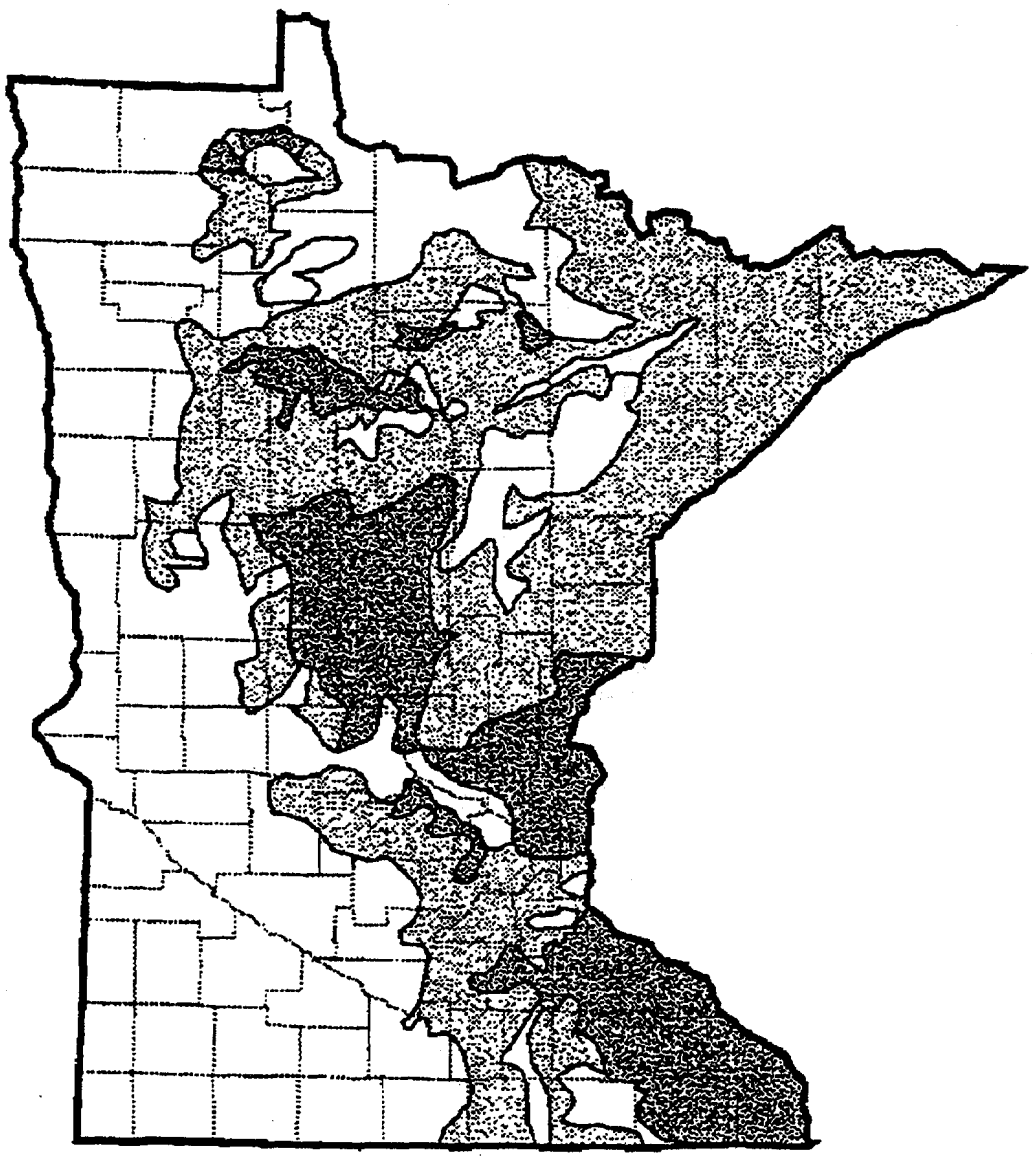
When no soil samples have been taken for a particular project, and thus no laboratory test data are available, three methods may be used to determine the fertilizer needs requirement. These methods are as follows:

- Use the fertilizer recommendation from an adjacent project that has similar soil conditions.

- Obtain the approximate phosphorous and the approximate potassium concentrations of the in-place soils from Figures 5-2.4 and 5-2.5 and use these values as if they were test data. Nitrogen requirements can be obtained from Table 5-2.6 by assuming an organic content of the in-place soils (usually five percent or less). Then proceed with determining the fertilizer requirement as shown in the above example.

- Use the "shotgun" fertilizer analysis and rates listed in Table 5-2.8.

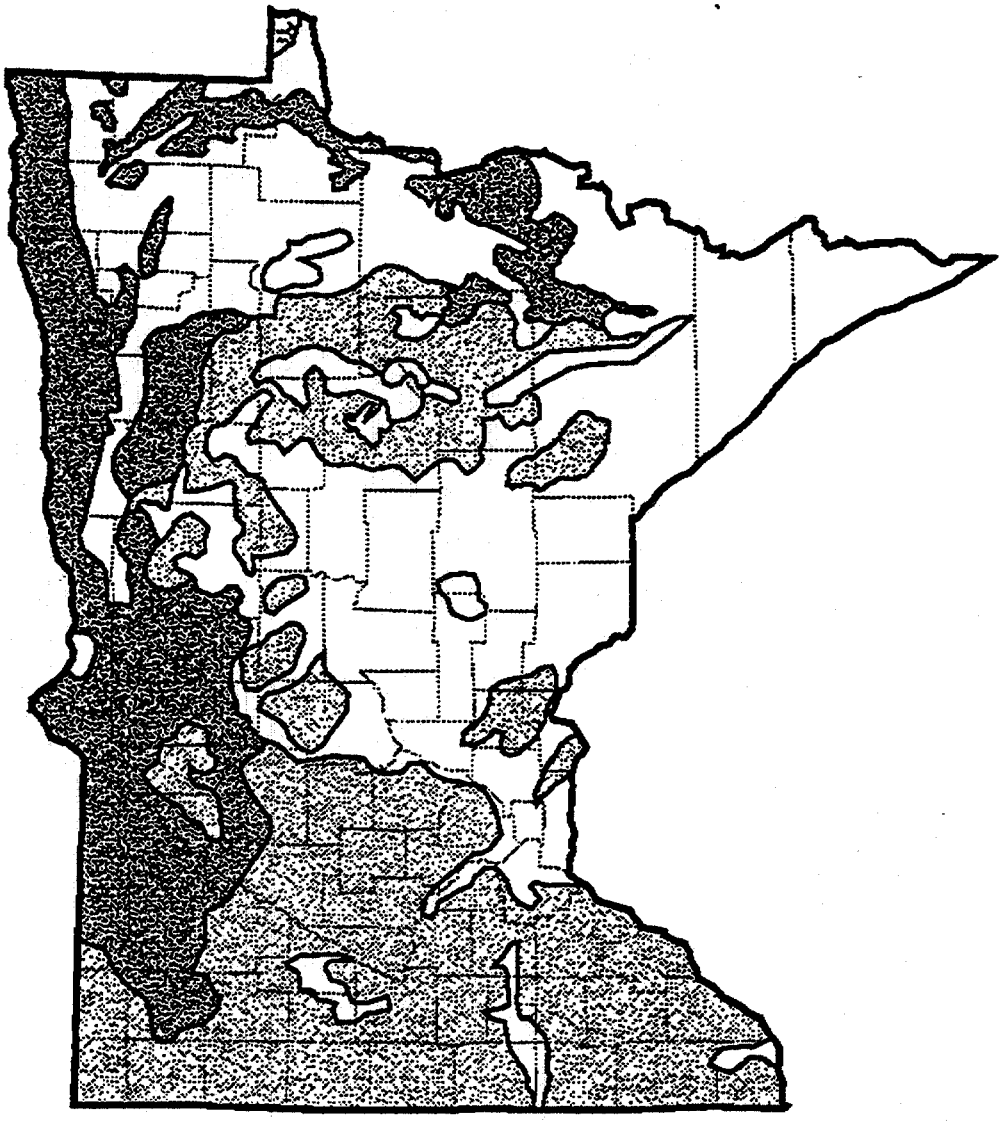
Figure 5-2.4 Relative soil phosphorous content.



LEGEND

- 10 POUNDS/ACRE
- 15 POUNDS/ACRE
- 20 POUNDS/ACRE

Figure 5-2.5. Relative soil potassium content.



LEGEND

- 100 pounds/acre
- 150 pounds/acre
- 250 pounds/acre

An example using the above two Figures 5-2.4 and 5-2.5 to determine the fertilizer requirement follows:

Example. Determine the fertilizer requirement and rate for a turn lane project near Willmar. Seed Mixture 800 is to be used.

First, the in-place soil fertility level is determined. Assume a low organic matter content of in-place soil (0 - 5%). Then, from Figure 5-2.4, the soil phosphorous level is 10 lb./acre and from Figure 5-2.5, the soil potassium level is 150 lb./acre.

Second, the nutrients needed are determined. From Table 5-2.6, the nitrogen required is 30 lb./acre; from Figure 5-2.2, the phosphorous required is 96 lb./acre; and from Figure 5-2.3, the potassium required is 120 lb./acre.

Third, the fertilizer analysis and rate required is determined. From Table 5-2.7, select 9-23-30 analysis. Next, determine an application rate by dividing the required nutrients by the percent active ingredient in the fertilizer, i.e., for nitrogen, 30 ÷ 0.09 = 333; for phosphorous, 96 ÷ 0.23 = 417; and for potassium, 120 ÷ 0.30 = 400. Thus, an application rate of 400 lb./acre appears appropriate.

Use a 9-23-30 analysis at an application rate of 400 lb./acre.

4. Mulch. Mulch performs a very important function in providing a proper environment for seed germination and controlling erosion while the seedling plants establish themselves. Several different mulching techniques are available for various types of projects. Therefore, it is important that the best mulching technique be selected to match project conditions.

   a. General. Available mulch types and their general usage are listed in Table 5-2.9.

Table 5-2.9. Mulch types and usage.

Type*	Rate	Use	Remarks
1	2 tons/acre	General - All soils types, flat to moderate slopes (3:1 or flatter)	Most effective and economical type. Include disc anchoring.
2	0.18 gal/yd^2	Drier sandy soils without much organic matter. Limit to small scattered areas such as roadway widenings (less than 10 ft.).	Does not work very well on soils high in organic matter. Water at a rate of 1,000 gal./acre should also be provided to pre-moisten the ground surface.
3	Not available at this time		
4	1-1/2 tons/acre of Type 1 plus 500 lb./acre of Type 5	All soils types. General use on slopes steeper than 3:1	Very economical mulching technique on slopes.
5	1,500 lb./acre	Inaccessible areas such as overburden and weathered rock slopes.	Can also be used on small flat areas near traffic and buildings.
6	For use around landscape plantings only		

*Defined in Mn/DOT's Specifications

b. Shoulder Mulch Tacking. Shoulder mulch tacking is an additional technique for preventing erosion and establishing vegetation in the critical area immediately adjacent to paved shoulders. Basically, shoulder mulch tacking consists of applying an approximately three-foot wide band of asphalt emulsion with a distributor spray bar over the top of Type 1 mulch. Projects where shoulder mulch tacking would be very beneficial are those with heavier traffic volumes and/or those that are located in open wind-swept locations.

Shoulder mulch tacking is described in Mn/DOT's Specification 2575.3F10. It should be applied at a rate of two tons per acre of Type 1, oversprayed with 0.10 gallons per square yard of Type 2. It can be used on all soil types and is very effective in holding

straw/seed in place next to bituminous shoulders. Also, provide for disc anchoring the Type 1 mulch.

c. Nettings and Blankets. Plastic nettings, consisting of Type 1 mulch with netting rolled out over the top of the mulch, or wood fiber blankets should be recommended for establishing vegetation and controlling erosion on slopes steeper than 3(H) to 1(V) where an erosion problem is anticipated. Nettings, which generally work better on cut slopes than they do on fill slopes because of better soil density. They should be used only where runoff does not pour over the top of the slope; whereas wood fiber blankets can be used in such situations. Wood fiber blankets are generally more effective at controlling erosion than the straw/netting combination because of better overall material density and better fastening capability.

Generally, wood fiber blankets are considered to be the best mulch available. They can also be used in lieu of sod around culvert apron ends.

Plastic nettings and wood fiber blankets are covered by Mn/DOT's Specifications 3883 and 3885, respectively.

5. Seed. Perhaps the most difficult recommendation for the District Soils Engineer, relative to turf establishment, is the seed mixture to use on a specific project. However, standard mixtures have been formulated for just about all situations encountered throughout the state. Some of the situations and the potential for successful performance of the various seed mixtures are listed in Table 5-2.10.

Table 5-2.10. Performance of various seed mixtures.

	Performance			
Factor	Very Poor	Poor	Good	Very Good
Adaptability to wet sandy soils	800, 1,000	100, 150, 200, 300, 650	400, 500, 600, 700	900
Adaptability to dry sandy soils	800, 1,000	400, 500, 600, 700, 650	100, 150, 900	200, 300
Adaptability to plastic - soils, clays and silts	650, 900	400, 500,	100, 150 600, 700, 800	200, 300, 1,000
Adaptability to peat disposal	100, 150 200, 300 400, 650, 800, 1,000	600	700, 900	500
Tolerance to alkaline soils, pH > 7.5	-	-	500, 600, 650, 700, 800, 900, 1,000	100, 150, 200, 300, 400
Tolerance to acidic soils	800, 1,000	400, 600, 650, 900	100, 500, 700	150, 200, 300
Tolerance to salt	600, 650, 800, 900, 1,000	500, 700	300	400
Adaptability to mowing	100, 150, 200, 1,000	700, 800, 900	300, 500	400, 600, 650

a. General Roadside Seed Mixtures. General roadside seed mixtures are identified as Mixtures 500, 600, 700, 800, 900, and 1,000. These mixtures have a large range of adaptability and can be used statewide. However, each mixture is developed for a specific use and/or project soil condition. Historically, Mixture 500 has been used statewide and for all types of situations

and/or soils. This does not mean, however, that Mixture 500 should be recommended as a "boiler plate" selection in the Design Recommendation Report. One of the other mixtures may be more adaptable to the site and/or soils conditions, as indicated in Table 5-2.11.

Table 5-2.11. Seed mixtures and usage.

Mixture	Use	Soil Types
500	General roadside use statewide. May be used on back slopes, inslopes, ditches, and waste areas. Can be mowed.	All soil types. Best on loams, tolerates most pH ranges and fertility levels.
600	Roadsides in urban areas that are mowed. Can also be used on rural roadside shoulders.	Good topsoils, medium organic and high fertility levels.
700	Rural mixture most widely used by counties. Area of use is similar to Mixture 500.	All soil types. Best on loams, most pH ranges.
800	Rural mixture used on heavy soils or for seeding plastic subsoils without topsoil. Mainly used in agricultural areas of the state.	Clays, clay loams, silts, silt loams, plastic sandy loams.
900	Rural mixture used on sands, silts, and poorly drained soils. Does well on red silty soils and gravelly areas.	Silts, silt loams, plastic sandy loams, loamy sands.
1000	Where it is desired to establish crown vetch.	Clays, clay loams, silts, silt loams, loams.

b. Lawn Seed Mixture. Seed Mixture 650 is specially formulated for establishing a high-quality lawn. The mixture contains 75 percent bluegrasses. Some of the bluegrass components are low-maintenance types, while the remaining bluegrass components are dense, dark green colored, and luxuriant. Seed Mixture 650 should be used at building sites, at rest areas, and at other places a quality lawn is desired, but are not sodded. Whenever Mixture 650 is specified, at least six inches of quality topsoil should be placed.

c. Salt-tolerant Seed Mixture. Seed Mixture 400 is specially formulated for highway roadsides or boulevards that are subjected to heavy salt use. Species contained in the mixture are low growing (up to about eight inches high) and sod forming. The alkali grass contained in the mixture requires a salty environment in order to survive. Therefore, Mixture 400 should be recommended for shoulders and medians in urban areas of heavy salt use.

d. Mixtures for Temporary Cover. Seed Mixtures 1100 (Oats) and 1150 (Rye) are available to provide temporary cover. Oats should be used in the spring and summer; rye should be used in the fall. Both oats and rye are annuals and provide vegetative cover for only one year. For projects that need a temporary cover crop lasting more than one year, Mixture 900 should be specified. Mixture 900 contains a considerable amount of legumes in it and, as a result, establishes quite easily, even without suitable topsoil.

e. Native Grass Mixtures. The use of native grasses is encouraged whenever possible, especially in rural areas of the state. The native grasses have proven to be extremely drought tolerant and hardy, and they do very well in poor soils. They also provide excellent wildlife habitat for birds that nest and forage on roadsides.

Mixtures 100, 150, 200, and 300 are native grass seed mixtures designed for specific areas of the state (Figures 5-2.6, 5-2.7, 5-2.8, and 5-2.9), or growing conditions described in Table 5-2.12, which follows.

Table 5-2.12. Native grass mixtures and usage.

Mixture	Use	Soil Types
100	Specifically in western Minnesota where conditions are dry and soils are relatively poor. Suggested for use in ditch bottoms and back slopes.	Alkaline soils of western Minnesota. Clays, silty clays, and silts.
150	General mix that can be used nearly anywhere in the state. Suggested for use in ditch bottoms and back slopes.	All soil types, except peat.
200	Statewide, where soils are very sandy and well drained, especially for the Anoka Sand Plain Region, etc. Suggested for use in ditch bottoms and back slopes.	Sandy soils, sandy loams, loamy sands, and sands.
300	Statewide, composed of short grasses and intended for use on the inslopes of roadsides and in medians.	All soil types, except peat.

Figure 5-2.6. Suggested planting mixtures by region.

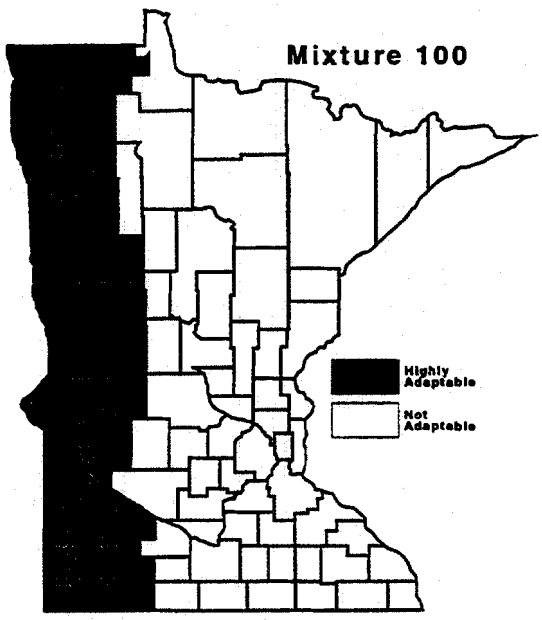


Figure 5-2.6

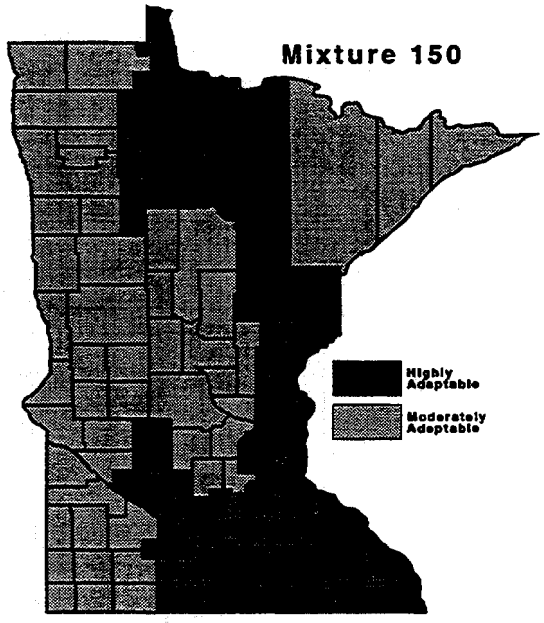


Figure 5-2.7

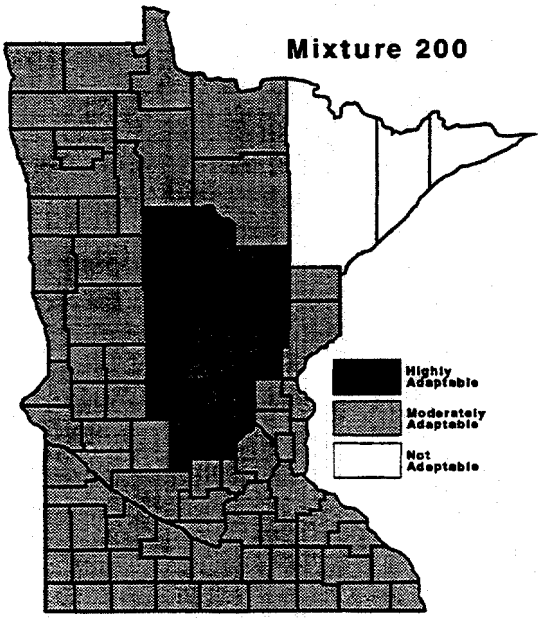


Figure 5-2.8

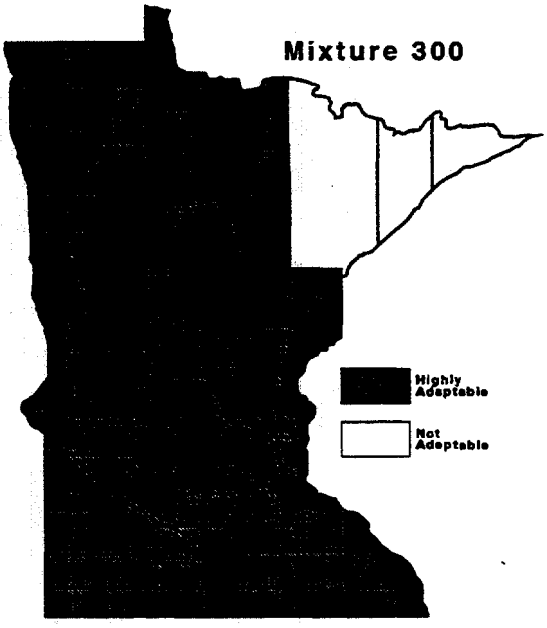


Figure 5-2.9

f. Wildflower Seed Mixtures. Wildflower seed mixtures are for specific situations. Basically, wildflowers have been seeded on rural projects and rest area sites. They can be used throughout the state. The goal of the Department is to match the species to be seeded with the species that grow naturally in the local project area. Therefore, most wildflower mixtures should be formulated as Special Seed Mixtures by the Agricultural Engineer.

Wildflower seed Mixture 1200 is a mixture that can be used for the southern half of Minnesota (south of St. Cloud).

5-2.02 SPECIAL TREATMENTS AND CONSIDERATIONS

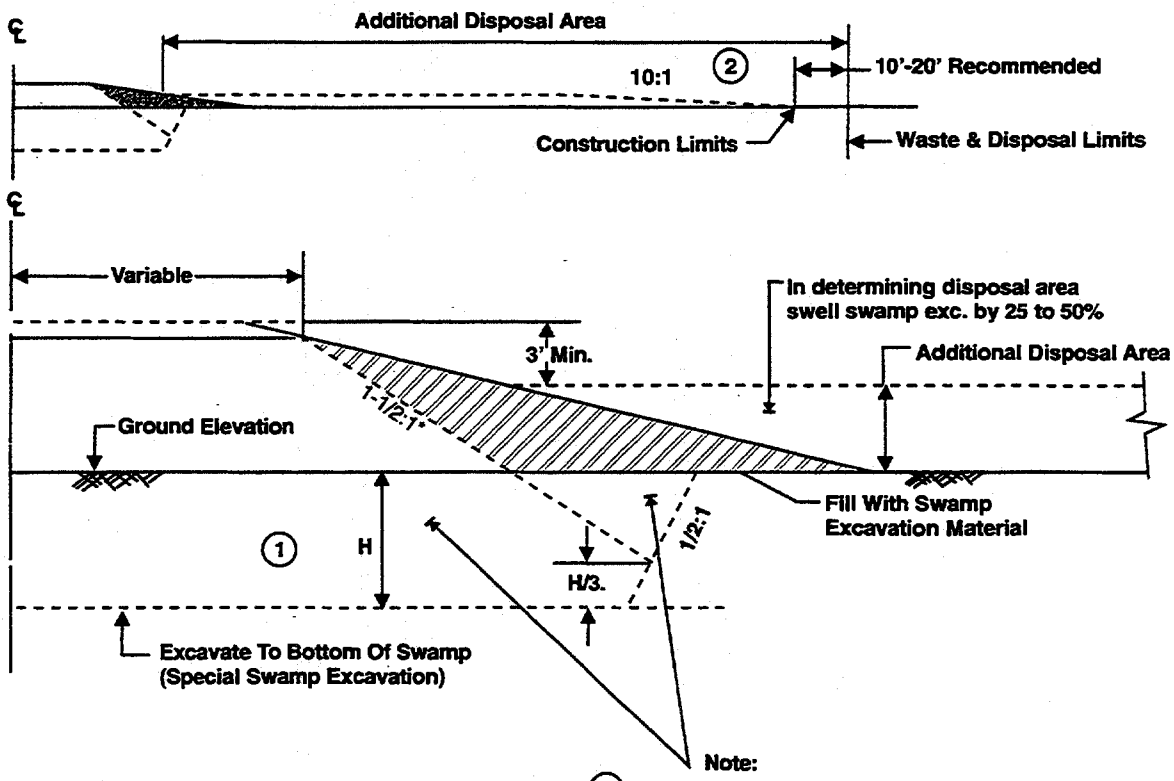
Special treatments and considerations are required for non-typical foundation materials and conditions. These are discussed in the following sections.

5-2.02.01 SWAMPS

It is the responsibility of the District Soils Engineer to determine the depth of required swamp excavation, location and character of required backfill material, and associated shrinkage factors. Figure 5-2.10 shows typical swamp sections. The Foundations Unit (Office of Materials Research & Engineering) may also be contacted for recommendations.

1. Excavation and Backfill. In this method, unstable swamp materials are excavated and replaced with suitable backfill. For fills of intermediate height (five to 20 feet), the typical section shown in Figure 5-2.10 should be used as a guide. In general, swamp material should be excavated outward (from the road centerline) and down from the point of intersection of the proposed side slopes and roadway surface (P.I.) to a depth of about two thirds the swamp thickness. From that point, the swamp material should be excavated on a slope of 1/2(H) to 1(V) outward and up to the existing grade and in and down to the bottom of the swamp. For shallow fills of less than about five feet, a floated embankment may be used; or the excavation may be made on a steeper slope of 1/2(H) to 1(V), but must extend outward and all of the way down to the bottom of the swamp. For high fills of more than about 20 feet in height, where more excavation is required to maintain stability, the typical section (1-1/2(H) to 1(V) slope), or even flatter, should be maintained and extend outward and all the way down to the swamp bottom.

Figure 5-2.10. Typical swamp sections.



Note:

1. Depths of Swamp Excavation, Blackfill Material, And Shrinkage Factor (Including Subsidence) To Be Determined By Soils Engineer.
2. For Design Purposes, Use 10:1 Slope Unless Otherwise Recommended By Soils Engineer.

*Normal swamp excavation. For special cases use table and consult with MR&E Foundations Unit.

	Low Fill ≤5'	High Fill >20'
SHALLOW SWAMP ≤5'	Float fill or excavate down on 1/2:1 from PI to bottom	Excavate full width or at least on 1-1/2:1 from PI to bottom
DEEP SWAMP >20'	Float fill or excavate down on 1/2:1 from PI to bottom	Excavate down on 1-1/2:1 or flatter from PI to bottom of swamp

Excavated swamp material may be disposed of along the completed toe of the roadway embankment. In general, disposal areas for swamp material should have 10(H) to 1(V) or flatter slopes, unless otherwise specified by the Foundations Unit.

Backfill material should consist of either granular or plastic borrow type materials. If the excavation is performed under water, the excavation should be backfilled with granular material to a level at least two feet higher than the local water level. If the excavation is performed in the dry (due to natural causes or the water having been removed from the excavation by sumps and pumping), the excavation may be backfilled with either plastic or granular soils. However, granular soils are generally the preferred swamp backfill material.

The placement of swamp backfill material should, in general, be in accordance with the recommendations in Section 5-2.01.04.

Table 5-2.13 should be used as a guide in estimating the shrinkage factor for swamp backfill.

Table 5-2.13. Approximate shrinkage factors for swamp backfill.

Placement	Shrinkage Factor (%)
Removing small amount of topsoil	130
5 ft. below natural ground	135
10 ft. below natural ground	140
15 to 20 ft. below natural ground, irregular bottom	145
About 30 ft. below natural ground, very irregular bottom	150

Caution should be exercised if sumps and surface pumping are used, as such methods may decrease overall stability and result in flowing material or slides and damage to adjacent structures. In some swamp excavations it may be necessary to maintain the natural water level, even by pumping in water, to keep the sides

of the excavation stable or to minimize possible settlement of adjacent structures.

The proximity of adjacent structures to the excavation should be a concern to the District Soils Engineer. The damage that would result from flowing material or slides should be addressed. Installation and monitoring of settlement plates, hub lines, or other instrumentation may be necessary, and should be coordinated with the Foundations Unit.

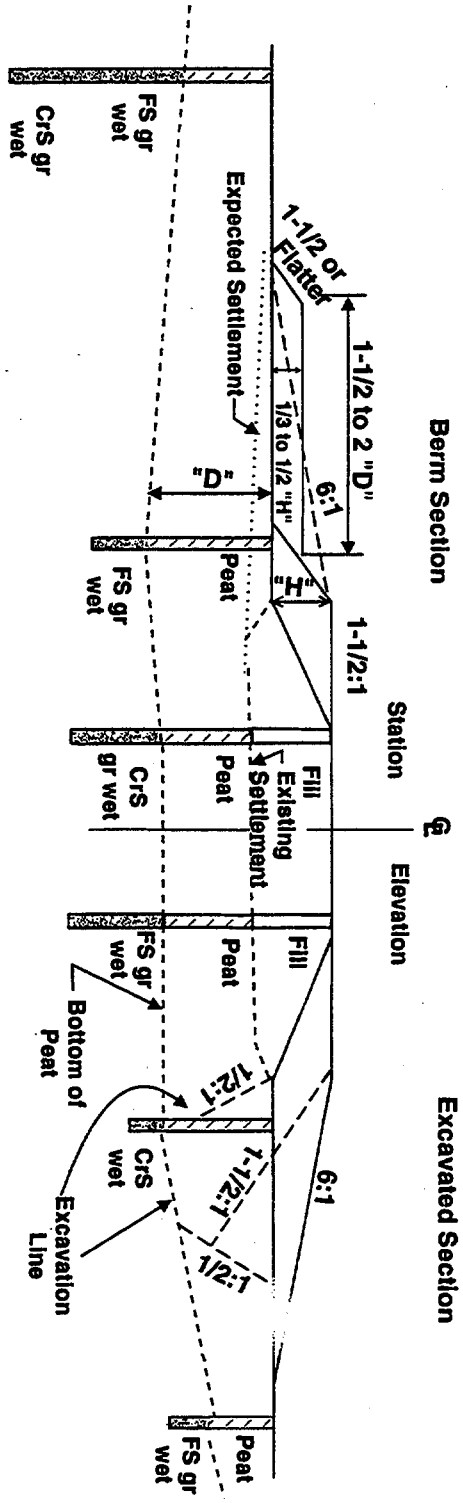
2. Floated Embankment. In the placement of a floated embankment, existing trees, brush, and other surface vegetation is clear cut with only minimal disturbance to the in-place vegetative mat. The embankment is then placed directly over the existing swamp, proceeding from the toes in toward the road centerline, with the slope width, including berms, exceeding twice the depth of the swamp. Typically, this method requires less fill material because the swamp material does not have to be dug out. Placement of geosynthetics (geotextiles or geogrids) between the swamp and placed embankment materials may be desirable in order to minimize ruts, expedite work, increase allowable stresses on the swamp subgrade and to allow the floated embankment to act as a cohesive whole. It should be noted that the use of geosynthetics will not eliminate settlement. They will tend to make it more uniform.

   It may also be desirable to use light weight fill (e.g., wood chips, shredded tires, etc.) for floating the embankment.

   The use of geosynthetics for earth reinforcement (Mn/DOT's Specification 3733, Type VI), or the use of light weight fill, should be coordinated with and studied by the Foundations Unit. (See Section 5-2.05 for additional information on geosynthetics.)

3. Widened Section. The placement of additional fill adjacent to an existing roadway may be accomplished by either excavation and backfill or use of a floated embankment. In either case, particularly if excavation and backfill is used, the stability of the old road core should be considered. Figure 5-2.11 depicts embankment widening over weak soils. This type of work should be studied by the District Soils Engineer and coordinated with the Foundations Unit and/or Pavement Design Unit (Office of Materials Research & Engineering).

Figure 5-2.11. Cross section of embankment over existing roadway in swamp.



Note: Berm may be constructed in the winter. In any case the berm shall be constructed from outside inward and before the embankment widening.

5-2.02.02 ROCK EXCAVATION

Preliminary designs for rock excavation (subcuts and backslopes) should be based on Mn/DOT's Road Design Manual, Section 4-6.02.02. Final designs should be established only after completion of a detailed geologic investigation.

1. Subcuts. Subcut excavations require the determination of rock excavation method (mechanical or explosive) and transitions into and out of rock sections. Both transverse and longitudinal transitions should be provided in the design to minimize differential cracking. (Generally, rock designs provide for 6" of paid overbreak below the bottom of the rock subcut).

2. Rock Backslopes. Rock cuts averaging six feet or less in height should have the same backslope as the adjacent soil sections. Intermediate heights (about six to ten feet) should generally be laid back as soil slopes, but may be treated as a rock section if the rock is competent and not highly weathered, and especially if such cut heights lead into higher rock cuts. Cuts between 10 and 45 feet in height in competent rock generally require a backslope shaped by controlled excavation (such as presplitting or cushion blasting) and a ditch width of 12 feet. Rock cuts higher than 45 feet require similar controlled excavation methods, with a ditch width of 20 feet. The recommended ditch widths are based on minimizing rockfall onto the roadway, removal of fallen rock, and safety clear zone areas. The use of intermediate horizontal rock benches is not recommended. Rock backslope angles, which typically range between vertical and 1/4(H) to 1(V), should be based primarily on the rock mass characteristics, but may need to be modified due to right-of-way constraints. Assistance in determining the backslope angle, modification of ditch widths, or any special rock cut designs, is available from the Geology Unit (Office of Materials Research & Engineering).

   Blasting is normally required for the removal of rock and consideration must be given to the potential impact of blast vibrations on nearby structures and their occupants and waterwells. Under conditions where vibrations may have an adverse effect, they should be limited by contract language. Assistance in developing vibration controls should be obtained from the Geology Unit.

5-2.02.03 UNSUITABLE MATERIAL

Materials defined as unsuitable [topsoil, organic material (greater than 5% organic) debris, undesirable material, etc.] must be removed from the embankment foundation, unless otherwise designated. The unsuitable materials shall be disposed of in accordance with Mn/DOT's Specifications 2104 and 2105 (i.e., peat, muskeg, marl, muck, and other unstable materials

that are not to be used in the roadbed embankments shall be deposited in the areas indicated in the Plans, or elsewhere as approved by the Engineer). All other materials, including bituminous and concrete waste, that are considered unsuitable for use in the upper portion of the roadbed shall be placed in embankments at least three feet below the subgrade surface, or outside of a 1(H) to 1(V) slope down and outward from the shoulder lines on fills under 30 feet in height, or outside of a 1-1/2(H) to 1(V) slope down and outward from shoulder lines on fills over 30 feet in height; used to flatten the embankment slopes; or disposed of elsewhere as approved by the Engineer (Mn/DOT's Specification 2105).

For embankments of three feet or less in height, unsuitable materials should be removed from the upper four feet of the roadbed between the shoulder lines. See Mn/DOT's Specification 2105.3D.

5-2.02.04 EXISTING STRUCTURES/PAVEMENTS

Abandoned structures and other obstructions within the right-of-way that have no salvage value should be demolished and removed from the project area, with their location and disposition indicated on the Plans. Those structures that interfere with new construction should be demolished and removed prior to construction of the affected work. Typical salvageable materials are aggregate, topsoil, debris, and rocks not exceeding three inches in the greatest dimension. Removal operations should comply with Mn/DOT's Specifications 2102, pavement; 2103, buildings; 2104, pavement/miscellaneous structures; and 2442, removal of old bridges.

5-2.02.05 SLOPE PREPARATION

Embankments which are to be widened and have inslopes steeper than 4(H) to 1(V) require that the slopes be flattened to this slope, or flatter; or that steps (benches) be cut into the slope. See Mn/DOT's Specification 2105.3B. The material used for the embankment widening should substantially match the inplace embankment material in terms of textural classification, moisture and density.

5-2.03 STRUCTURE FOUNDATIONS

The foundation of a structure serves to transmit the weight of the building, bridge, or other structure to the supporting soil or rock below. Shallow foundations consist of spread footings, either continuous strips or individual pads, and are utilized when a soil stratum suitable for supporting the structure is located at a shallow depth. A continuous strip footing is one that supports a wall, while an individual pad footing is one that supports a single column.

When soft soils overlie a suitable supporting stratum at depth, the foundations are required to be extended to and supported by the deeper material by use of

deep foundations - piles and/or drilled shafts. Piles are structural members of timber, concrete, and/or steel, with a small cross-sectional area when compared to their length, and are usually driven into the ground by means of a large hammer or vibrator. Drilled shafts generally have larger cross sections and are constructed by drilling a cylindrical hole to the required depth, placing steel reinforcement in the excavated shaft, and backfilling with concrete.

All foundation borings and designs are performed by the Foundations Unit (Office of Materials Research & Engineering).

5-2.04 DRAINAGE/DEWATERING

All excavations, whether for embankments or structure foundations, should be maintained in a well-drained condition at all times. This requires removal of ground water and control of surface runoff during preparation of the foundation.

Failure to dewater excavations slows the construction process and can lead to unstable slopes and softened subgrade. In low-permeability silty and clayey materials, dewatering can generally be accomplished through a system of gravity ditches and sumps and pumps. In high-permeability granular soils or bedrock, where large quantities of water flow are required to be cut off, a more elaborate system consisting of dewatering wells may be required. Such construction water control is generally the responsibility of the contractor, but the Foundations Unit or Geology Unit (Office of Materials Research & Engineering) may be consulted to assess the potential impact on nearby structures and waterwells.

Control of surface runoff is also important. Surface runoff should be directed away from excavations. Ponding water will lead to softened subgrades, slowing the construction process and requiring additional earthwork. At the end of the day the embankment surface should be shaped, compacted and sealed (by rolling) to facilitate any surface run off. Stockpiled material should be kept at locations that will not inhibit site drainage. Temporary drainage facilities should be provided by the District Design Engineer.

5-2.05 GEOSYNTHETICS: GEOTEXTILES AND GEOGRIDS

Geosynthetics is a broad generic term for a family of synthetic materials, usually various polymers of petroleum, which are frequently used to enhance, replace, or make more cost effective use of, soil and rock materials. Included under geosynthetics are products such as geotextiles, geogrids, geonets, geomembranes, geocomposites, plus several others. Only geotextiles (5-2.05.01) and geogrids (5-2.05.02) will be briefly discussed here, because they are or will be the most frequently used. Geomembranes are impermeable and may be included as part of the overall design of maintenance facilities involving salt storage or brine ponds, or in ponding areas where leakage must be

prevented. (Short mention of geosynthetics may be also found within other specific chapters of this manual.)

In addition to the routine uses of geosynthetics, such as for wrapping perforated pipe or under riprap, the need for and use of such materials should be recommended by the District Soils/Materials Engineer. Geosynthetics are generally not recommended by Mn/DOT for applications such as lining trenches for drainage, to prevent transverse reflective cracks in bituminous overlays, or to reduce the required design thickness of granular or bituminous materials in paved roadways. However, the use and acceptance of geosynthetics for both conventional and unconventional designs is constantly changing. The use of geosynthetics should comply with applicable specifications: Mn/DOT 2511.3 for Riprap, 2501.3C for Pipe Culverts, 2502.3C for Subsurface Drains, and all other pertinent special provisions. Depending on the specific requirements, assistance with such designs can be provided by the Geology Unit, Foundations Unit, or Pavement and Base Design Unit. Four references listed at the end of this chapter will be helpful for additional reading and may be borrowed from the Geology Unit: two FHWA publications, a text book by Koerner and the student text for Grading and Base II, Course Book. Please see one of these references for more specific characteristics of geosynthetic materials and also design procedures for various applications.

5-2.05.01 GEOTEXTILES

Geotextiles are permeable geosynthetics, often referred to as fabrics. They typically perform five functions, the most important of which are filtration, separation and reinforcement. Mn/DOT 3733 specifies six types of geotextile and summarizes the use for each type. Types I, II, III, and IV function primarily as filters, i.e., permit the movement of water, but retain soil particles. These four types of geotextile involve the greatest number of individual applications by project (due to uses such as wrapping perforated pipe, wrapping concrete culvert joints or as replacement for granular filter under riprap), but do not constitute the largest volume of use. Type V functions primarily as a separator i.e., prevents two dissimilar materials from mixing, and it may also provide some soil reinforcement. The Type V fabric likely includes the largest volume of geotextile used in Minnesota. Typical uses might include: bottoms of subcuts in soft/wet soils to facilitate equipment movement and promote compaction of backfill material, culvert designs to separate the granular bedding from underlying soft and/or wet soils, and occasionally for separation/reinforcement designs for low fills over shallow swamps.

Type VI geotextile is used for reinforcement, i.e., it facilitates construction and provides additional strength to minimize the possibility that traditional or light weight fill materials will "fail" during or after construction. Applications such as "floating" fills over swamps or steepening side slopes are examples of the reinforcement function. Note that in Mn/DOT 3733, Type VI specifications are site specific and recommendations for use and geotextile properties can/should

be provided by the Foundations Unit. The decision whether to dig out or displace swampy soils with granular material vs. "floating" the fill over the swamp by the use of geotextiles (or possibly a geogrid) involves consideration of factors such as: function/type of roadway (tolerance for some unevenness), depth of swamp, environmental concerns with both excavation and disposal of organic material, when the roadway must be open to traffic (settlement period), and availability of and haul distance for granular replacement soil. It should be noted that geosynthetics will not prevent settlement, but rather will provide more uniform settlement and likely reduce total settlement to some extent.

Geotextiles may be either woven, nonwoven or knit, with the nonwoven and knit varieties generally having greater elongation (stretch) under load. Nonwoven fabrics are almost universally supplied for Mn/DOT Type I thru IV applications. Knit geotextiles should only be used to wrap perforated drain pipes. While both woven and nonwoven materials will meet the requirements for Type V, woven slit-films are most commonly supplied because they are less expensive. Major reinforcement designs (Type VI), almost always require high-strength woven products.

Generally, geotextile recommendations should be referenced to one of the six types specified in Mn/DOT 3733. For various reasons, modified types are often appropriate, and this should be clearly indicated to avoid confusion. Again, note that project-specific specifications are usually required for Type VI reinforcement applications. Typical specifications for separation and reinforcement applications are available from the Special Provisions Unit.

5-2.05.02 GEOGRIDS

Geogrids are a small but growing subset of geosynthetics and are a network of integrally connected polymer elements formed into a very open, grid-like configuration, i.e., they have large apertures (1/2" - 1 1/2", or more). Individual grid elements may be solid or composed of several strands of polymer. These products have relatively low elongation (stretch) under load and are primarily used in reinforcement applications. Grid products are currently being widely used in conjunction with modular block retaining walls. Geogrids and high strength geotextiles often compete for the same market niche. Assistance in the use of and specifications for geogrids can be provided by the Foundations Unit, in the Office of Materials Research and Engineering.

5-3.0 PAVEMENT DESIGN

Pavement design involves the determination of the most cost-effective combination of pavement type and structure for a roadway, which will be functionally and structurally adequate during the design life of the pavement. Consideration of the traffic conditions, environmental conditions, availability of materials, and material properties are required for an adequate design, while at the same time ensuring the constructability and maintainability of the designed pavement.

Pavement design should include a consideration of funding constraints and an exhaustive economic analysis to determine the best choice of pavement type and structure with the lowest life-cycle costs. Past procedures placed the most emphasis on the determination of pavement type and initial thickness. However, it has since been acknowledged that it is important to accurately predict pavement performance, forecast future maintenance, and plan rehabilitation needs and treatments. It is such a combination of initial design and planing for future maintenance and rehabilitation needs that constitutes a pavement design strategy.

This section contains information on the Department's approach to determining pavement type and the subsequent design of the pavement structure which will best meet the conditions discussed above. Specifically, the basic requirements of a pavement section are to:

- Provide a durable travel surface and structure that will withstand both the repeated wheel loads of vehicles and the effects of the environment;
- Provide adequate drainage capability for the removal of excess water from the roadway and the subsurface portions of the pavement, so that the structural integrity of the pavement remains intact; and
- Provide a surface with adequate friction characteristics.

5-3.01 PAVEMENT TYPES

A number of pavement types are currently used. The types of pavements constructed by Mn/DOT to meet the requirements described above are bituminous (asphaltic concrete), portland cement concrete (PCC), and composite pavements. These pavement types are described below.

5-3.01.01 BITUMINOUS

Bituminous pavements, also called flexible pavements, are based on the concept of using increasingly stiffer layers from the natural roadbed material upwards, to carry traffic loads. These layers each contribute to the structural capacity of the pavement and are topped with a bituminous wearing course.

There are two main types of bituminous pavements. The first type is the conventional flexible pavement, which consists of bituminous pavement placed on an aggregate base on a prepared subgrade.

The second type of bituminous pavement is a full-depth bituminous pavement. These consist of bituminous pavements placed directly on the subgrade. A variation of this pavement type, which consists of bituminous pavement placed on a granular subbase layer, is called deep-strength bituminous pavement.

5-3.01.02 PORTLAND CEMENT CONCRETE

Portland Cement Concrete (PCC) pavements, also known as rigid pavements, are pavements that have a PCC wearing surface. The PCC surface may be placed directly on the subgrade, but in most instances a base layer(s) is placed between the two. PCC pavements are classified into four types, based on the nature of the transverse joint spacing and reinforcement used to limit temperature cracking within the pavement. These are jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), continuously reinforced concrete pavement (CRCP), and prestressed concrete pavement (PCP).

JPCPs are jointed, unreinforced, PCC pavements made up of short slabs, typically up to 20 feet long, with all transverse joints doweled. The short joint spacing eliminates the likelihood of the formation of transverse cracks within a single slab and reduces warping and curling stresses. Mn/DOT uses an effective 15-foot joint spacing (13-16-14-17 feet) for JPCP slabs which are less than 10.5 inches thick. JPCP slabs which are 10.5 inches or greater in thickness have a 20 feet joint spacing (since 1992).

JRCPs are PCC pavements that are reinforced with steel bars or mesh. The reinforcement is added to keep transverse cracks tight which invariably form between the joints, especially in longer slabs. Slab lengths are typically between 20 and 40 feet, and all transverse joints are doweled. Mn/DOT uses 27-foot joint spacing for JRCP. (Note: Mn/DOT currently constructs only JPCPs).

CRCPs are PCC pavements that are continuously reinforced and have no transverse joints, except for construction joints and at structures. Enough reinforcement is provided so that the transverse temperature cracks which form are spaced between one and eight feet and are less than 0.02 inches wide. Mn/DOT started CRCP construction in 1963, but discontinued their use in 1970 due to poor performance.

PCPs are concrete prestressed slabs. Post-tensioning of reinforcement placed in ducts in the concrete slabs, the usual method of prestressing, exerts compressive forces on the pavement slabs and eliminates temperature cracks or keeps them tight when they do form. These pavements are constructed in

slabs that range from 200 to 400 feet in length. They are considered experimental by most highway agencies and have not been used in Minnesota.

5-3.01.03 COMPOSITE

Pavements which are made up of a combination of PCC and bituminous layers are given the generic name "composite" pavements. Usually, these composite pavements result from rehabilitation treatments (overlays); however, they may rarely be new designs.

5-3.02 PAVEMENT DESIGN CONSTRAINTS

The pavement design procedure involves the selection of a pavement type, initial structure, and maintenance and rehabilitation strategies, which will be the most economic and adequate alternative for carrying the combination of projected traffic and environmental loadings during its design life. A number of constraints influence that choice, and their full consideration is required if the best solution is to be obtained. The major constraints that have to be considered are discussed below.

5-3.02.01 DESIGN PARAMETERS

There are a number of constraints associated with obtaining necessary and accurate information on traffic conditions, environmental conditions, and material properties required for the design process.

The problems associated with determining traffic loading for pavement design are twofold. First, there is the need to forecast the expected traffic volumes and classifications during the design life of a pavement. Second, the accurate characterization of axle-load distributions during the design life is required. Traffic volume and classification forecasting techniques are still approximations and need further improvement. The axle-load distributions of truck traffic are also generally site specific and based on sparse weigh-station data.

Environmental effects also have a considerable effect on the performance of pavements. The designer should directly consider the provision of good drainage and the treatment of swell- and frost-susceptible soils in construction, rather than try to account for such environmental factors by increasing thickness.

In addition, the adequate characterization of pavement materials and the use of the proper material properties have a large effect on the design obtained for a particular case. The designs arrived at may not meet the requirements of the design life, where such inputs do not accurately characterize the conditions of materials and the expected trends in their properties with time, loading, and environmental variations. For example, it is well known that, in most cases, the materials used in pavements do not exhibit elastic or linear properties. However,

in most design processes, pavement materials are modeled as linear elastic materials — an apparent limitation.

The availability of good quality materials for pavement construction is another constraint that is often faced across the country. Minnesota is no exception; and, increasingly, pavement engineers are having to use some materials of less than desirable quality. In most instances, such materials are stabilized or treated in some manner to allow their effective use. Design procedures must take such constraints into account.

Aside from the constraints discussed above, there is always the important matter of designing a pavement that is constructible and maintainable. The initial constructability of the pavement and the successful implementation of maintenance and rehabilitation treatments are important aspects that have to be considered in a successful pavement design process.

5-3.02.02 FUNDING

Pavement projects increasingly compete with projects in other sectors and for funds available for new construction, as well as maintenance and rehabilitation. It is, therefore, incumbent upon the pavement engineer to document that the benefits to be accrued from a pavement design are at least as economically attractive as those of other projects. Even with the acquisition of funds for a project, it is often the case that the funds available are not adequate for the projected needs. An effort is then necessary to ensure that the best use is made of the limited funds to provide, to the greatest extent possible, a design which meets minimum requirements.

5-3.02.03 ECONOMIC ANALYSIS/PAVEMENT SELECTION

A major facet of pavement design is an economic analysis to determine the most cost-effective pavement to meet the particular needs of a given project. In such analyses, economic principles, which take into account the time-value of money, are used to compare alternatives to determine the most cost-effective strategy of pavement type, initial structure, and maintenance and rehabilitation treatments for the analysis period of a given project.

A major constraint of the process is the use of imprecise information in the analysis procedure. Material costs, discount rates, salvage values, service life, and other such factors often have to be approximated as they are tied to events which may occur years into the future. However, by applying sound principles and analysis techniques, it is possible to obtain results that will adequately answer the necessary questions about the best selection to make in any particular case. In Minnesota, a comparison of the total annual costs of alternative strategies is used in such analyses.

5-3.03 ANALYSIS METHODS

Pavement analysis methods can be divided into three major groups. These are the empirical, mechanistic, and mechanistic-empirical analysis methods. The concepts behind these methods are described in this section, with references to some existing pavement analysis methods.

5-3.03.01 EMPIRICAL

Empirical pavement analysis methods are experience-based methods which have been developed by observation of the performance of actual pavements in service. Examples of such pavement analysis methods are those that were derived from the Maryland, WASHO, and AASHO Road Tests. Such empirical methods are not based on scientific principles of how pavement materials behave when subjected to traffic and environmental loadings, but on what practical experience has shown about their performance.

Although empirical methods of analysis do not allow the design of pavements which fall outside the scope of the original experience the methods are based on, they have provided adequate solutions in a considerable number of instances. As an example, the empirical pavement analysis methods used by the Department in the design of flexible and rigid pavements are discussed in the following paragraphs. The analysis procedures for flexible pavements were developed from research studies conducted by the Department; the analysis method for rigid pavements is based on the 1981 Modified AASHTO analysis procedure for rigid pavement design.

1. Mn/DOT's Investigations 183 and 195 (Flexible Pavements). The two types of flexible pavements designed by the Department are asphaltic concrete pavement with aggregate bases and full-depth asphaltic concrete pavements. Research conducted by Mn/DOT in Investigation

   183 forms the basis of the asphaltic concrete on aggregate base pavement analysis; and the analysis method for the design of full-depth asphaltic concrete pavements is based on the results of research conducted in Investigation 195.

   In Investigation 183, the results of the AASHO Road Test were used as the basis for the development of an analysis method for the design of asphaltic concrete pavement on aggregate base. Data was collected from 58 test sections in Minnesota, selected to include a wide variety of soil types, pavement structures, and traffic conditions. Using this data, the effects of the various subgrade soils, local construction procedures, local paving materials, climatic conditions, and cumulative traffic loading on pavement performance were evaluated. The performance of the test sections, as determined by serviceability trends, were then related to the structure, subgrade R value, and also to Benkleman beam deflections to

allow the extension of the AASHO research results to the analysis and design of flexible pavements in Minnesota.

The research conducted in Investigation 195 employed data collected from 26, full-depth, test sections, with a variety of thicknesses, and constructed on an assortment of foundation soils. Benkleman beam deflection and pavement temperature measurements were taken on the test sections in the spring, summer, and fall. These data were used to determine the effects of temperature and seasonal variation on the deflections. They were also used to develop correction factors for application to measured deflections to allow their conversion to a standard peak season deflection at 80° F (26.7° C). These standard deflections were then compared to deflection measurements on flexible pavements with aggregate bases, and a relationship was developed between the thickness of full-depth pavements and the "granular equivalency" of comparable granular base pavements. This relationship was used to develop a design chart for full-depth pavements, which was the deflection equivalent of the granular base, flexible pavement, design chart then used by the Department. The structural design procedure for the two types of flexible pavements is based on the design-lane, cumulative, equivalent single-axle loads (ESALs) over a 20-year period; and a subgrade strength based on the design R value.

Mn/DOT also uses an empirical procedure for rating flexible pavements for designation as 7-, 9-, and 10-ton routes. These load ratings refer to spring-time thaw restrictions and are based on deflection measurements. The procedures used in determining the spring-time carrying capacity of a pavement are based on Investigation No. 603. A computer program (TONN) was developed, which incorporated Investigation No. 603, and is used for estimating the spring-time load carrying capacity. In addition, a procedure is available to determine if and when a particular roadway has the strength to withstand 10-ton loads without undue loss of useful pavement life ("Methods for Determining 10-Ton Routes on Flexible Pavements based on NDT").

2. Modified AASHTO (Rigid Pavements). The rigid pavement design procedure used by the Department is an empirical method which is based on a combination of the modified 1981 AASHTO pavement design procedure and the knowledge gained from the performance of rigid pavements in Minnesota. The structural design procedure is based on the ESAL applications over a 35-year period; and the modulus of subgrade reaction, *k*. These may be adjusted, depending on whether the pavement edge is protected or unprotected.

5-3.03.02 MECHANISTIC

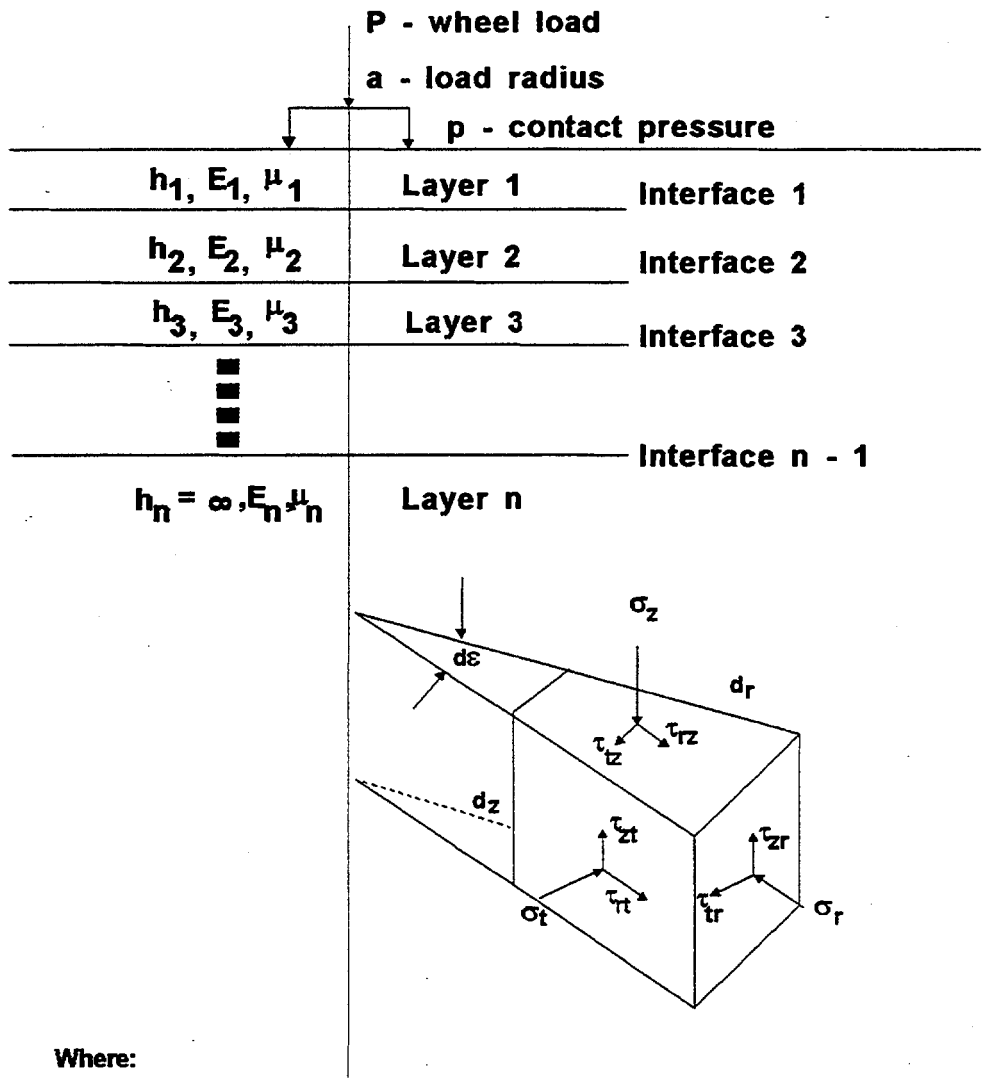
In recent years, there has been a move towards the use of mechanistic analysis procedures for the design of pavements. Mechanistic procedures involve the use of analytical methods to determine pavement responses and the actual stresses, strains, and deflections that occur in pavements from the application of traffic and environmental loads. They are based on engineering mechanics principles. By allowing proper characterization of all aspects of a pavement in terms of actual material properties, variability in the properties, traffic loading, environmental variations, and the physical aspects of the different types of pavements, mechanistic procedures seek to provide a more accurate representation of the pavement structure.

Another useful aspect of mechanistic analysis for pavement design is the use of damage and distress models in the analysis process to check designs. Damage models permit a prediction of what combination of traffic and environmental loading for a particular pavement structure, with defined material and physical characteristics, would accumulate enough damage to result in failure, as determined by some, pre-defined, distress failure criteria, e.g., fatigue cracking, rutting, or faulting. The distress model allows the prediction of the severity and extent of the various distress types in the pavement, subjected to a combination of traffic and environmental loadings over time. With this combination of structural response and damage and distress prediction models, mechanistic analysis procedures yield designs which more closely simulate real pavement conditions.

Elastic layer and finite element theory are the basis of the two major mechanistic analysis approaches currently used in the determination of structural response in pavements. These procedures, by themselves, are not design methods and have to be complemented with the proper damage and distress models in a comprehensive mechanistic pavement design procedure.

1. Elastic Layer. The application of elastic layer theory to the analysis of pavements has primarily been in the area of flexible pavement design. In this type of analysis, the pavement is considered as an axisymmetric multi-layered structure, with each layer extending infinitely in the horizontal direction, and with a semi-infinite bottom layer, as shown in Figure 5-3.1. Each layer is assumed to be homogeneous, isotropic, and linearly elastic in response to stress. The material properties of the layers are characterized by a modulus of elasticity and a Poisson's ratio. It is assumed that there is full friction between the layers and that the load on the top layer of the pavement is applied through a circular area. Information on the stresses, strains, and deflections at any location from the axis of symmetry can be computed using computer software. Elastic layer analysis programs are now available that allow for a finite bottom layer, non-linear elastic material response, partial or no friction between layers, and multiple loads.

Figure 5-3.1. Idealized elastic multi-layer pavement structure.



Where:

h_n = Thickness of layer n.

E_n = Elastic Modulus of layer n.

μ_n = Poisson's Ratio of layer n.

s_i = Stress component in the ith direction.

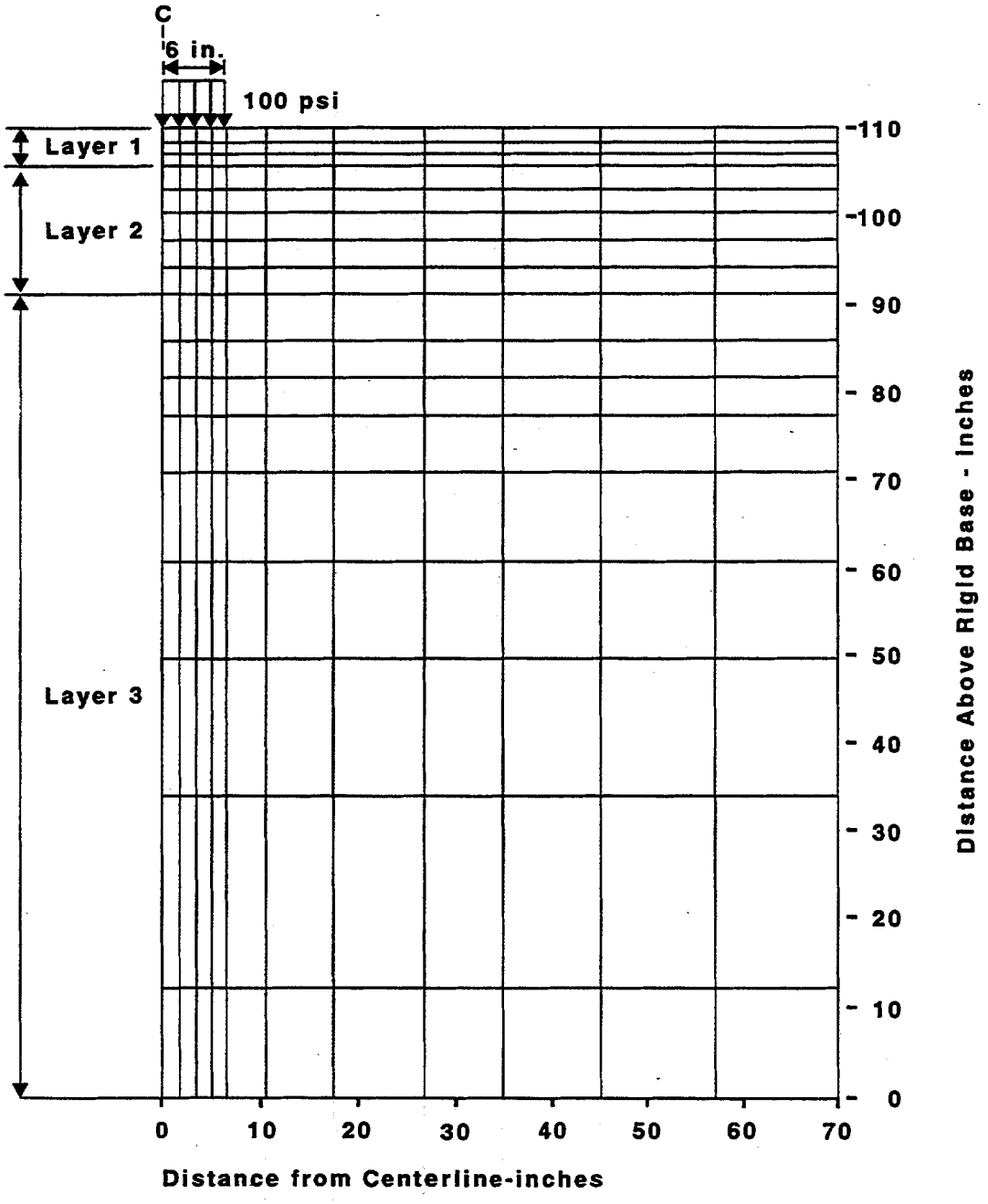
t_{ji} = Stress component in the ith direction in the plane ji.

Multi-layered elastic analysis is not applicable to the solution of stresses, strains, and deflections at discontinuities in pavements and is, therefore, not suitable for solving rigid pavement problems (because of joints and cracks), except at the interior of slabs. There are also some limitations connected with the analysis of pavements on granular layers. Multi-layer elastic analysis programs sometimes yield solutions which show excessive tensile stress in the granular layers, which are known to have no tensile strength. Examples of elastic layer programs for pavement response analysis are the ELSYM 5 and BISAR programs.

2. Finite Element. The finite element analysis method is applicable to the design of both rigid and flexible pavements and can take into consideration the effect of discontinuities, such as joints and cracks; design parameters, such as dowels and load transfer; and the stress sensitivity of materials.

For the application of finite element techniques to the analysis of a pavement, the pavement structure is divided into rectangular elements connected at nodes, as shown in Figure 5-3.2. Each rectangle represents a combination of four triangular elements with their common node at the center eliminated. The continuous and non-linear variation of stress and strain in the real pavement are replaced by a constant stress and strain in each triangular element by assuming a linear variation in the displacement at the nodes. For each triangular element, a matrix of the stiffness of each node is set up relating the load and displacements. These stiffness matrices of all the elements are combined to obtain a symmetric banded matrix for the pavement structure. With known boundary displacement conditions, this matrix is modified, and the system of linear equations is solved to give the displacements at all nodes. From these nodal displacements, the stresses and strains of each triangular element are determined. Then, the values for the four triangular elements of each rectangle are combined to give the average stress and strain at the center. The smaller the elements of the finite element representation of the pavement, i.e., the finer the mesh used, the more accurate are the solutions obtained. However, this increased accuracy has to be balanced with the increased computer time and cost required to solve the larger system of linear equations.

Figure 5-3.2. Finite element configuration used for analysis of homogeneous and layered systems (from Dehlen, 1969).



There have been many enhancements to this elementary application of the finite element principle to allow the technique to be used for more complex and realistic problems. Finite element programs now exist which can take into account the effects of temperature variations in pavements, and there are now provisions for the consideration of non-linear and inelastic materials in finite element analysis. Three-dimensional finite element programs are also now available, but their use is limited as a result of the very high computer cost and storage capacity needed to run such programs. Examples of finite element analysis programs available for the analysis of pavement response include the ILLIPAVE, ILLISLAB, and JSLAB.

5-3.03.03 MECHANISTIC-EMPIRICAL

Mechanistic and empirical design concepts can be combined to obtain procedures which incorporate mechanistic parameters and empirical performance models in the design process. In such procedures, called mechanistic-empirical procedures, an attempt is made to use mechanistic responses (stress, strain, and deflection) and empirical-based damage and pavement distress prediction regression models as the basis for pavement analysis and design. Such techniques, by relying partly on mechanistic principles, allow the analysis and design of pavements which more closely take into account actual pavement conditions. The Asphalt Institute Manual Series (MS-1) design method for flexible pavements and the Portland Cement Association (PCA) design procedure for rigid pavements are two examples of design methods which incorporate mechanistic and empirical principles.

Another method which attempts to move the design of pavements toward the mechanistic-empirical approach is the method presented in the 1986 and 1993 AASHTO "Guide for Design of Pavement Structures." The empirical design procedures developed as part of the AASHO Road Test in 1961, with subsequent revisions in 1972 and 1981, are the basis of the design procedures presented in this guide. Some of the aspects of mechanistic-empirical principles incorporated into the design process include the indirect use of mechanistic principles for determining the coefficients of drainage and load transfer. The procedures for evaluating the effects of seasonal variation and the use of quasi-elastic modulus parameters, such as the resilient modulus (M_r), for characterizing pavement materials, represent an introduction of some aspects of mechanistic principles into the design process. This method and the Asphalt Institute procedure are briefly presented below to illustrate the various aspects of these procedures.

1. Asphalt Institute MS-1 Design Method. The Asphalt Institute MS-1 method for the design of asphaltic concrete pavements is one of the design methods which can be considered to be a true mechanistic-empirical procedure. In this method, the pavement is considered as a multi-layered elastic structure, with each layer characterized by an elastic

or resilient modulus and a Poisson's ratio. The horizontal tensile strain at the bottom of the asphaltic concrete layer and the vertical compressive strain at the surface of the subgrade are the critical criteria used in the design procedure.

The object of the procedure is to provide a pavement structure which, for the given pavement materials, environmental characteristics, and traffic loading, would be adequate to resist fatigue failure and subgrade rutting resulting from the tensile strain and compressive strain, respectively. A manual procedure is available. The mechanistic pavement analysis Chevron N-layer program, which can consider a wide range of material input parameters, environmental conditions, and traffic conditions, has also been incorporated in the computer program, DAMA, for determining the pavement layer thicknesses.

2. 1986/1993 AASHTO Design Method. The current AASHTO design method for pavement structures includes a number of improvements, based on mechanistic principles, to those procedures which existed prior to the publication of the 1986/1993 AASHTO Guides. There are two design equations, one each for asphaltic concrete and PCC pavements. The previous design procedures were, for the most part, empirical and were based directly on the results obtained from the AASHO Road Test. In some instances, such as in the case of rigid pavement design, these results were supplemented with then-existing theory and pavement design methods.

   In the procedures presented in the new guide, an attempt is made to incorporate some mechanistic-based principles. In the characterization of material properties for design, for example, mechanistic-based parameters are used in place of empirical parameters, such as the CBR. Characterization of pavement materials for use in this procedure is based on the resilient modulus. The method of determining the damaging effect of seasonal variations on the pavement design also incorporates some mechanistic principles, as do the procedures for establishing the coefficients for drainage and load transfer. AASHTO is also currently moving towards the development of a mechanistic overlay design procedure for flexible and rigid pavements.

5-3.03.04 SUMMARY

The analysis procedures presented above form the basis of currently used pavement design methods. Most of the pavement design methods, especially in the area of flexible pavement design, are based on empirical principles. However, with a number of computer programs which employ mechanistic principles now available, it is appropriate to move towards the use of mechanistic design methods. The advantages associated with the use of such methods are many. For example, by using material input parameters which are related to

fundamental material properties, the actual responses of the materials which are used in design can be modeled. This will improve the design for a particular material, as well as permit the use of a wider range of materials. The effects of physical characteristics, such as loss of subgrade support, widened lanes, and load transfer in rigid pavement design, can also be modeled more accurately in a mechanistic design. The acceptance of the need for such improvements is reflected in the mechanistic orientation of the design procedures presented in the subsequent sections of this manual.

5-3.04 MATERIAL PARAMETERS

Some of the major inputs necessary for the analysis and design of pavement structures are the parameters which characterize the materials of the roadbed or foundation soils and of the pavement construction materials themselves. The proper characterization of such materials is essential to the development of a design which would be adequate for the projected traffic loading and the effects of the variations in the environment. Typically, the material parameters used to characterize the roadbed soils and pavement construction materials are either derived empirically or are mechanistic based.

In the past, most of the material inputs for pavement design, such as the CBR, have been empirically based. Although such empirical parameters have served pavement designers well, by their very nature it has been difficult to design pavements with new materials or that make new use of old materials, as such use would require extrapolation beyond the original inference space. Consequently, attempts have been made to use mechanistic-based material parameters in the design of pavement structures. In this section, the material parameters used by the Department in the design of pavement structures are discussed. Although some of the empirical-based material parameters that have been used in the past are presented, emphasis is placed on the mechanistically derived parameters used to characterize pavement and roadbed materials.

5-3.04.01 ROADBED SOILS

The parameters used by Mn/DOT to characterize roadbed soils for pavement design have included the modulus of subgrade reaction, resilient modulus, the R value, and the soil support value. The parameters are defined, and the important considerations that have to be taken into account in the use of these material parameters in pavement analysis and design are discussed below.

1. Resilient Modulus. The resilient modulus (M_r) has become accepted as the most appropriate parameter for defining the stiffness of subgrade soils. It characterizes the behavior of subgrade soils under a moving wheel load, and it is amenable to mechanistic analysis.

The M_r is dependent on a number of factors and testing conditions which must be taken into account when determining the appropriate M_r chosen for design. The magnitude of the applied stress affects the M_r of most soils. It is necessary, therefore, that the stress used in the determination of the M_r be representative of the conditions expected in the pavement structure to be designed.

For cohesive soils, the M_r varies inversely with the deviator stress. ($\sigma_D = \sigma_1 - \sigma_3$). A typical M_r response for fine-grained soils is shown in Figure 5-3.3. For triaxial conditions, the M_r of cohesionless or granular soils is also affected by the applied bulk stress — the sum of the principal stresses. ($\sigma=\sigma_1 + 2\sigma_3$) The relationship for such soils is shown in Figure 5-3.4. Existing data also show that the M_r of granular soils is affected by the degree of saturation, with the M_r increasing with decreasing degree of saturation for a particular stress state. AASHTO and ASTM are working on a standard test for M_r, which should be available soon.

Figure 5-3.3. Idealized resilient modulus curve for a fine-grained cohesive soil.

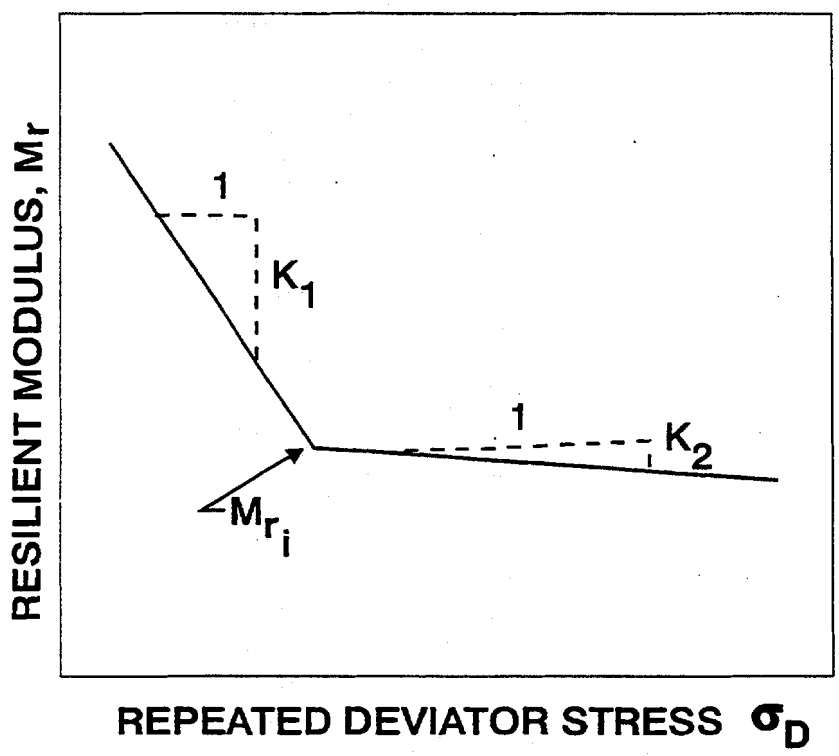
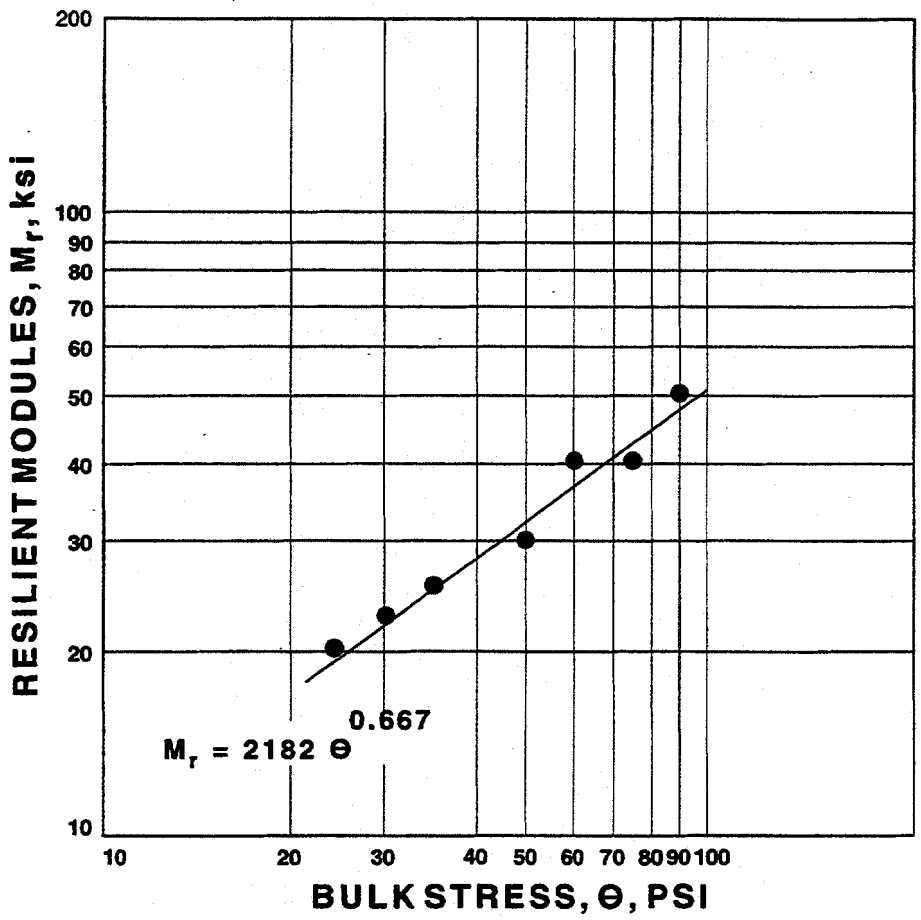


Figure 5-3.4. Resilient modulus - θ relation for a sandy gravel (AASHTO A-1-b (0)).



A discussion of the engineering significance of M_r and a brief summary of the test procedures for its determination are presented in Sections 3-2.03.04 and 4-2.06.05, respectively.

2. Modulus of Subgrade Reaction. The modulus of subgrade reaction (*k*) is used to characterize subgrade soils for rigid pavement design. The modulus of subgrade reaction is measured on the in-place soil on which a pavement is to be built. To determine the *k* value, a stress is applied to the subgrade through a 30-inch diameter plate, and the deflection is measured when the rate of deformation decreases to a predetermined constant. The modulus of subgrade reaction is calculated using Equation 5-3.1.

$$k = \frac{p}{d} \qquad \text{Eq. 5-3.1}$$

where

k = modulus of subgrade reaction, psi/in.;
p = stress, applied to the subgrade, psi; and
d = deflection, measured at the center of the load plate, in.

Typically, large loads are required in order to obtain the stresses which will induce the kind of measurable deflection necessary for the determination of the *k* value. Consequently, although the concept of modulus of subgrade reaction has been in existence for some time, few actual determinations have been carried out in the field in the past. As a result, the *k* value has been correlated to soil classifications and soil strength parameters, such as the R value.

As part of Mn/DOT's Investigation 183, fractional plate bearing tests and Hveem Stabilometer R-value tests were conducted for 50 test sites. Regression analysis gave the following relationship (Equation 5-3.2), which is particularly suitable for Minnesota conditions.

$$k = -1.17 + 63\sqrt{R} \qquad \text{Eq. 5-3.2}$$

where

k = modulus of subgrade reaction, psi/in., and
R = R value of the subgrade.

This relationship should be used to convert R values to equivalent *k* values for subgrades. Some agencies routinely increase the *k* value when stiff base layers are used. This procedure is not recommended in Minnesota because it does not represent the actual support a pavement slab will experience.

3. R Value. The engineering significance of the R value is discussed in Section 3-2.03.04, and a summary of the test procedure for its determination is presented in Section 4-2.06.05.

For design purposes, the selection of the R value is the responsibility of the District Soils and/or Materials Engineer, in conjunction with the Pavement Design Engineer. Since small changes in the R value considerably influence the structural requirements of pavements, its determination is a critical step.

R values are difficult to determine because of the limitations of the test itself, variations within a single embankment soil classification, variations in an embankment as constructed, and changes in environmental conditions. As a result, the selection of the appropriate R value to use in design is left to judgement and must be based on the test data obtained from the geotechnical survey. In current practice, for preliminary design the mean R value minus one standard deviation is often selected as the design value.

Table 5-3.1 in Section 5-3.05.02 establishes guidelines for the frequency of sampling for the determination of R values as a function of the subgrade soil texture. For small projects, where it is impractical to obtain R values from tests, estimates for design can be obtained from Table 5-3.2 in Section 5-3.05.02, which gives the R value as a function of the AASHTO subgrade soil type.

4. Soil Support Value. The use of a soil support value (S) for classifying subgrade soils was first introduced in the design procedures which resulted from the AASHO Road Test. This material characterization parameter cannot be obtained from a test and has to be established by correlations with other standard soil parameters, such as the CBR and R value, using the AASHO Road Test results.

At the Road Test, an S value of 3.0 was arbitrarily assigned to the subgrade soils of the test sections. By studying the performance of the pavement structures with sufficiently thick bases to minimize the effect of the silty clay subgrade soils, a soil support value of 10 was selected to represent the support offered by a thick crushed-rock base on a subgrade that would provide good performance. The soil support value for all other subgrade soils is then determined on the scale established by these two points, based on the following relationship (Equation 5-3.3).

$$\log W_{t18} = \log N'_{t18} + 0.372\,(S_i - S_r) \qquad \text{Eq. 5-3.3}$$

where

W_{t18} = total load applications for any condition, *i*;
N'_{t18} = total load applications for Road Test conditions;
S_i = soil support value for any condition, *i*; and
S_r = soil support value for Road Test conditions.

Because of the arbitrary nature of this scale, it is important to apply sound engineering judgment in the determination of the soil support value for a given subgrade soil.

5-3.04.02 PAVEMENT MATERIALS

Pavement materials have been characterized in a number of ways. The Department uses resilient modulus to characterize the strength of bituminous and base layers of pavements. As a fundamental material property, the resilient modulus best expresses the stress-strain relationship of materials subjected to

loads. A granular equivalent (G.E.) is also used by the Department to characterize the supporting strength provided by the bituminous and granular layers of flexible pavements, in comparison to a selected, high-quality, aggregate base. For concrete layers, the modulus of rupture is used to characterize strength.

1. Bituminous Resilient Modulus. There are a number of parameters for characterizing the modulus or stiffness of the bituminous layers in pavements. However, the resilient modulus (M_r) has gained wide acceptance for use in pavement design. The diametral resilient modulus of bituminous mixtures is determined in a test that involves the repetitive dynamic loading of a cylindrical specimen. The M_r is calculated from the magnitude of the repeated load and the measured total deformation using Equation 5-3.4.

$$M_r = P(\mu + 0.2734)/\delta t \qquad \text{Eq. 5-3.4}$$

where

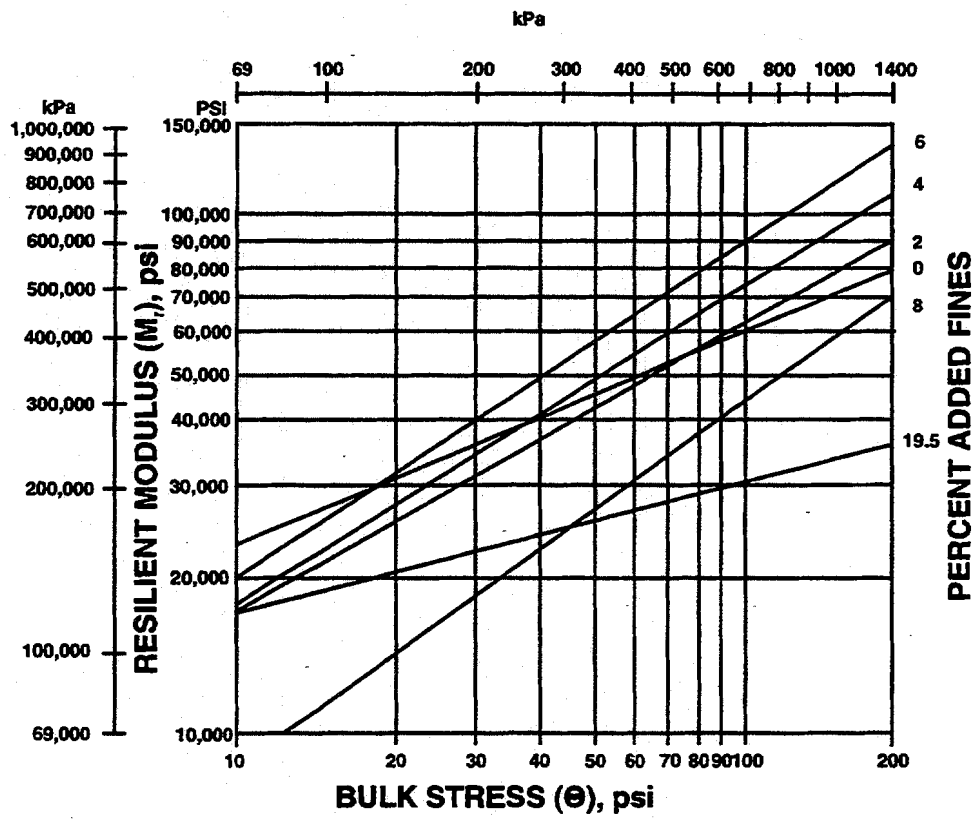
P = magnitude of dynamic load, lb.;
μ = Poisson's ratio (taken as 0.35 in most cases);
δ = total deformation, in.; and
t = thickness of specimen, in.

Since the M_r is influenced by temperature, in this test the M_r is determined at 40, 70, and 100° F. These values represent the general range experienced by bituminous mixtures in use.

2. Aggregate Base Modulus. The M_r is the parameter used most often in the mechanistic characterization of granular base materials. The factors that influence the M_r of materials were discussed in Section 5-3.04.01. Of particular importance is the effect of confining stress state. The typical relationship between M_r for granular base materials and confining pressure is given in Section 4-2.06.05.

Figure 5-3.5 shows the relationship between M_r and bulk stress for a granular soil and shows how this relationship is influenced by the percent of fines. This figure shows the increase in M_r with increasing bulk stress, similar to the relationship shown previously for coarse-grained roadbed soils; and also shows that an increase in the percentage of fines in the granular base results in a decrease in the M_r. In addition, the M_r of granular soils has been shown to increase with increasing density, decreasing saturation, and increasing particle angularity.

Figure 5-3.5. Effect of added fines above original 5.5 percent, on resilient modulus of an aggregate base course (from Jorenby and Hicks, 1986).



3. Modulus of Rupture. The modulus of rupture, or flexural tensile strength, is widely used to characterize Portland Cement Concrete (PCC). The modulus of rupture is defined as the stress that will cause the extreme fibers in a test specimen to break in the conventional beam-breaking test.

Typically, the flexural test (ASTM C 78, AASHTO T 97) involves third-point loading of a specimen, measuring 6 x 6 x 20 inches, cast from the design mix. Tests are performed on 7-, 28-, and 90-day cured specimens to permit the determination of the relationship between time and modulus of rupture. The modulus of rupture (f_r) is generally calculated from the conventional flexural equation for a beam in bending (Equation 5-3.5).

$$f_r = \frac{PL}{bd^2} \qquad \text{Eq. 5-3.5}$$

where

f_r = modulus of rupture, psi;
P = maximum total load, lb.;
L = span length of beam, in.;
b = average specimen width, in.; and
d = average specimen depth, in.

This equation is only valid if the beam breaks within the middle third of the specimen. If the beam fractures outside of the middle third, but less than five percent of the span length from either support, L in the equation is replaced by 3*a*, where *a* is the average distance on the tension side of the beam between the line of fracture and the nearest support.

Center-point loading can also be used to determine the flexural strength of concrete, using specimens smaller than described above (AASHTO T 177, ASTM C 293). This method has been used by the Department in the past. The result from the center-point load test, however, is not as reliable as the third-point loading test. Unlike the third-point loading test, where the specimen is subjected to pure moment in the middle third with zero shear, substantial stresses and shear forces develop in the test specimen with center-point loading. Studies by the PCA indicate that center-point loading, on the average, overestimates the modulus of rupture by about 75 pounds per square inch in comparison to the modulus determined by the third-point loading test.

4. Granular Equivalency Factors. Granular equivalency (G.E.) factors provide a means of equating the structural performance of all bituminous and aggregate courses which make up a pavement structure, in terms of the structural performance of a selected, high-quality, aggregate base. Mn/DOT's Class 5 and 6 aggregate bases are used as the selected standards. The G.E. concept is convenient for defining or rating pavement structures in similar "units" for the purposes of comparison.

The concept of G.E., which is expressed in inches, is the product of Mn/DOT's Investigation No. 183 (1969), discussed previously. In this study, the total granular equivalent thickness of a pavement was defined in terms of the subgrade soil R value and the cumulative, 18-kip, equivalent single-axle load required to reduce the Present Serviceability Index (PSI) of a pavement to a terminal value of 2.5. The required G.E. for the various pavement materials was determined by assigning granular equivalent factors to them, on the basis of their contribution to the pavement strength, in comparison to the strength offered by a layer of Mn/DOT's Class 5 or 6 aggregate base. The equivalencies are shown in Table 5-3.3 (in Section 5-3.05.02). These equivalencies have since been used to determine the thickness of the various layers required in a flexible pavement.

5-3.04.03 ENVIRONMENTAL CONSIDERATIONS

The current Mn/DOT pavement design methods do not directly take into account the affect of environmental factors on pavements. Instead, the approach adopted is to take measures to minimize or completely eliminate the detrimental effects of environmental variables. No formal procedure exists, for example, for directly treating the effects of moisture and temperature on pavements.

Moisture effects can be broken down into the affect of moisture on the subgrade soils and the affect of moisture on the pavement materials. In the case of the latter, measures are taken to prevent the moisture content of the materials used for the pavement layers from exceeding maximum limits. These include the use of materials that are at the appropriate moisture contents to construct the pavement, and ensuring that the pavement has a relatively waterproof surface.

During the geotechnical survey, information is collected on the moisture condition of the subgrade soils. From this information, unsuitable soils are eliminated and replaced by borrow materials with the appropriate moisture content. For example, where frost-susceptible material exists and the water table is high enough to result in detrimental frost penetration and frost heave, the frost-susceptible material is replaced with non-frost-susceptible material to the depth of frost penetration.

In cases where the ground water table is high and it is estimated that inflow rates are high enough to be detrimental to the pavement, drainage should be provided to ensure adequate outflow of water from below the pavement structure. This is accomplished by the use of subsurface drains. Refer to section 5-4.03 for more details on subsurface drains.

5-3.04.04 EVALUATION OF SALVAGED MATERIALS

It often becomes necessary to use materials salvaged from old pavements in the construction or reconstruction of pavements. In some cases, these materials are used directly. Some materials, however, may have to be recycled or rejuvenated in some manner before they can be used. Material tests should be conducted on the salvaged materials to determine what they may lack, and which virgin materials or other strengthening materials are needed to improve their properties and qualities. The material tests that are typically used for the evaluation are gradation, extraction and Marshall Mix design. And in certain situations freeze-thaw and Abson recovery tests also may be performed.

Invariably, the resulting salvaged and improved materials will have different properties than virgin materials, as a result of the affects of aging, traffic loading, environmental variations, and contamination. Consequently, characteristics determined from the virgin materials cannot be relied upon in a design process involving the use of such salvaged materials. Rather, the properties of the salvaged materials themselves should be determined for use in the design process. The effect of future traffic loadings and environmental variables may also be different on pavements constructed with salvaged materials, in comparison to those with virgin materials. These factors should be taken into account during pavement design involving the use of salvaged materials.

5-3.05 NEW CONSTRUCTION OR MAJOR RECONSTRUCTION

Pavement design for new construction or major reconstruction in Minnesota consists of the selection of the pavement type and the determination of a pavement structure that will adequately carry the projected traffic, while maintaining an acceptable serviceability level throughout the analysis period. The procedures for selecting the appropriate pavement type and for the design of the various types of bituminous and PCC pavements in Minnesota are described in the following sections.

5-3.05.01 ECONOMIC ANALYSIS AND PAVEMENT TYPE SELECTION

Ideally, the type of pavement chosen for a highway construction project should be the product of an analysis of a combination of economic and engineering factors, to determine the pavement that will cost the least over the entire life of the facility, and at the same time meet the traffic, environmental, and safety requirements imposed during its design life. A comparison of the total annual cost per mile of roadway is the approach that is used by the Department. Inherent in the procedure is the need to make assumptions about future maintenance, rehabilitation, materials, and labor needs, and the costs associated with these activities. This is necessary to make an equal and accurate comparison of alternatives. With reasonable choices, comparisons can be made in most instances that adequately meet the established needs of the process.

The pavement selection procedure currently used by the Department was adopted in 1983 after a comprehensive review. A discussion of the procedure followed in the selection of pavement type can be found in Mn/DOT's Engineering Standards publication "Method of Pavement Selection" (January 1983). Essentially this document establishes policy whereby the type of pavement for each new surfacing and/or major reconditioning project shall be selected by a permanently constituted committee. The committee is charged with the responsibility of determining pavement type on the basis of an economic analysis process which involves consideration of service life, interest rates, and equivalent pavement designs. The design alternative with the least annual cost per mile over a 35-year analysis period is generally selected. The 35-year analysis period was chosen based on experience in Minnesota and spans the life of most pavements, including life after major rehabilitation but before major reconstruction is required. The pavement design alternatives considered for comparison for new construction and reconstruction include concrete pavements, bituminous pavements with aggregate base and full depth bituminous pavements. In January 1993, unbonded concrete overlays, which consist of new concrete pavement with an interlayer placed over an existing concrete pavement, were added as another alternative for consideration and comparison in the replacement/rehabilitation strategies for an existing concrete pavement structure. (Refer to Appendix D for policy on unbonded overlays relating to pavement selection.) Findings of the committee are subject to the final approval of the Deputy Commissioner.

New construction and major reconditioning (other than simple overlay for rideability or minor widening in conjunction with simple overlay) which are more

than three-quarters mile in length or other projects as determined by the Committee shall be submitted for surface-type determination. If the Committee feels that a project is not a major project or is an experimental type project, the District can design the roadway based on its own analysis.

Projects which are exempt from the selection process, at the present time, include the following:

- Projects less than three-quarters mile in length;
- Concrete grinding, planing, or texturing for rideability, skid resistance, etc.;
- Improvements that result in less than a 9-ton capacity route;
- Simple overlay;
- Minor widening or channelization; and
- Combination minor widening and simple overlay.

It is the responsibility of the District Engineer to request a surface-type determination based on the previously mentioned design alternatives. The request, using Form 24213, is made to the Committee Chairman and is directed to the Pavement Management Engineer. Upon receiving the request, the Pavement Management Engineer is responsible for requesting and expediting comparative rigid and flexible designs, including first costs and annual costs.

Directive 2-013 defines the detailed information and data which must accompany a surface determination request. As a minimum the following documentation must be included:

- A small scale map and profile showing the project, with intermittent key points indicating stationing and equations in stationing (if any);
- Traffic data, should include the following 20 year data: one-way design lane BESALs, one-way design-lane AADT, one-way design-lane TST, one-way design-lane CESALs, and one-way DHV. The traffic data must be approved and signed by the Traffic Forecast Engineer;
- A description of the project in terms of termini, total mileage, number of lanes, use of any in-place pavement, and exceptions (if any);
- A preliminary soils report delineating the major soil areas and potential borrow sites by textural classification, by AASHTO soil type, and by R value; listing any significant topographic features such as swamps, deep cuts, etc.; and identifying aggregate production sites, including commercial sources, in the area. It should also provide information on subcuts, subdrainage, etc. for projects to be built on existing subgrades;

- Typical sections of the inplace roadway pavement structure including shoulders. The sections should show the widths, thicknesses and material types of the various pavement layers. If there are several different typicals, the limits of each typical should be indicated by the station or reference point;

- Typical geometric cross-sections of the proposed roadway (driving lanes and shoulder widths). If there are several different typicals, the limits of each typical should be indicated by station or reference point; and

- Project layout and profile with soil information plotted.

Any additional pertinent information, such as special features, condition and type of existing roadway surface, and recommendations should also accompany the request.

Two methods are used in making surface-type determinations, depending upon the magnitude of the project. Method 1 requires that detailed comparative designs and cost studies be developed for rigid and flexible alternatives. Determination by Method 2 is restricted to very short projects, normally less than one mile, and permits a simplified means for developing a pavement type study whereby the District Engineer's request can be accompanied directly with comparative designs and a simplified cost estimate based on similar recent work in the same general area.

For reconditioning projects, a Modified Method 1 is used. This method takes into account the requirements for removing and replacing the old pavement, including cracking, milling, and recycling. More than one design may be used on this type of project because various sections may have different needs in order to restore the surface to an acceptable level of performance. See Directive 2-013 for a detailed description of these procedures.

The Selection Committee is composed of the Director, Office of Materials Research & Engineering (Chairman); Assistant Chief Engineer/Director, Engineering Services Division; Assistant Commissioner, Operations Division; Manager, Pavement Engineering Section; and the District Engineer (for subject project). The manager of the Pavement Engineering Section serves as Secretary to the Committee.

The Pavement Management Engineer is responsible for assembly of all pertinent data for the project under consideration, presentation of this material to the Committee, and maintenance of a permanent file for each project. The file shall contain all pertinent data developed, including the findings of the Committee by majority vote, dissenting opinions, and approvals by the Deputy Commissioner, Engineering and Operations Bureau.

5-3.05.02 BITUMINOUS PAVEMENT DESIGN

Bituminous or flexible pavements may be designed as either bituminous pavements with an aggregate base or as full-depth bituminous pavements. In both cases, the structural requirements are determined as a function of the

cumulative traffic load applications expected during the design period of the pavement, the subgrade soil strength, and the strength provided by each component of the completed pavement.

The design procedure consists of the determination of the total thickness of pavement required above the subgrade, as well as the various thicknesses of each of the pavement components, for given traffic and subgrade conditions. The procedures developed for the design of bituminous pavements with aggregate base and full-depth bituminous pavements are based on research conducted by the Department and reported in Mn/DOT's Investigations 183 and 195, respectively.

The structural designs are based upon the cumulative damaging effect of traffic over a 20-year period. Where 20th year projections for design lane AADT volumes equal or exceed 10,000 AADT, a plant-mixed bituminous overlay is assumed at the twelfth year to restore ride quality, the cross section, and skid resistance; and a designed overlay is assumed at the twentieth year to provide a roadway service life of 35 years. For AADTs below 10,000, only an overlay at the twentieth year is assumed.

The following steps are taken in the determination of the thickness and material design for flexible pavements. A request for a traffic analysis should be submitted to the District Traffic Forecaster who in turn will request the assistance or approval from the Traffic Forecast and Analysis Section, Intermodal Programs Division, for the traffic analysis performed or required. The information requested should include one-way design-lane ADT, and the cumulative 18-kip ESALs. These data should be provided for each year of the design life of the roadway. Where high design-lane AADT volumes are forecast, Design Hour Volume (DHV) estimates will lead to a determination of the one-way lane-volume requirements. It is customary to design arterial highways with a sufficient number of lanes to accommodate the forecast DHV for the design year. Occasionally, for multi-lane designs, there may be a significant difference in pavement loading in the opposing directions. In such cases, estimates of the cumulative ESALs in each direction should be provided. The sources of the traffic data and the process of forecasting ESALs are discussed in Section 4-4.0.

As previously described, the District Materials and/or Soils Engineer, in conjunction with the Pavement Design Engineer, must determine a design R value for the subgrade soil. This should be done during the course of the geotechnical survey for the project, and the results provided in a preliminary soils report. This report must be available for the pavement surface-type determination and should include, as a minimum, delineation of the major soils areas and potential borrow sites by texture, by AASHTO soil type, and by R value; a listing of any significant topographic features, such as springs, swamps, and deep cuts; and a listing of the potential aggregate production sites in the area, including commercial sources. The R value selected for design is generally based on the average value minus one standard deviation of the test results obtained on samples taken during the geotechnical investigation.

Table 5-3.1 establishes the sampling frequency guidelines for stabilometer R values as a function of major soil textural classifications. Tables 5-3.2 (a) and

(b) illustrates typical R values associated with AASHTO soil types. These tables are used for small projects where it might be impractical to obtain and test R-value samples.

Table 5-3.1. Stabilometer R-value sampling frequency guidelines.

Major Soil Texture	Recommend Minimum Sampling Rate	Minimum Number of Samples
Sands	0 (assume a value of 70 75)***	0***
Clays, Clay Loams	1 every 2 miles	3**
Sandy Loams (non-plastic to slightly plastic)	3 per mile	5
Silt Loams	3 per mile	5
Silty Clay Loams	3 per mile	5
Plastic Sandy Loams	3 per mile	5
Sandy Clay Loams	3 per mile	5

* Major soil texture refers to a soil texture significant enough in areal extent to economically justify a change in pavement design.

** Given sufficient local experience, this may be reduced to 1 or 2 samples.

*** If % passing #200 exceeds 15%, then sample and select a Design R value in the same manner as for clay, clay loams. This means that a sufficient number of gradation checks of the sand areas will have to be made to determine if Stabilometer tests are required.

NOTE: Samples should be representative of the upper 4 ft. of the proposed road grade as much as possible. In other words, in unbalanced jobs, concentrate on the borrow sources; in balanced jobs, concentrate on the cuts. If practical, resample the embankment after construction.

Table 5-3.2(a). Stabilometer R values by soil type, based on data collected by Mn/DOT through 1974.

AASHTO Soil Type	Textural	Assumed R Value	Comments
A-1-a	Sands, Gravels	75	Excellent confidence in using assumed value.
A-1-b	Sands, Sandy Loams (nonplastic)	70	If percent passing #200 sieve is 15 to 20%, R value may be as low as 25. In such cases, it is highly desirable to obtain laboratory R values.
A-2-4 & A-2-6	Sandy Loams (nonplastic, slightly plastic, or plastic)	30 (70 for LS and LFS)	Loamy Sands and Loamy Fine Sands commonly have R value of 70. Laboratory R values range from 10-80 for the entire A-2 classification. It is highly desirable to obtain laboratory R values for the Sandy Loams. See Table 5-3.1 for sampling frequency.
A-3	Fine Sands	70	Excellent confidence in using assumed value.
A-4	Sandy Loams (plastic) Silt Loams Silty Clay Loams Loams Clay Loams Sandy Clay Loams	20	Laboratory R values range from 10 to 75. It is highly desirable to obtain laboratory R values. See Table 5-3.1 for sampling frequency.
A-6	Clay Loams Clays Silty Clay Loams	20	Laboratory R values commonly occur between 8 and 20.
A-7-5	Clays Silty Clays	12	Data available is limited.
A-7-6	Clays	10	Laboratory R values commonly occur between 6 and 18.

NOTE: In using the above assumed R values for bituminous pavement design, it is essential that the subgrade be constructed of uniform soil at a moisture content and density in accordance with Mn/DOT's Specification 2105 and be capable of passing test rolling, Mn/DOT's Specification 2111. To minimize frost heaving and thaw weakening, it is also essential that finished grade elevation be placed an adequate distance above the water table. This distance should be at least equal to the depth of frost penetration. In the case of silty soils, the distance should be significantly greater.

Table 5-3.2(b) Typical assumed R values for granular subgrades

Subgrade	Assumed R Value
Four feet of select granular	70
One foot of select granular and three feet of granular	60
Four feet of granular	50

Granular - 20% or less passing No. 200 sieve (Mn/DOT Spec. 3149.2A)
Select Granular - 12% or less passing No. 200 sieve (Mn/DOT Spec. 3149.2B)
(AASHTO Soils Types - A-1-a, A-1-b, and A-3)

1. Bituminous Pavement with Aggregate Base. The design of bituminous pavement with aggregate base is based on the concept of the Granular Equivalent (G.E.), which is the product of Mn/DOT's Investigation No. 183 (1969). In this study, the total granular equivalent thickness of a pavement was defined in terms of the subgrade R value and the cumulative 18-kip ESALs required to reduce the Present Serviceability Index (PSI) of a pavement to a terminal value of 2.5. (A PSI of 2.5, by definition, is a surface condition at which trunk highways would require a structural overlay to restore rideability and load support capacity.) The required G.E. for the various pavement materials is determined by assigning granular equivalent values to them on the basis of their contribution to the pavement strength in comparison to the strength offered by a layer of Mn/DOT's Class 5 or 6 base aggregate.

   Figure 5-3.6 is the design chart for bituminous pavements with aggregate base. This chart is used to determine the required G.E., expressed in inches, for the design-lane, cumulative, 18-kip ESALs and subgrade R values. After the required G.E. is determined, it is converted into the appropriate bituminous wearing course, bituminous binder course, bituminous base, and aggregate base thicknesses for the pavement by using Table 5-3.3. These factors are a function of the type of material and its intended use. Once these layer thicknesses have been established, total pavement thickness and layer composition is determined.

Figure 5-3.6. Bituminous pavement design chart (aggregate base).

Figure 5-3.6 Bituminous Pavement Design Chart (Aggregate Base)

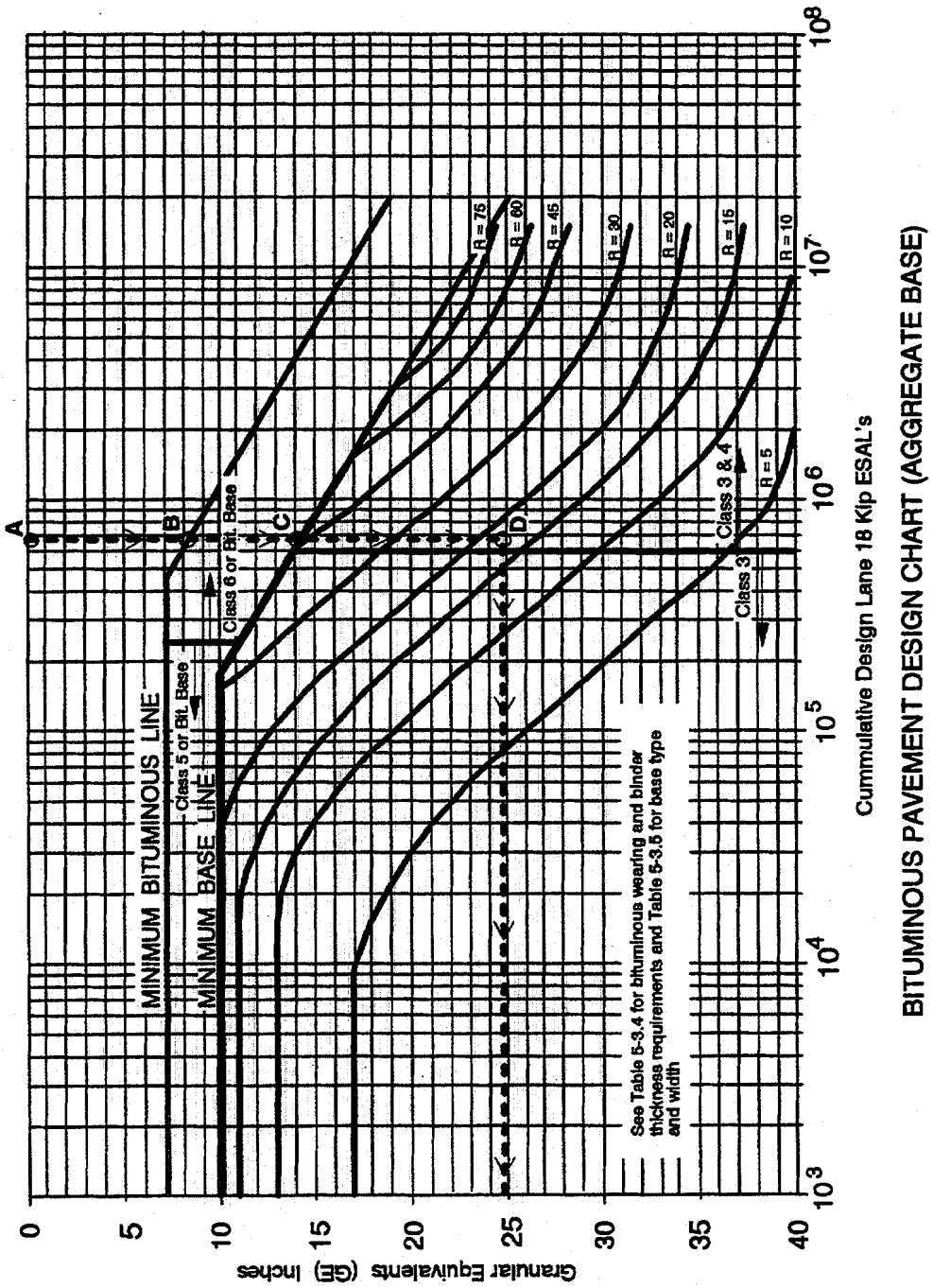


Table 5-3.3. Granular Equivalent (G.E.) factors.

Material	Specification	G.E. Factors
Plant-mix Bituminous Pavement	2331 - Type 41 & 61 (All Courses)	2.25
Plant-mix Bituminous Pavement	2331 - Type 31 (All Courses)	2.00
Road-mix Surface	2321	1.50
Road-mix Base	2321	1.50
Bituminous Treat. Base	(Rich) 2204	1.50
Bituminous Treat. Base	(Lean) 2204	1.25
Aggregate Base	(Cl. 5, Cl. 6) 3138	1.00
Aggregate Base	(Cl. 3, Cl. 4) 3138	0.75
Selected Granular Material		0.50*

* May be used in design when so approved by the Pavement Design Engineer.

NOTE: Where the subgrade consists of granular material, the District Materials and/or Soils Engineer may recommend the treatment of the upper portion of the selected granular material with 150 lb./yd^2, or more, of stabilizing aggregate (Mn/DOT's Specification 3149.2C).

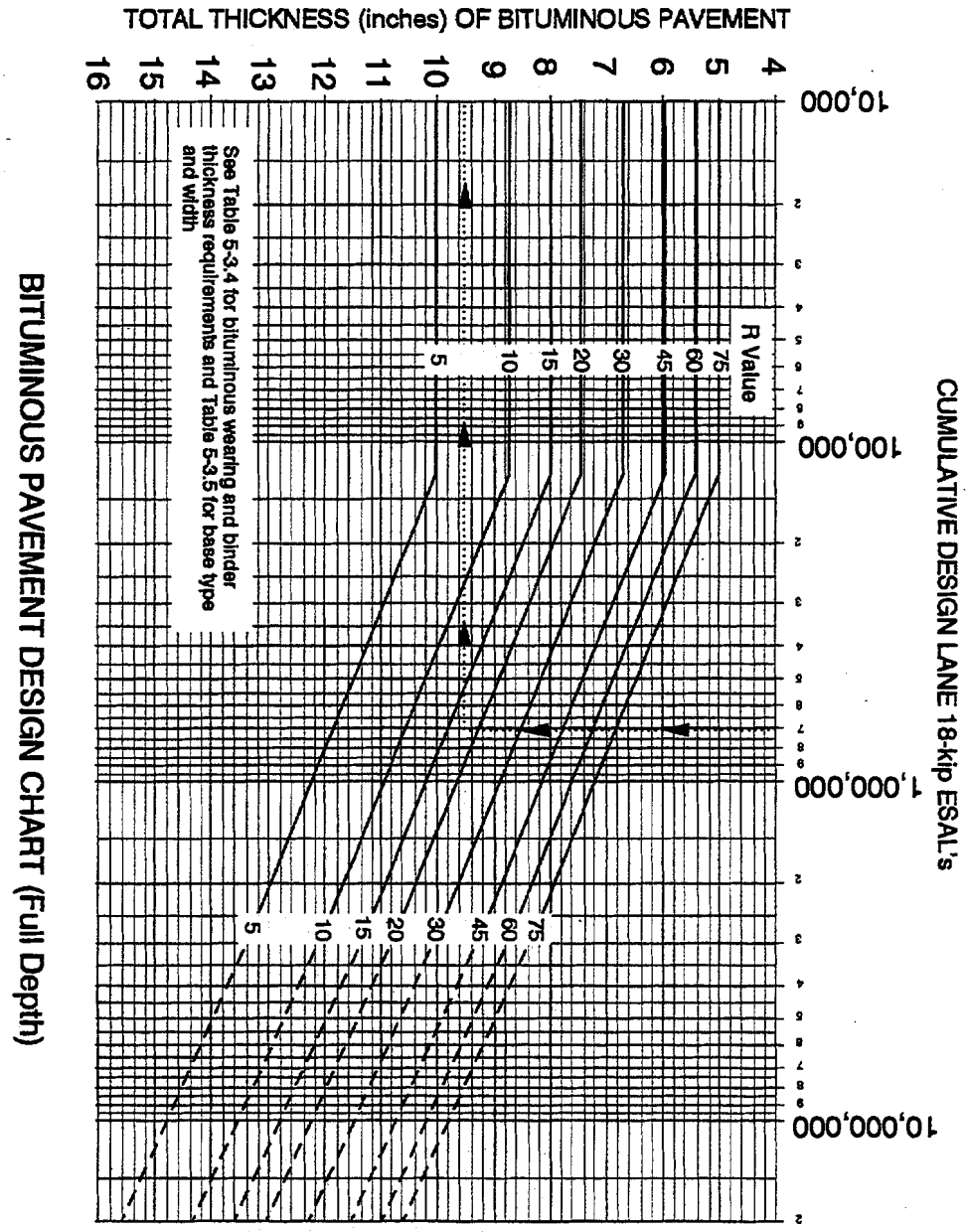
2. Full-depth Bituminous Pavement. Full-depth bituminous pavement is defined as a pavement structure in which bituminous mixtures are employed for all courses above the subgrade or improved subgrade. This type of pavement structure became a standard design alternative for flexible pavements in Minnesota in 1978. The design procedure for this kind of pavement was derived from the procedure for bituminous pavements with aggregate base, which was extended to cover full-depth pavement, based on the results of Mn/DOT's Investigation 195. The design procedure is, therefore, similar to the design of bituminous pavements with aggregate bases. The principal factors evaluated for the structural design are the traffic expected throughout the design period, the subgrade, and other construction materials characteristics; and the environmental factors which may affect pavement behavior or service. The pavement thickness is determined as a function of the 20-year, cumulative, 18-kip ESALs; and the subgrade R value.

As with bituminous pavements with aggregate bases, an estimate of the cumulative 18-kip ESALs expected in the design lane during the design period of the pavement is determined. The procedure is the same as previously described in Section 4-4.0.

The design chart shown in Figure 5-3.7 is used to determine the bituminous pavement thickness for full-depth pavements. Mn/DOT's Investigation 195 is the basis for this design chart. The cumulative 18-kip ESALs and the design R value are used to determine the required

thickness for the given project from this design chart. These are the thicknesses which are equal to the G.E. required to limit the deflection of the pavement plus two standard deviations to the maximum deflection appropriate for the expected traffic loadings. The minimum design thicknesses limit the spring and summer deflections plus two standard deviations to the maximum allowable of 0.075 inches. Thicknesses more than the minimum may be required in some cases to provide surface continuity, leveling, or ease of construction.

Figure 5-3.7. Bituminous pavement design chart (full-depth bituminous).



BITUMINOUS PAVEMENT DESIGN CHART (Full Depth)

3. Structural Thickness Determination

a. Bituminous Pavement with Aggregate Base. The design thickness, in terms of G.E., is determined from Figure 5-3.6. The G.E. is converted into appropriate layer thicknesses using the G.E. factors found in Table 5-3.3.

Some pavement component thicknesses may need to be increased or modified in order to meet the requirements for the wearing and the binder courses, or to conform with desirable construction practices. These requirements are shown in Table 5-3.4 and are based on one-way design-lane ESALs. Table 5-3.5 identifies base type and width requirements as a function of one-way design-lane ESALs. Other options may also be developed to arrive at a feasible design. For example, if an additional one-inch lift of bituminous is needed to meet the minimum requirement for G.E., this could be added to the thickness of the binder course or the base course.

Table 5-3.4. Design Criterion for Mainline Bituminous Pavements.

TABLE 5-3.4 DESIGN CRITERION FOR BITUMINOUS PAVEMENTS

ESALS Design Lane X10^6		BASE	BINDER/LEVEL	WEAR
< 1 and SHOULDERS	SPECIFY →	31BBB50000Y	31BIB50000Y/ 31LVB50000Y	31WEA50000Y
	CONTRACTORS OPTION TO → SUPPLY	TYPE 32 AGG. SIZE A,C	TYPE 32	TYPE 32
1 - 3	SPECIFY →	31BBB50000Y	41BIB50055Y/ 41LVB50055Y	41WEA50055Y
	CONTRACTORS OPTION TO → SUPPLY	TYPE 32 AGG. SIZE A,C	TYPE 42	TYPE 42
NOTE: No Type 31 or Type 32 mixture shall be placed in upper 3 inches.				
3 - 10	SPECIFY →	31BBB50000Y	47BIB50070X/ 47LVB50070X	47WEA75070X
	CONTRACTORS OPTION TO → SUPPLY	TYPE 32 AGG. SIZE A,C	TYPE 48	TYPE 48
NOTES: No Type 31 or Type 32 mixture shall be placed in upper 4 inches. Special Requirements for Type 47/48 mixes apply.				
> 10	SPECIFY →	41BBB50055Y	61BIB75100X/ 61LVB75100X	61WEA75100X
	CONTRACTORS OPTION TO → SUPPLY	TYPE 42 AGG. SIZE A	NONE	NONE
NOTES: No Type 41 or Type 42 mixture shall be placed in upper 4 inches. Class B aggregates allowed in non-wear provided they meet Quality Requirements of Spec 3139.2D2c including the #200 insoluble residue.				

GENERAL NOTES:

1. All WEAR and BINDER courses, including Type 61, shall be 1.5" thick minimum.
2. Aggregate sizes specified and contractors options are to be used unless lift thickness precludes large aggregate or otherwise specified(eg Size B WEAR) Specify Size A when lift thickness is <1.5" or 1" minimum.
3. All recycled mixtures, Types 32,42 & 48, require AC Pen Y=120/150.

MIX DESIGNATION Example: 47BBB50070X

Type	Lift	Agg Size	No. Blows	% +4 Crush	AC Pen
31	*BB*	A	*50*	000	*X=85/100*
41	BI	*B*	75	055	Y=120/150
47	LV	C		*070*	Z=200/300
61	WE			100	
	SH				

Table 5-3.5. Aggregate base type and width design.

20-Year Design Lane ESALs	Base Type	Base Width [e]
Less than 250,000	Bituminous ((2331 (Type 31), 2204, 2321))[a] Class 5 Class 3[b]	24 Feet[f] Full Width Full Width
250,000 to 600,000	Bituminous ((2331 (Type 31), 2204, 2321)[a] Class 6[c] Class 3[b]	24 Feet[f] 30 Feet Full Width
More than 600,000	Bituminous ((2331 (Type 31/41), 2204, 2321)[a] Class 6[c] Class 4[d] Class 3[d]	24 Feet[f] 30 Feet Full Width Full Width

a. District Materials and/or Soils Engineer in conjunction with the Pavement Design Engineer may substitute Mn/DOT's Specification 2331, 2321, or 2204 for all or a portion of Class 5 and/or Class 6.

b. District Materials and/or Soils Engineer in conjunction with the Pavement Design Engineer may substitute the use of Class 4 in place of a portion of Class 3.

c. District Materials and/or Soils Engineer in conjunction with the Pavement Design Engineer may substitute Class 5 for Class 6.

d. When Class 3 and 4 are required, the minimum thickness of Class 4 over Class 3 should be 6 in. unless otherwise approved by the Pavement Design Engineer. If less than 6 in. of Class 3 and 4 are required, use all Class 4 unless otherwise approved by the Pavement Design Engineer.

e. On urban sections, all bases are full width.

f. If the total thickness of the bituminous base exceeds 3 inches, use width of 27 ft.

In cases where the original rigid pavement section is to be widened, additional thickness is required. Because deflections are higher near the free edge (widened section), which results in shorter pavement life, it is necessary to multiply the total design thickness by a factor of 1.2. This increase will then result in a widened section capable of carrying the projected design traffic.

Based on an elastic layer analysis, additional thickness may be required at specific locations for flexible pavements (both aggregate base and full-depth designs) under certain conditions. For sections which experience low speed traffic and/or high shear stresses due to stopping and turning movements, such as urban freeways, bus stop, intersections, and weight stations, etc., the additional recommended thickness is as follows:

Predicted Percent of Slow Traffic	Increased Thickness
1% - 3%	0.5 in.
≥ 3%	1 in.

Special mix designs should also be considered for such locations.

b. Full-depth Bituminous Pavement. For full-depth pavement, the bituminous mixture for the total thickness may include a combination of Type 31 Base, Binder, or Wearing Course; Type 41 Base, Binder, and/or Wearing Course; and Type 61 Binder and/or Wearing Course. Table 5-3.4 gives the wearing, binder and base course thicknesses and mixture types as a function of the one-way design-lane ESALs. The difference between the total pavement thickness determined from the design chart in Figure 5-3.7 and the combined thicknesses of the wearing and binder courses is the bituminous base thickness required for the pavement. It should be noted that since the total thickness obtained from the design chart is bituminous thickness, no conversion is necessary, as in the case of bituminous pavement with aggregate bases.

Full-depth bituminous pavement structures which are designed to be constructed on untreated Granular or Select Granular subgrades, should have two inches of Class 5 (Mn/DOT's Specification 2211) Aggregate Base placed on the completed subgrade or have a minimum of 150 pounds per square yard of Stabilizing Aggregate (Mn/DOT's Specification 2105) incorporated into the upper portion of the subgrade to establish a stable paving platform. The type of treatment to be used on each project should be determined by the District Soils and/or Materials Engineer and should be specified in the Design Recommendation Report. The placement of these materials should extend a minimum of 18 inches beyond each side of the bituminous base width. Any aggregate materials used in this manner should not be included in the overall pavement structural thickness determination.

Again, as discussed above, additional thickness may be required for low-speed areas. Also, the guidelines previously given for increasing the pavement thickness of bituminous pavements with

aggregate base at areas of high shear forces, such as bus stops, intersections, and weigh stations, also apply to full-depth pavements.

It should be noted that because of strip-loading conditions where "full-depth" pavement is used to widen existing concrete pavement, all chart-derived thickness requirements are to be increased by a factor of 1.2 for the widened section.

4. Design Examples.

a. Bituminous Pavement With Aggregate Base. Design of bituminous pavement with aggregate base, which is based on Figure 5-3.6, requires the design-lane ESALs and the design R value for the embankment soils. The design procedure is as follows:

(1) Using Figure 5-3.6, the total structural requirement (in terms of G.E.) for a specific ESAL traffic value and embankment R value is determined by extending a line from the design lane ESALs value to intercept the design R-value curve. From this intercept, extend the line to the G.E. ordinate line to locate the G.E. Requirement to structurally satisfy these conditions. Note that in the process of intersecting the R-value curve, the "traffic" line has intersected two minimum requirement lines. These are the minimum bituminous material and minimum base material for which the requirements are established in terms of G.E. Note that bituminous base or Class 6 aggregate base may be used to satisfy minimum base requirements.

(2) Subtracting the sum of minimum bituminous G.E. and minimum base G.E. from the total G.E. requirement over embankment defines the additional requirement (over the two minimums) for structural sufficiency.

(3) The G.E.s thus determined - minimum bituminous, minimum base, and "additional requirement" (if any) - can be converted into thickness of specific construction materials by dividing the G.E. value by the G.E. factor in Table 5-3.3. to arrive at the required thickness of each material course.

However, it is necessary to select materials and apportion thicknesses with regard to policy, economy, and the dictates of construction necessity. The thickness of materials selected must always possess a combined G.E. value at least equal to the required G.E. over embankment. Once the total G.E. requirement is determined, the respective layer thicknesses and base

course materials and widths should be determined in accordance with the following tables:

(a) Table 5-3.4, provides the bituminous thickness requirements and mixture types for the wearing, binder, and base courses based on design lane ESALs.

(b) Table 5-3.5, provides base course material types and width corresponding with design lane ESALs. Note to this table should be carefully examined concerning permissible substitutes of aggregate types.

In addition to the use of the above tables, the designer must be aware of minimum and maximum practical lift or course thicknesses for the various materials for reason of both construction limitations and costs.

b. Full-depth Bituminous Pavement. Design of full-depth bituminous pavement, which is based on Figure 5-3.7, requires the design lane ESALs and the design R value for the embankment soils. The design procedure is as follows:

(1) Using figure 5-3.7, the total bituminous structural thickness in inches for the design lane ESALs and R value is determined.

(2) As with the bituminous pavement with aggregate base, policy and procedural considerations must be followed.

(3) Table 5-3.4, provides the thickness requirements for the wearing and binder course mixtures and the mixture types for the various courses based on design lane ESALs. The difference between the combined thickness of the wearing and binder courses and total structural thickness requirement represents the required thickness of bituminous base.

(4) Width of the base should be 28 feet for two-lane roads, 14 feet on each side of centerline. For multi-lane roads, extend top of Type 31/41 base three feet beyond the edge of the outside lane and one foot beyond the edge of the inside lane. For urban design, the base will be full width and will extend under the curb and gutter as shown in the shoulder structural design details.

5. Example Problems. Two example design problems are provided to demonstrate the use of the bituminous pavement design charts for aggregate base and full-depth pavement.

a. Example 1. Bituminous pavement with aggregate base.

Given: 20-year design-lane ESALs = 700,000
Design R value = 18

Determine the thickness of each layer of the pavement structure as follows:

(1) In order to determine the minimum required wearing and binder course for the project, refer to Table 5-3.4. and select the thickness and mixture type based on the design-lane ESAL.

For design-lane ESALs of 700,000
Wearing course = 1-1/2 in. 2331 Type 31 (31 WEA 50000Y)
Binder = 1-1/2 in. 2331 Type 31 (31 BIB 50000Y)
Total wearing and binder G.E. = 6.0 in.

(2) To determine the granular equivalent, enter the chart in Figure 5-3.6 at the design-lane ESAL value of 700,000 and move down to the minimum bituminous line and read the corresponding G.E.

G.E. = 8.4 in. (point B)

(3) Subtract appropriate wearing and binder G.E. value (6.0 in.) from G.E. determined in step 2. (8.4 in.). This difference (2.4 in.) represents the minimum bituminous base (Type 31) G.E. required. This minimum bituminous base G.E. is converted to actual design thickness by dividing by the G.E. factor for 2331 Type 31 Plant-Mix Base (2.00) from Table 5-3.3.

2.4 in. G.E. ÷ 2.00 = 1.2 in. Type 31 Base

(4) Move to Minimum Base Line (point C). Read corresponding G.E. value by moving left from point C.

Point C = 14.4 in. G.E.

Determine G.E. of increment BC.

BC = 14.4 in. - 8.4 in. = 6.0 in. G.E.

This represents the G.E. of Class 6 aggregate base, or bituminous base required.

If aggregate base is to be used, see Table 5-3.5 for determination of type(s) and width(s) dependent upon ESALs.

The G.E. thickness is converted to actual design thickness by dividing by the G.E. factor for Class 6 Aggregate Base (1.00) from Table 5-3.3.

6.0 in. G. E. ÷ 1.00 = 6.0 in. Class 6

Depending on economic factors and availability of materials, all or part of this portion (BC) of the base can be designed as bituminous base. This G.E. quantity would, in the case where all of BC is designed as bituminous base, be added to that determined in step 3. (e.g., 6.0 in. G.E. + 2.4 in. G.E. = 8.4 in. (G.E.). Converting to an actual thickness would yield 8.4 in. G.E. ÷ 2.00 = 4.2 in. 2331 Type 31 Base.

See Table 5-3.5. for determination of bituminous base width dependent on ESALs.

(5) Move to appropriate R-value intercept of 18 (point D). Read corresponding G.E. value by moving from point D to the left edge.

(6) Determine G.E. of increment CD.

CD = 25.0 in.- 14.4 in.= 10.6 in.G.E.

This represents the G.E. of Class 3 or Classes 3 and 4 aggregate base required.

This G.E. thickness is converted to actual design thickness by dividing by the G.E. factor for Class 3 and Class 4 Aggregate Base (0.75) from Table 5.3.3.

10.6 in. G.E. ÷ 0.75 = 14.1 in.Class 3 or Classes 3 and 4.

Because aggregate base is to be used, see Table 5.3.5 for determination of type, distribution, and width, dependent upon ESALs.

Because ESALs = 700,000, use Classes 3 and 4 aggregate base and distribute the 14.1 in. in the following manner: 6 in. Class 4 over 8.1 in. Class 3, both full width.

(7) Based on the above analysis, the completed design thickness and layer types in the pavement structure are:

Thickness	Specification/Mixture Type
1.5 in.	2331 Type 31 wearing course (31 WEA 50000Y)
1.5 in.	2331 Type 31 binder (31 BIB 50000Y)
1.2 in.	2331 Type 31 base (31 BBB 50000Y)
6.0 in.	2221 Class 6 aggregate
6.0 in.	2221 Class 4 aggregate
8.1 in.	2221 Class 3 aggregate

Some of the layer thicknesses above would not be acceptable, as listed, for design purposes, because they would not always conform to desirable construction practices. For example, the 1.2 in. of Type 31 base in the granular base design would not be a separate item in a design layout, because it would be undesirable to construct only a 1.2-in. thick base bituminous lift. Therefore, this design thickness should be rounded to 1.5 inches and included with either the Type 31 binder or Type 31 base. Another change would be to round the aggregate base to the nearest 1.0 in. and the bituminous base to the nearest 0.5 in. The 8.1 in. of Class 3 in the granular base design would then become 8.0 in.

Based on these revisions to the basic design thickness determination, the following designs would be acceptable:

Option 1	Actual G. E. (in.)
1.5 in. 2331 Type 31 wear (31 WEA 50000Y)	3.0
3.0 in. 2331 Type 31 binder (31 BIB 50000Y)	6.0
6 in. Class 6	6.0
6 in. Class 4	4.5
8 in. Class 3	6.0
	25.5 > 25.0 req'd ok

Option 2		Actual G. E.
1.5 in. 2331 Type 31 Wear (31 WEA 50000Y)	3.0	
1.5 in. 2331 Type 31 Binder (31 BIB 50000Y)	3.0	
4.5 in. 2331 Type 31 base (31 BBB 50000Y)		9.0
6 in. Class 4		4.5
8 in. Class 3		6.0
		25.5 > 25.0 req'd ok

b. Example 2. Full-depth Bituminous Pavement.

Using the same input information as in Example 1, determine the total design thickness for the roadway.

(1) Refer to Table 5-3.4 to determine the wearing and binder course bituminous mixture types and layer thicknesses. For one-way design-lane ESALs of 700,000:

Wearing course = 1.5 in. 2331 Type 31 (31 WEA 50000Y)
Binder = 1.5 in. 2331 Type 31 (31 BIB 50000Y)

Total = 3 in.

(2) Using Figure 5-3.7, enter chart with design-lane ESALs of 700,000 and move to an R value of 18. Read corresponding "total design thickness" value from design chart.

Total design thickness = 9.5 in.

(3) Subtract appropriate wearing and binder thickness (3 in.) from total design thickness (9.5 in.). This difference represents the required bituminous base course thickness.

9.5 in. - 3.0 in. = 6.5 in. base

(4) Again, refer to Table 5-3.4 for the appropriate bituminous mixture type to be used in production of the base course. In this case, the base course would be constructed with 2331 Type 31 mixture. (31BBB 50000Y)

6. Typical Cross Sections. Refer to Chapter 7 of Mn/DOT's Road Design Manual for typical sections for bituminous pavement with aggregate base and full-depth bituminous designs.

7. Shoulder Designs - Refer to Section 5-3.08

8. Ramp and Loop Designs - Refer to Section 5-3.09

5-3.05.03 PORTLAND CEMENT CONCRETE PAVEMENT DESIGN

Portland cement concrete (PCC) or rigid pavements may consist of a slab placed on a base over a prepared subgrade, or an existing PCC pavement (unbonded and bonded overlay) or on an existing flexible pavement ("white topping"). The design procedure presented in this section is for PCC pavement over a prepared subgrade. The procedure primarily consists of the determination of the thicknesses of the slab and base. The design procedure for an unbonded overlay is described in Section 5-3.06.04. The Pavement Design Unit (Office of Materials Research & Engineering) should be contacted relative to bonded overlay and white topping design procedures.

Depending on the type of PCC pavement, the design procedure may also include the design of joints and the required reinforcement steel.

The design considerations that are essential to the satisfactory long-term performance of a rigid pavement structure are:

(a) reasonably uniform support for the pavement;

(b) the elimination of pumping by use of dense-graded aggregate bases and/or treated or untreated open-graded bases with an edge drain systems

(c) adequate joint design

(d) and a thickness that will keep repeated load stress within safe limits.

The overall objective in concrete pavement design is to have a pavement which provides satisfactory performance over its design life of 35 years at the least annual cost. In the past, Minnesota designed and constructed, two types of PCC pavement. These were jointed plain non-reinforced concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP). At the present time, the Department is constructing only JPCP. The Department's design procedures for lane width, slab thickness, joints and reinforcement for JPCP and JRCP are presented in this section.

1. Pavement Lane Width Design (Protected vs. Non-protected Edge Design). Pavements in Minnesota are normally designed with widened sections unless tied concrete shoulders are used. Both of these designs are referred to as a protected edge design. The protected edge design for a widened two-lane roadway design has a width of 27 feet (with each lane 13.5 feet). For a divided roadway the lane widths are 13 and 14 feet for the inside and outside lane, respectively. The interior lanes of a three or more lane roadway are 12 feet wide. If the pavement design includes tied concrete shoulders, the travel lanes are each 12 feet wide. An urban design with tied curb and gutter (B 624, etc.) or integrant curb is also considered a protected edge design, in most cases.

The protected edge design (widened pavement section) has two advantages over the non-protected edge design; namely, the pavement has substantially lower edge stresses and runoff water is drained further away from the traffic wheel path. Water accumulation in the joint between a concrete pavement and a bituminous shoulder has been recognized as a major factor in poor pavement performance. Concrete shoulders and lane widenings have the potential for practically alleviating this drainage problem.

In the protected edge design (widened section/tied concrete shoulders), rumble strips are provided. The strips are constructed on the widened section or on the tied concrete shoulder. The use of these strips significantly lessens the number of vehicles encroaching out onto the

widened section or the tied concrete shoulder, thereby, minimizing edge stresses. The paint striping is carried 12 feet from centerline.

Concrete pavements which are designed with 12-foot lanes and bituminous shoulders or untied concrete shoulders are referred to as a non-protected edge design.

The protected edge design is the standard design for use in Minnesota.

2. Slab Thickness Design. Rigid pavement thickness design in Minnesota is based on the AASHTO Interim Guide procedure, modified to take advantage of experience with concrete pavements in Minnesota. Slab thickness is determined on the basis of the cumulative ESALs and the modulus of subgrade reaction. A comprehensive discussion on the determination of the ESALs for concrete pavement design is presented in Section 4-4.0. The modulus of subgrade reaction and the method for its determination are discussed extensively in Section 5-3.04.01. As indicated, the modulus of subgrade reaction is not easily determined; however, it has been correlated to the R value by the Department.

The thickness design is based on the following design Equation 5-3.6, which was developed from the results of the AASHO Road Test. This design equation expresses the relationship between the number of load applications of the standard, 18-kip, single-axle load required to bring a pavement to a given serviceability level.

$$\log W_t = 7.35 \log (D + 1) - 0.06 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32 p_t) \log \left[\left(\frac{S_c}{215.63 J} \right) \left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{(E/k)^{0.25}}} \right) \right] \qquad \text{Eq. 5-3.6}$$

where

W_t = number of applications of 18-kip, equivalent, single-axle loads required to reduce serviceability to p_t;
D = slab thickness, in.;
G_t = the logarithm of the ratio of the loss in serviceability at time t to the potential loss taken to a point where p_t = 1.5, or log [(4.5 - p_t) / (4.5 - 1.5)];
p_t = serviceability at end of time t (terminal serviceability);
S_c = modulus of rupture, psi (third-point loading);
J = load transfer coefficient (assumed as J = 3.2);
E = Young's modulus of elasticity of concrete, psi; and
k = modulus of subgrade reaction, psi/in.

For the projected number of 18-kip ESALs to the selected terminal serviceability (p_t) this equation is solved to determine the pavement thickness required, given the physical properties of the pavement and foundation. The Department currently has a computer program called PAVE which solves this equation in an iterative process. In addition to the PAVE program, the Department uses DNPS 86 and DARWIN software programs to analyze/evaluate the effects that changes in various parameters (i.e., subgrade resilient modulus, pavement width, drainage, etc.) may have on the pavement thickness. (The DNPS 86 and DARWIN 2.1 programs are based on the 1986 and 1993 AASHTO Design Guides for Pavement Structures, respectively.)

The basic input parameters required in the use of the PAVE program or Eq. 5-3.6 are as follows:

a. Serviceability Factor. A Serviceability Factor (p_t) of 2.5 should be used in the program/design formula for both urban and rural designs.

b. Modulus of Subgrade Reaction. The mean modulus of subgrade reaction (k) should be the gross value from a 30-inch diameter plate bearing test.

   As indicated before, field plate bearing tests for the modulus of subgrade reaction are rarely performed by any agency. The relationship given in Equation 5-3.2 in Section 5-3.04.01, is recommended for converting R values to the equivalent k value. This equation has been incorporated into the computer program PAVE. No increase in the k value is recommended in cases where non-stabilized/stabilized base layers are used.

c. Modulus of Rupture. The mean modulus of rupture (flexural strength) used in the analysis should be from the 28-day, third-point loading, beam tests. Presently, Mn/DOT's test procedures for concrete beams require that the test beams be broken at the third points. The available 28-day flexural strength data shows that the average third-point loading strength value is approximately 675 pounds per square inch.

   A safety factor of 1.33 is applied to the modulus of rupture to obtain the design working stress of 500 pounds per square inch used in the PAVE program or Equation 5-3.6 ((675 psi) ÷ 1.33 = 507 psi).

d. Young's Modulus of Elasticity. The static modulus of elasticity test (ASTM C469) is rarely conducted for pavement design. A value of 4,200,000 pounds per square inch is recommended at the present time for use in the concrete pavement design equations.

e. Cumulative ESALs. The 35-year design lane ESALs, previously determined in Section 4-4.0, are modified as follows:

- An adjustment factor of 0.93 is applied to the cumulative ESALs to account for the difference in severity and duration of winter conditions between Ottawa, Illinois (AASHTO Road Test) and the State of Minnesota;

- Non-protected edge design is based on 35-year design-lane ESALs and a load transfer coefficient (J) of 3.2;

- Protected edge design is based on 35-year design-lane ESALs multiplied by 0.5 and a J = 3.2 or the 35-year design-lane ESALs and a J = 2.6.

Following are examples for non-protected and protected edge designs:

For non-protected designs, ESALs x 0.93 with J = 3.2. For protected edge designs, ESALs x 0.93 x 0.5 with J = 3.2, or ESALs x 0.93 with J = 2.6.

(If only the 20-year design lane ESALs have been provided to the designer, it is possible to estimate the 35-year design lane ESALs by multiplying the 20-year design lane ESALs by a factor of 2.)

On a multi-lane roadway the District Soils Engineer and Materials Engineer should consult with the Pavement Design Unit (Office of Materials Research & Engineering) in determining whether a protected edge or non-protected edge design should be used. The determination of which edge design concept to use should be based on the manner in which the various lanes will be tied together.

The minimum pavement thickness to be used for State Highways is seven inches. This minimum thickness requirement does not apply to county or city roads.

A detailed explanation of the various factors/parameters and design concepts can be found in the report titled "Review of Minnesota's Concrete Pavement Design by the Minnesota Department of Transportation Concrete Task Force," dated March 4, 1985.

3. Base Design. The base normally consists of one or more compacted layers of aggregate placed between the subgrade and the slab. This provides for uniform stable support of the slab; increases the modulus of subgrade reaction; prevents pumping of fine-grained soils at slab joints, cracks, and edges; and reduces cracking and faulting. In addition to these attributes, if a permeable base is provided it will assist in

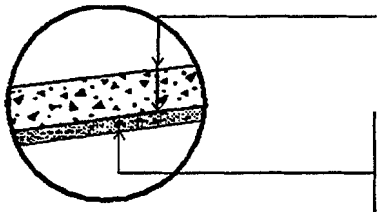
reducing or eliminating moisture related pavement damage that may be associated in certain situations with dense graded bases, non-granular subgrade soils and non-durable concrete aggregates.

It is essential that the subgrade be constructed of uniform soil at a moisture content and density in accordance with Mn/DOT's Standard Specification 2105.

Typical base thicknesses and classes of material for different subgrade soil classifications are illustrated in Figure 5-3.8. The base normally extends 3 feet beyond the outside edges of the concrete pavement structure, primarily to accommodate construction operations. Refer to Chapter 7 of the Road Design Manual for specific details.

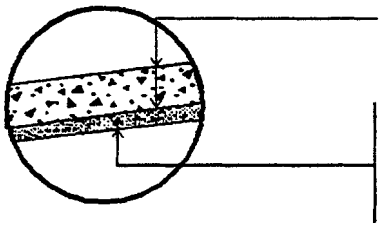
Concrete pavement performance/serviceability, as related to functional and structural distresses, is directly related to both subgrade and surface-moisture infiltration. It is imperative that drainage, both from the standpoint of preventing moisture infiltration and the prompt removal of any infiltration from the pavement system, be given high consideration in all aspects of the pavement design. Section 5-4.0 discusses the various drainage aspects.

Figure 5-3.8. Rigid pavement base type and thickness design.



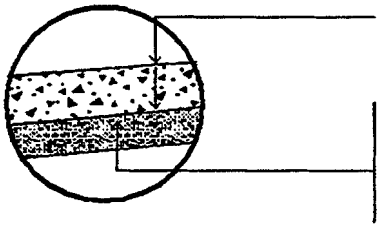
Concrete Pavement

INSET R-1
(For Soil Classes A1a, A1b, Stable A3 and A2)
3" Aggregate Base - Class 5 - Spec. 2211 or stabilizing Aggregate - Spec. 2105 incorporated in subgrade at a rate of 150 lb./sq. yd. or as recommended by the District Materials and/or Soils Engineer in conjunction with Pavement Design and Concrete Units (Office of Materials, Research and Engineering).



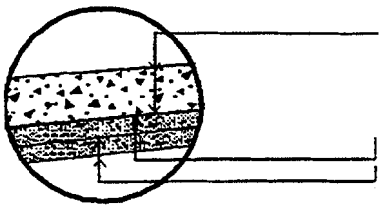
Concrete Pavement

INSET R-2
(For Soil Classes Unstable A3)
3" Aggregate Base - Class 5 - Spec. 2211 and stabilizing Aggregate - Spec. 2105 incorporated in subgrade at a rate of 150 lb./sq. yd. or as recommended by the District Materials and/or Soils Engineer in conjunction with Pavement Design and Concrete Units (Office of Materials, Research and Engineering).



Concrete Pavement

INSET R-3
(For Soil Classes A4, A6, and A7)
5" Aggregate Base - Class 5 - Spec. 2211 or may be changed to 2" aggregate base - Class 5 and 3" aggregate base - Class 3 when recommended by the District Materials and/or Soils Engineer in conjunction with Pavement Design and Concrete Units (Office of Materials, Research and Engineering).



Concrete Pavement

INSET R-4
See Design Guidelines for use of PAB.
4" Permeable Aggregate Base (PAB)
3" Aggregate Base - Class 5 - Spec. 2211 place on Subgrade.

4. Joint Design. Joints are placed in rigid pavements for the purpose of controlling transverse and longitudinal cracking. This process relieves stresses in the pavement resulting from shrinkage of the concrete and differential temperature and moisture conditions between the top and bottom of the slab. Stresses caused by vehicle loads superimposed on these stresses can increase their detrimental effect on concrete pavements and must also be taken into account in joint design. Note that all joint layouts should be reviewed by the Concrete Unit (Office of Materials Research & Engineering).

a. Transverse Joints. Basically, three types of transverse joints are used in concrete pavements. Properly designed transverse contraction joints will control cracking in the pavement by relieving the stresses mentioned above. Transverse expansion joints give room for slab movements, especially at locations where the pavement must interface with fixed structures or other pavements. Transverse construction joints also divide the pavement into suitable increments for construction. As much as possible, construction joints are eliminated by making them coincide with the other joint types.

Current Department policy identifies three types of transverse joint patterns for mainline rigid pavement design. The first is a 27-foot, skewed, joint spacing for reinforced concrete mainline pavement (JRCP). The second is a 15-foot effective, skewed, joint spacing for non-reinforced concrete mainline pavement (JPCP). The 15-foot effective joint spacing indicates a pattern of spacings of 13, 16, 14, and 17 feet for four consecutive panels in a repeated pattern. This type of joint pattern is used for pavements that are less than 10.5 inches thick. The third is a 20-foot, skewed joint spacing for non-reinforced concrete mainline pavement (JPCP) that is 10.5 inches or greater in thickness.

It is the department's policy to skew transverse construction joints; except in urban design situations with curb and gutter, the joints are generally constructed perpendicular to the roadway centerline. Where skewed joints are provided the joints are skewed at 2 to 12 with the obtuse angle at the outside pavement edge ahead of the joint in the direction of traffic. The skew is designed in this direction because this corner receives the greatest impact from the sudden impact of load. Skewed joints have the advantage of preventing the application of two wheel loads to the joint at the same time. This results is reduced deflections, stresses and vehicle impact reaction at joint crossings. Thus skews joints may reduce load associated distresses.

As previously indicated, short joint spacings are randomly placed in order to provide improved pavement ride quality.

Details of the skewed joint designs are provided in Mn/DOT's Standard Plan Sheet 5-297.217 for the 15-foot effective and 20-foot joint spacings and in Mn/DOT's Standard Plan Sheet 5-297.216 for the 27-foot joint spacing. The 15-foot effective joint spacing design is also used for ramps and loops. The details are provided in Mn/DOT's Standard Plan Sheet 5-297.219.

In urban design situations (curb and gutter section) the transverse contraction joints should be a non-skewed design. The curb and gutter joints and the pavement joints should match.

Contraction joints for mainline roadway should be specified in accordance with the details in Mn/DOT's Standard Plans Sheet 5-297.221.

The main purpose of an expansion joint is to prevent the development of compressive stresses, due to volume change, from damaging the pavement slab and the transmission of excessive forces to adjacent structures. Mn/DOT's Standard Plan Sheet 5-297.221 provides details for the various types of expansion joints used by the Department. The expansion joints are normally used for rigid pavement surfaces or bases adjacent to fixed structures, curbs and walls, crossroads, and crossovers. In addition, unusual conditions, such as cold weather construction or the use of materials with a high coefficient of expansion may require special consideration.

b. Dowels. Dowels are required in all the transverse joints for mainline roads, loops, and ramps in Minnesota, to enhance load transfer and to guard against joint faulting. The dimensions and spacing of the required dowel bars depends on the support provided by the pavement foundation, the concrete aggregate type, and the projected traffic. It is recommended that the dowel bars be 15 inches in length and spaced at 12 inches. A rule of thumb is to use a dowel bar diameter equal to one-eighth the thickness of the pavement, e.g., an eight-inch thick pavement would require a one-inch diameter dowel bar. However, dowel bars should be sized in accordance with Table 5-3.6(a). It is recommended that corrosion-resistant or epoxy-coated dowels be used.

Table 5-3.6(a). Rigid pavement dowel bar sizes.

Pavement Thickness(ins.)	Dowel Bar Diameter (in) - 15 inches long
7	1.0
8	1.0
8.5	1.0
9	1.25
10	1.25
10.5	1.25
11	1.50
12	1.50
12.5	1.50
13	1.75
14	1.75

c. Longitudinal Joints. Longitudinal joints identified in Mn/DOT's Standard Plan Sheet 5-297.221 are used to prevent the formation of irregular longitudinal cracks in pavements. These may be butt joints, or mechanically formed or sawed grooves. Keyed joints are not recommended without steel tie bars. Adjacent lanes and tapers should be kept from separating and faulting by using steel tie bars in accordance with Table 5-3.6(b).

Table 5-3.6(b). Rigid pavement tie bar sizes.

Panel Length	Pav't Thickness (in.)	Spacing (in.)	Length (in.)	Size No.
15 foot effective	10 or less	30	30	4
15 foot effective	Over 10	30	36	5
20 foot	10.5 or greater	30	36	5
27 foot	10 or less	30	30	4
	Over 10	30	36	5

In most cases, the width of a single paving pass is determined by the capabilities of the paving equipment, which can pave widths of up to about 27 feet. When pavement widths exceed 27 feet and a tied longitudinal joint is required, an option for longitudinal joints should be given, such as L1T or L2KT (Mn/DOT's Standard Plan Sheet 5-297.221). With this option, a contractor may pave in one or two stages, depending upon available equipment.

Mn/DOT's standard design practice is to tie two lanes together. These lanes can be either 12 feet wide for a total width of 24 feet, or a 13- and a 14-foot wide lane for a total width of 27 feet. A

third 12-foot wide lane or a concrete shoulder may also be tied for a total width of 34 feet (two 12-foot wide lanes and a 10-foot wide shoulder). The normal configuration is a 13-foot wide inside lane, a 12-foot wide center lane, and a 14-foot wide outside lane for a total width of 39 feet. The contractor should be given the option of using an L1T or L2KT for the longitudinal joints in these configurations. If three lanes are tied together, supplemental steel is normally placed in only the center lane.

If the geometrics of the pavement design should require that four or more lanes, including turn lanes, loops, and ramp connections, be adjacent to each other, then the Concrete Office should be consulted for recommendations. The general rule of thumb is that no more than three lanes are tied together for any extended length. If four lanes are adjacent to one another an L3 joint would separate the roadway with each adjacent pair of lanes tied with an L1T joint. If five lanes are necessary, then the outside three lanes and the inside two lanes are tied with L1T or L2KT joints and these two sections are separated by an L1 or L3 joint. The general rule is that if there are three lanes (or two lanes and one concrete shoulder), then they all should be tied together; if there are four lanes, an L3 joint in the center should be used; and if there are more than four lanes, then the outside three lanes should always be tied together and the L3 joint should be kept, if possible, at least two lanes away from a pavement edge to prevent lane separation in this joint. Also refer to Item No. 5, Reinforcement Design.

It is recommended that mechanically formed or sawed longitudinal joints should be used wherever necessary to ensure that the distance between adjacent longitudinal joints does not exceed 14 feet, to forestall the formation of irregular longitudinal cracks. The use of longitudinal joint inserts is not allowed.

d. Joint Sealer. The Department recommends the use of an approved silicone joint sealer for all transverse contraction joints in the 15-foot effective and 20-foot (JPCP) pavements and neoprene joint sealer in the 27-foot (JRCP) pavement. All longitudinal joints should be filled with a hot-pour elastic joint sealer (Mn/DOT's Specification 3723).

5. Reinforcement Design. Concrete pavements have a tendency to crack due to material shrinkage. To minimize and provide for an orderly arrangement of the cracks that may form, transverse and longitudinal joints are provided. If the joints are properly designed and spaced, cracking of the panels at places other than the joint will be held to a minimum. The purpose of distributed steel reinforcement - which is not to prevent cracking - is to hold tightly closed any such cracks that may form and to maintain the pavement as an integral structural unit.

When panel lengths are such that curling stresses are low and joint and crack openings are not enough to cause a loss of aggregate interlock, it is not necessary to use steel reinforcement. In current Mn/DOT practice, no wire reinforcement is required for rigid pavements with 15-foot effective or 20-foot, skewed, joint spacings (JPCP), except where the pavement crosses a culvert. In such cases, bar reinforcement should be provided in accordance with Mn/DOT's Standard Plate Number 1070.

For rigid pavements with 27-foot panels, wire reinforcement is required for all thicknesses. The reinforcement requirements for JRCP with 27-foot, skewed, joint spacing are shown in Mn/DOT's Standard Plan 5-297.216. A summary of reinforcement requirements is given in Table 5-3.7.

For 27-foot reinforced panels over culverts, either bar reinforcement or wire mesh fabric can be used as shown in Standard Plate 1070.

Table 5-3.7. Rigid pavement reinforcement requirements for 27-foot panel lengths.

Pavement Thickness	Left or Right Lane Mesh
$t \leq 8$ in.	12 X 12 D5.8 X W4.0 or 12 X 12 W6.2 X W4.0
8 in. $< t \leq 9$ in.	12 X 12 D6.5 X W4.0 or 12 X 12 W7.0 X W4.0
9 in. $< t \leq 10$ in.	12 X 12 D7.2 X W4.0 or 12 X 12 W7.8 X W4.0
10 in. $< t \leq 11$ in.	12 X 12 D7.9 X W4.0 or 12 X 12 W8.5 X W4.0
11 in. $< t \leq 12$ in.	12 X 12 D8.6 X W4.0 or 12 X 12 W9.3 X W4.0
12 in. $< t \leq 13$ in.	12 X 12 D9.4 X W4.0 or 12 X 12 W10.1 X W4.0
13 in. $< t \leq 14$ in.	12 X 12 D10.1 X W4.0 or 12 X 12 W10..8 X W4.0

NOTE: All transverse wire is W4.0.

A discussion of the procedure used in determining the reinforcement requirements for all the types of rigid pavements can be found in the AASHTO Design Guide for Pavement Structures.

As mentioned in the longitudinal joint section, Mn/DOT does not normally tie more than three lanes together. When three or more lanes are tied together, then supplemental steel is used. Through past experience, Mn/DOT places supplemental reinforcement in panels which exceed 16 feet in width without a longitudinal joint, and in the middle lane where tied pavement widths exceed 30 feet. Number 4 bars are used when the pavement thickness is 10 inches or less and number 5 bars are used in pavements greater than 10 inches.

6. Parking Lot Design. Concrete surfacing for parking lots should be in accordance with the report "Guide for Design and Construction of Concrete Parking Lots," by ACI Committee 330.

7. Miscellaneous. Concrete Pavement Structure Appurtenances.

   a. Lug Anchors. Lug Anchors shall be placed under all rigid pavements where the grade exceeds three percent. The intent of this criteria is to prevent the concrete pavement from sliding down the grade and causing damage to the joints. On grades in excess of three percent it has been found that, as the pavement contracts each winter due to thermal change and with a frozen subgrade, the pavement has a tendency to move in the down-hill direction. Thus, the joints toward the top of the hill will usually become wider. When the spring heat thaws the subgrade, the subgrade friction increases and with the raising temperature each concrete panel expands. Because the subgrade friction is greater and with the gravity forces pulling the pavement down hill, the pavement will continue to slide in the down hill direction a little bit every year. This will continue to keep the joints "tight" all year long at the base of the hill, while the joints will get larger at the top. Eventually these joints will spall at the top from being too wide and at the bottom from excess stress. Therefore, lug anchors should be placed periodically beneath the concrete pavement to control the down-hill movement of the pavement and prevent future joint deterioration. A summary of the size and spacing of the lug anchors is found in Mn/DOT's Standard Plans Manual.

   b. Pavement Headers. Pavement Headers shall be installed at all locations where concrete pavements terminate. There are four types of headers: permanent headers, terminal headers, construction headers, and railroad crossing headers.

      The purpose of the "permanent header" is to resist the normal expansion and contraction of the concrete pavements due to the change in temperature, keep the joints adjacent to the ends of the concrete pavement from excessive movement, and prevent joint deterioration. The headers will be placed at the beginning and the end of concrete pavement jobs where the concrete pavement is placed adjacent to a bituminous pavement.

      If the concrete pavement is placed adjacent to an existing concrete pavement, a "terminal header" should be used. In this method, the new concrete pavement is doweled into the existing concrete pavement to maintain a smooth ride and prevent faulting between slabs.

      The "construction header" is used at the end of a day's paving. This header is a non-moving joint used to tie the concrete pavements together to maintain ride. See Mn/DOT's Standard

Plate No. 1150 for details on permanent, terminal, and construction headers.

The last header type is a "railroad crossing header" and is designed to resist the pavement movement due to the change in temperature and protect the railroad. See Mn/DOT's Standard Plate No. 1210 for details.

c. Approach Panels. Approach Panels are used between all bridges and the adjoining pavements. The approach panel is reinforced and is designed to carry the traffic load from the pavement to the bridge. When the bridge is built within a concrete pavement section, a special header/sleeper slab is designed to transfer the load from the pavement to the approach panel. The header/sleeper slab is used to again resist the pavement movement due to temperature changes and has an expansion device built in as a safety factor to resist possible encroachment from the concrete pavement to the approach panel and the bridge. The details of the approach panel can be seen in Mn/DOT's Standard Plans Manual, Sheets 5-297.222 to 5-297.229.

5-3.05.04 DRAINAGE CONSIDERATIONS

The provision of good drainage is a very important aspect of pavement design. Without good drainage, water from various sources can lead to saturation of a pavement structure, which in time will result in deterioration of the pavement materials. Water in the soil immediately below the pavement structure itself will also lead to a weakening of the foundation of the pavement. The combined effect of water, frost, and increased traffic volumes and loads will eventually lead to various types of premature moisture-related distress in the pavement structure.

In rigid pavements, some of the common types of problems which result from "excess" water include pumping of the base; non-uniform support from void creation; and deterioration of susceptible aggregates and mortar in concrete mixtures. Excess water in flexible pavements will also result in pumping of base fines, stripping, and accelerated fatigue or alligator cracking of the surface. Heaving of swelling soils and frost heave are some of the other detrimental effects of water that affect both types of pavements.

Typically, the water that is of concern in pavement design comes from two sources. Surface water can get into the pavement through surface defects in the pavement itself and its shoulders. The primary source of such water is rain and melting snow. Subsurface water is the other source of water in pavements. A high ground water table and water from natural sources, whose flow may be interrupted by the pavement, are often the source of such subsurface water.

The measures that should be taken to reduce the harmful effects of water to pavements consist of: (1) the prevention of water from entering the pavement structural section; (2) the provision of drainage measures to ensure that the

water that does enter the pavement is promptly removed; and (3) the design and construction of pavements adequate to withstand the expected combined effect of water, traffic loads, and other external factors. The first two measures are the recommended treatments for decreasing the detrimental effects of water in pavements. Such measures allow the pavement materials and foundation soils to retain their design properties and qualities. The measures to take in preventing water infiltration into pavements, and to remove water promptly from pavements, are discussed in Section 5-4.0. In all cases adequate ditch depth is of prime importance.

Experience has shown that attempts to solve drainage problems through pavement design, usually by increasing thicknesses and using special materials, frequently fail to meet the requirements of the combined effect of water and traffic load. To a large extent, the capability to accurately characterize the performance of the combined effect of water and traffic load on pavement materials is still limited. This makes it difficult to determine the appropriate design parameters that can actually reduce the detrimental effects of excess water.

5-3.05.05 CONSTRUCTION CONSIDERATIONS

An aspect of pavement design that is often neglected by pavement engineers is the proper implementation of the design into a useable facility. During this implementation, involving a translation of the design into a physical reality, consideration has to be given to the handling of construction traffic, scheduling of the activities of the project, and achieving a stable grade for construction of the pavement. In this section, each of these considerations is discussed.

1. Construction Traffic. Construction traffic, if not handled properly, can cause damage to pavement structures. Methods for efficiently handling traffic during construction are therefore essential. The actual methods employed will depend on the demands of a given project; consequently, no specific guidelines can be given that will be appropriate for all cases. However, certain general guidelines can be followed.

   It is always desirable for the construction of a pavement to proceed in the direction towards the sources of the materials being used. In this way, construction traffic will not be required to use the newly constructed sections of a pavement. For every project, a plan of how construction traffic will be channeled to, and from, the project at the various stages should be drawn up before the start of construction and not left to chance. Such advanced planning will ensure the smooth and timely performance of operations. Advanced planning will also minimize heavy construction traffic either having to travel at very low speeds, or even come to a stop, on incomplete pavement structures. Such occurrences can result in the initiation of certain types of distress or the outright failure of the pavement soon after construction.

   Advanced planning of the channeling of construction traffic and adherence to such plans will also considerably improve the economy of the construction and increase on-site safety. Personnel awareness of

the established patterns of construction traffic will also reduce the occurrence of accidents.

2. Scheduling. The proper scheduling of the various activities of a project is the best way to keep track of the pace of work and complete an entire project in the allotted time. A number of scheduling techniques are available for highway construction projects. Such techniques are discussed in detail in most of the available construction management texts. These techniques range from the use of simple bar chart drawings showing the expected progress of activities with time, to the use of the critical path method (CPM) which shows the progress of individual activities as well as the relationship between the various activities. The Department encourages the use of such planning and scheduling techniques in the construction of pavements. The exact technique to use depends on the nature and requirements to be met for the project.

Horizontal time bar (Gantt) charts can quickly show the planned times of start and completion of activities, and activities which must proceed concurrently, which overlap and by how much, and which must precede others. Secondary bars, color coding, and other such methods can be used to show the actual progress of activities adjacent to the bars to allow the engineer to keep track of the work in progress. A disadvantage of bar charts is that they often fail to show the interrelationships and interdependencies between activities.

Triangular bar charts have also been used in planning and scheduling. In addition to the time allotted to each activity on the horizontal scale, these show the percentage, or quantity, of work completed on the vertical scale. There are obvious advantages to being able to keep track of the rate at which the various activities are carried out. In highway projects, construction activities are often repetitive in nature, with the same activities being carried out for the completion of successive sections of the pavement. Knowledge of the rate at which repetitive activities are conducted will allow changes to be made during the construction of subsequent sections, which will be beneficial to the entire project. Other names given to this method and variations of the method, which all attempt to give an indication of the rate of progress, are line-of-balance (LOB), vertical production method (VPM), and linear schedule method (LSM).

Relatively advanced scheduling techniques, such as CPM and Project Evaluation and Review Technique (PERT), can also be used for highway projects. However, since CPM and PERT are likely to take more time to use than the simple bar charts, they are often reserved for specific activities within an entire project or for large projects where their advantages far outweigh the disadvantage of the excessive time required to use them. CPM and PERT are basically similar, and are networks which show activity duration and earliest start times, the relationship between an activity and its predecessors and successors, and the exact sequence in which activities must be performed for proper completion of the project. The major difference between the two techniques is in the

way in which the duration of activities is specified. In PERT, a most likely time, minimum time, and maximum time of duration are specified. CPM only requires a single estimate of the expected duration of a project.

3. Stable Grade. Providing a stable grade for the construction of a designed pavement is an important requirement for the successful performance and long life of a pavement. Rigid pavements are, for example, very dependent on the provision of a very uniform subgrade with no voids or distortions. Depending on the specifics of a project, there may be several requirements to meet in the provision of a stable subgrade.

   In some cases, it may only be necessary to provide the usual compaction of the subgrade soil to the required density and moisture content. In other instances, treatment of the subgrade with a stabilizing agent may be essential. Cement, asphalt, lime, and pozzolanic materials are some of the additives that are sometimes used to provide a stable grade (refer to TRB Record 705). Mn/DOT's experience with these materials has not been very successful. The approach of removing unsuitable materials and backfilling with aggregate is also a commonly used option.

5-3.06 REHABILITATION

With the passage of time and use, pavements reach a point where they have to be rehabilitated to bring them up to established performance standards. No single criteria exists for determining when a pavement requires rehabilitation, and a variety of factors play a significant role in the decision to rehabilitate. In this section, the factors that combine to determine when such rehabilitation is required are discussed. A standard procedure for determining the type of rehabilitation measure required and the various methods of repair for rigid and flexible pavements are also discussed.

5-3.06.01 PURPOSE

Poor ride quality, deteriorating surface friction, structural weakening, unprotected underlying materials, and combinations thereof are the factors which have the most influence on pavement rehabilitation decisions. For concrete pavements, rehabilitation may include surface texture planing, joint resealing, the installation of subsurface drains, partial-depth repairs, full-depth repairs, and overlays. Rehabilitation of asphaltic concrete pavements may involve crack sealing, patching of potholes, surface texture improvements, the installation of subsurface drains, and overlays. Although such rehabilitation measures may usually be adequate to improve performance, it is sometimes more cost-effective to reconstruct the entire pavement.

The choice between the rehabilitation or reconstruction of a pavement is dependent on the extent to which the factors mentioned previously have reduced the service provided by the pavement and the need to upgrade the geometric and safety characteristics of the roadway. Rehabilitation is normally selected for most projects, because it usually brings about an acceptable improvement in performance at the least cost. Reconstruction becomes the required alternative

when a pavement has deteriorated to the point where a replacement of the pavement structure is necessary and is the only cost-effective method for improving performance. The need to enhance the geometric characteristics of a pavement, such as by improving the original alignment of a roadway, may also play a leading role in the choice to reconstruct a pavement.

Other important considerations in rehabilitation are the choice of treatments and the times of implementation of the various treatments which, in the long run, will give the best results in performance and economic terms. An engineer should also be keenly aware of the effectiveness of preventive rehabilitation actions in comparison to corrective measures taken to remedy damage that has already occurred. It has become clear from experience that rehabilitation measures taken in response to damage, rather than to forestall damage, are much more expensive in the long run. All these factors are an integral part of the Department's program for pavement rehabilitation discussed in the subsequent sections.

5-3.06.02 REHABILITATION SELECTION

When pavements need rehabilitation, the goal is the selection of the appropriate strategy at the optimal time. Toward this goal, the following process was developed. It should be stressed that while this process focuses on economic analysis, there are other factors which will influence the final selection.

The two principle components needed for an economic analysis of different rehabilitation strategies are the cost and the anticipated life of the rehabilitation. The cost is fairly east to estimate with a fairly high degree of accuracy. The anticipated life is another matter altogether. Mn/DOT pavement specialists developed tables on service lifes for various rehabilitation strategies. These tables, which can be found in Appendix E., were developed by polling the specialists and should be used until better models can be developed.

The LIFE should be used, unless there is reason to expect higher or lower lives. Only in very rare cases should a life be used that is outside of the range of the LIFE plus or minus the RANGE. In other words, if the LIFE is 9 and the RANGE is 3, the life of 9 should be used unless there are reasons to expect a shorter or longer life. A shorter life than 6 years (9-3) or a longer life than 12 years (9+3) will only very rarely be allowed, with very good reason for exceeding the recommended range.

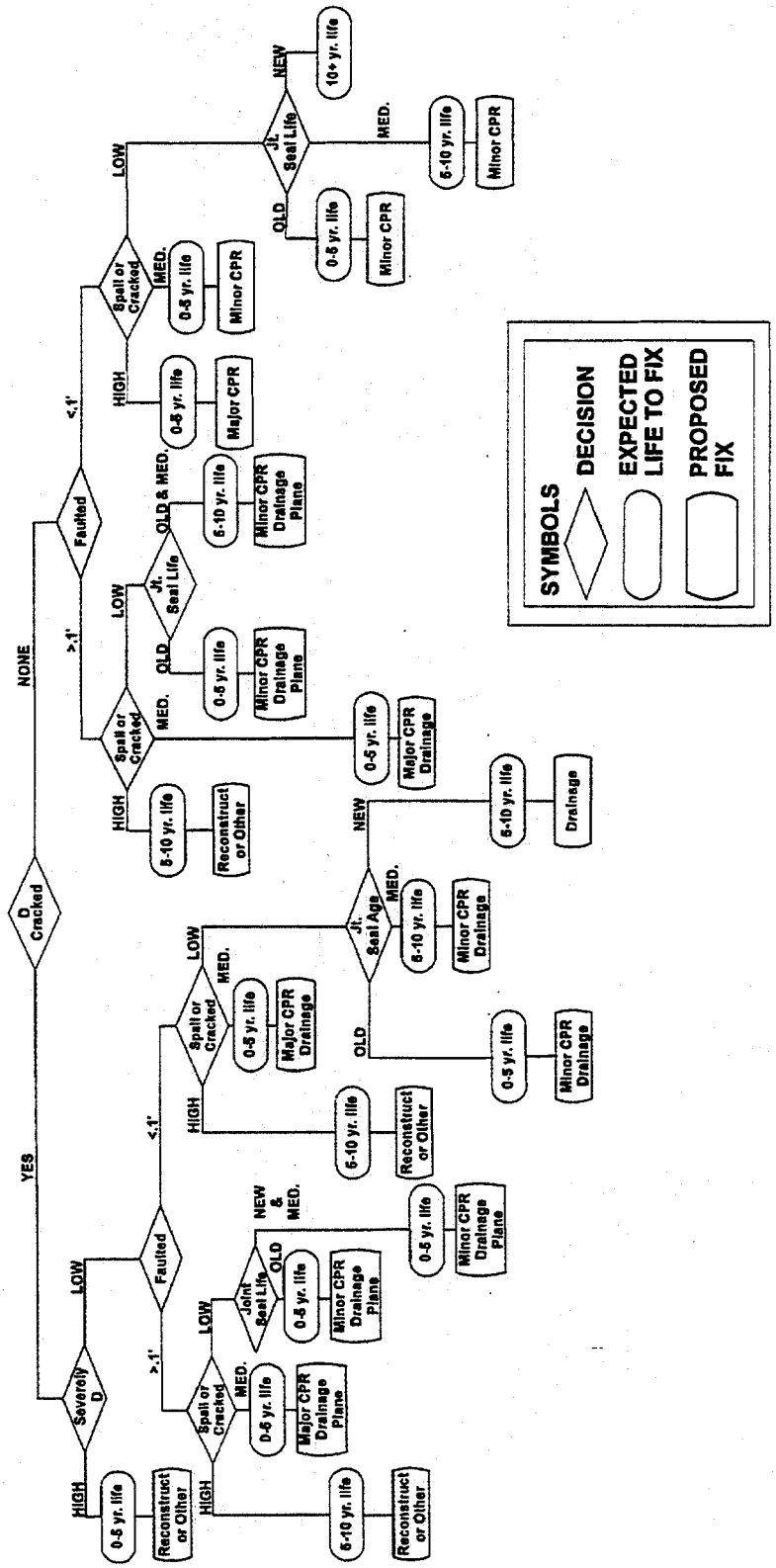
All scenarios must have an equal analysis period, and the interest rate used should be 4.5%. Note also that a series of very short (less than 8 year life) fixes should not be used. In higher traffic areas, perhaps the minimum life should be extended, so as to minimize disruption to traffic. This minimum life requirement may be lifted when the last rehabilitation selected is simply to carry the scenario to the end of the analysis period.

This process does not specify the method to be used in computing maintenance costs and user delay costs. If these costs are significantly different for various alternatives, they should be entered into the economic analysis.

All projects should undergo this rehabilitation selection process. The economic analysis should, as a minimum, be documented in the Design Recommendations Report. This analysis may actually be performed earlier, in the programming or scoping phase of project development.

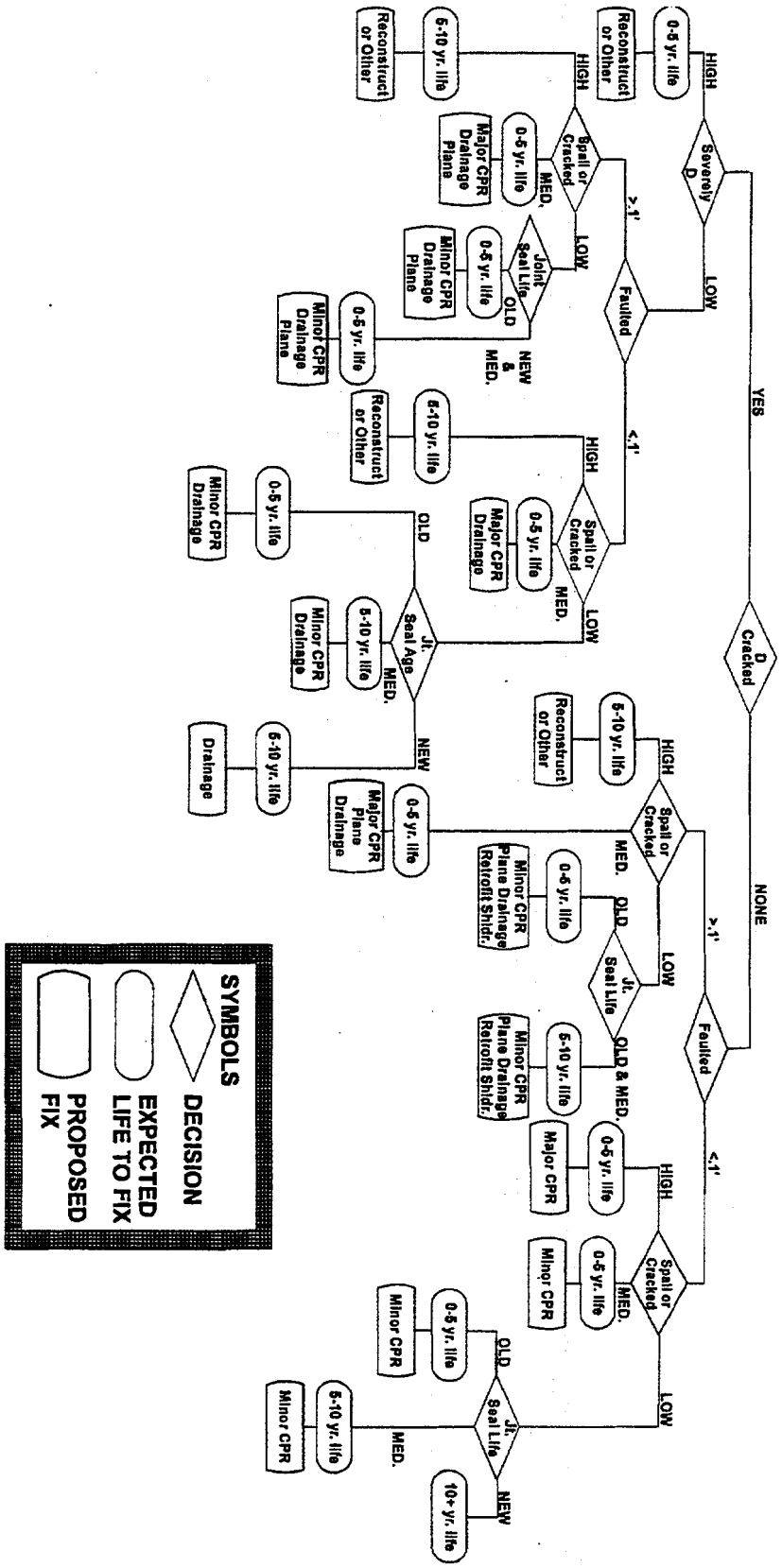
In addition to this rehabilitation selection process, the decision trees shown in Figures 5-3.9(a) and (b) may be used to help deciding what Concrete Pavement Rehabilitation (CPR) to use. These trees are applicable for concrete pavements 11 feet or greater lane width.

Figure 5-3.9(a). Decision tree for concrete pavement rehabilitation.



(a) ADT <5,000

Figure 5-3.9(b) Decision tree for concrete pavement rehabilitation.



On each of these decision trees, the amount of D-cracking, faulting, spalling, and transverse cracks; and joint sealant age are the criteria used to determine the appropriate rehabilitation strategy for a particular pavement section. The following are the criteria used to determine the path to follow to a particular strategy:

- D-cracking is "YES" if slight D-cracking is greater than five percent, otherwise D-cracking is "NO."

- Severe D-cracking is "HIGH" if medium D-cracking is greater than 25 percent, or high D-cracking is present.

- The amounts of high (H), medium (M), and low (L) transverse cracks (TC) and spalls (SP) determines the direction on the decision tree as follows:

  - Select "LOW" amount of spall and cracks, if HTC is less than five percent, MTC is less than five percent, and LTC is less than 25 percent; and HSP is less than five percent, MSP is less than five percent, and LSP is less than 25 percent.

  - Select "HIGH" amount of spalls and cracks, if HSP is greater than 60 percent, HTC is greater than 60 percent, or (HSP plus HTC) is greater than 100 percent.

  - Otherwise select "MEDIUM" amount of spalls and cracks.

  The percent of spalled joints or cracked panels is calculated as the ratio of the number of spalled joints or cracked panels to the number of total joints or panels in a given pavement section.

- Joint sealant age is categorized as:

  - "OLD" if a neoprene or silicone sealant is more than 10 years old, or a hot-poured sealant is more than five years old;

  - "MEDIUM" if a neoprene or silicone sealant is between five and 10 years old, or a hot-poured sealant is less than five years old; or

  - "NEW" if a neoprene or silicone sealant is less than five years old.

Using this system, the Department is able to generate the various rehabilitation strategies that should be carried out at a particular time for the highways in Minnesota. However, given the perennial limited amount of funds, a Minor CPR fix in the next zero to five years comes out as the most cost effective. This fix should be performed on non "D-cracked," non-faulted, low or medium spalled pavements where the joint seal is in poor condition. This fix would include resealing all joints as a Type A repair and a few Type B repairs. Major CPR work includes resealing all joints as a Type A repair, many Type B repairs and

also Type C repairs. The following priority was determined as the most cost-effective repairs:

Minor CPR,
Minor CPR & Edge Drains,
Minor CPR & Edge Drains & Plane,
Minor CPR & Edge Drains & Plane & Concrete Shoulders,
Major CPR & Edge Drains,
Major CPR & Edge Drains & Plane,
Asphalt and Concrete Overlay, and
Reconstruction.

A cost analysis was performed on these possible rehabilitation strategies.

The above mentioned priorities are constantly under review and depend on local conditions and budget constraints.

5-3.06.03 NON-OVERLAY METHODS OF REPAIR

The concrete and flexible pavement rehabilitation measures undertaken in Minnesota can be divided into non-overlay methods of repair and repair by overlay. The non-overlay methods of repair, discussed in this section, are carried out to improve the serviceability of pavements and thereby cost-effectively extend their life. These non-overlay methods of repair can also be applied before resurfacing to make rehabilitation by overlay more effective. Measures to improve subsurface drainage for both flexible and concrete pavements are also an effective, non-overlay, repair method of rehabilitation and are discussed in Section 5-4.03.

1. Bituminous Pavements. Non-overlay repair methods for flexible pavements can be generally categorized into patching with bituminous mixtures, crack sealing, surface treatments, and the cold milling of asphaltic concrete surfaces. The patching of flexible pavements with bituminous mixtures has been used as a successful rehabilitation measure for the following cases: repair of potholes, replacement of fatigue-cracked asphaltic concrete surfaces, and spot leveling to level a surface prior to overlay or to improve ride quality. Cold mixes, which are made up of a combination of aggregates and either cutbacks or asphalt emulsion binders, can be used for such patching. Cold mixes are primarily used for patching in the winter months when hot mix is not readily available. However, hot plant-mixed asphaltic concrete produced in plants, with their associated high-quality control, is inherently more suitable for patching of flexible pavements. In addition, cold mixes are not suitable for pavements which are expected to be overlaid in the near future.

   It must be noted that potholes and fatigue cracking are symptomatic of the local structural failure of a flexible pavement. Consequently, a proper evaluation of the damage and an application of the best repair methods are essential to prevent the recurrence of the damage.

Crack sealing is a rehabilitation measure often used to prevent the ingress of water into the underlying materials of a pavement following the formation of cracks. All cracks should be sealed prior to the development of spalling or base deterioration. Present practice is to seal all cracks within the first four years of a pavement's life. Crack sealing includes the routing or sawing of a reservoir with proper shape factor to hold the sealant. The reservoir is then cleaned with compressed air and/or heated compressed air (heat lance). The reservoir is filled, including an overband, with a hot-pour polymer-modified asphalt. Experimental silicone products are also available.

Reservoir shapes and overband configurations vary depending upon the specific project. Present practice is to use a 1/2 to 3/4 inch deep by 1-1/4 inch wide reservoir and a 1/16 inch thick overband extending 1 to 1-14 inches either side of the reservoir.

Pavements with cracking which have deteriorated beyond the ability of these practices to form a worthwhile seal should be treated with a crack-filling operation. Cracks should be blown clean with compressed air and filled with an asphalt-crumb rubber blend, asphalt cement (AC-3), cutback asphalt, or an emulsion. The practice of crack filling should be looked upon as a stop-gap procedure. While filling cracks does not prevent the ingress of water through the cracks, it does retard the oxidation of the crack edges by coating them with asphalt.

Surface treatments (seal coats, etc.) are a common rehabilitation measure for flexible pavements. They have long been the standard asphaltic concrete pavement maintenance and rehabilitation measure. They are used primarily on low-volume roads, where they extend the pavement life at a comparatively low cost. A host of surface treatments, classified according to the composition of asphalt and/or aggregates used, are currently used for the rehabilitation of flexible pavements. They are used to seal cracks in the surface, waterproof the surface, improve pavement surface friction and drainage, slow pavement weathering, and provide a wearing surface.

Cold milling of asphaltic concrete pavements is a restoration technique employed to correct certain types of surface distress or to prepare the pavement for an overlay. Milling machines, usually with carbide steel cutting bits, are used to chip off the surface of bituminous pavements for rut removal, restoration of the curb line of pavements, correction of cross slope, restoration of surface friction, removal of asphaltic concrete as part of surface recycling, removal of asphaltic concrete prior to overlay, and others. Cold milling, even when it is not associated with recycling or overlay construction, can in certain cases restore the serviceability of asphaltic concrete pavement and increase life.

2. Portland Cement Concrete Pavements. Non-overlay repair methods for rigid pavements include patching with concrete, joint and crack sealing, joint repairs, load-transfer restoration, surface texture planing, and slab jacking. Standard rehabilitation plates for these repair methods can be

obtained from the Concrete Unit (Office of Materials Research & Engineering). As in the case of flexible pavements, the restoration of subsurface drainage can be a very effective pavement restoration measure. It is discussed in Section 5-3.04.

The Mn/DOT concrete pavement rehabilitation standards can be classified into three Repair Types.

Type A Repairs - Joint and Crack Maintenance,
Type B Repairs - Partial-depth Repairs, and
Type C Repairs - Full-depth Repairs.

(Copies of the current pavement rehabilitation standards are available from the Office of Materials Research & Engineering, Concrete Unit.)

a. Type A. **Type A** repairs are used on all transverse joints or cracks that show no sign of spalling or delamination. This repair is also used in conjunction with the Type B and Type C repairs.

   Type A joint repairs include the removal of the in-place joint sealer by manual methods; resawing the joint to achieve a clean surface for the new sealer; and sandblasting and airblasting the joint clean of all debris. Installing the proper sized, closed cell, backer rod to achieve a two to one width to depth shape factor for the sealer. Then finally sealing the joint with an approved silicone sealer and tooling the joint to the desired shape.

   Type A crack repairs include the removal of old concrete in the joint area by sawing the joint with a small concrete saw or routing the joint. This process produces a consistent shape factor for the silicone sealer. Prior to the placement of the silicone sealer the crack should be sandblasted and airblasted clean to remove all loose particles. An approved bond breaker is necessary at the bottom of the crack to prevent adhesion. Then, finally installing an approved silicone sealer.

b. Type B. **Type B** repairs are partial-depth repairs which include the removal of the spalled concrete, replacing the concrete with high-strength concrete, and sawing and sealing the joints or cracks as necessary. The spalled concrete removal operations are performed by either of the two approved methods, sawing the concrete to aid jackhammer operations, or a milling method. This type of repair is used at spalled joints, spalled mid-panel cracks, delaminated mid-panel spalls, and at spalled pavement edges.

   The milling option for concrete removal is an excellent method and is used by most contractors in Minnesota, especially for full-width partial-depth repairs at joints. This type of repair utilizes a roto-mill that creates a half-moon shaped cross-section at the required location and leaves a bondable surface over the entire

area. In the milling operation, the joint is milled to a minimum depth of two inches, leaving the half-moon shaped area. By past experience it has been found that if over 40 percent of the joint is spalled, it is cheaper to mill the entire joint than just the spalled area.

The sawing method for removal of spalled concrete, while approved, is not used much and seems to have less success than the milling method. In this method, a saw cut to a minimum depth of two inches is made alongside the spalled area. The concrete is removed within the area by the use of an air hammer of a capacity not to exceed 30 pounds. To increase the bonding surface the saw cut edges are knocked off to approximately 45 degrees. The sawing and milling operation for partial-depth repairs may also apply to longitudinal joint rehabilitation.

All spalled concrete should be removed to a minimum depth of two inches, and a maximum depth of one-half the pavement thickness or to the top of the dowel bars, whichever is less. When the dowel bars are exposed they shall be coated with MC-250 oil or an approved equal. If the outside edges of the dowel bars are exposed, a spacer must be placed to ensure joint movement in the longitudinal direction. When heavily corroded dowel bars are encountered, that is, dowels bars that are "necked down," the dowel bars should be burned off. If there are more than three dowels removed in one joint, a full-depth repair should be recommended to restore load transfer.

If, after the removal operation, the area under the dowel bars is deteriorated, referred to as "undercutting," it is better to leave the debris in place since it will allow some movement in the joint. The determination as to when to use this type of repair method, versus full-depth repair, is made depending on the magnitude of deterioration due to "undercutting" at the joint. When "undercutting" is serious, a full-depth repair is performed. Each repair is job specific and requires trained inspectors.

In all Type B repairs, the working surface is cleaned by sandblasting and airblasting after the delaminated concrete is removed. A bonding grout is applied to this working surface just prior to the placement of the high strength concrete. The grout should have a creamy consistency and be brushed into the entire patch at the working surface. If the grout whitens, it has set up and this will require resandblasting and airblasting before the regrouting and placement of the concrete. Many patches have been known to fail due to the grout setting up prior to concrete placement.

The concrete mix the Department uses is a high-strength concrete called grade 3U18 concrete. This mix is placed in the repair area, vibrated into place, and finished to the proper grade,

slope, and texture. In all partial-depth repairs the final finishing or troweling operation should be toward the in-place concrete surface, in all directions, to prevent the patch from pulling away.

When a Type B repair is used over a joint, the new joint should be reestablished using an angle iron cutter bar at the original failure plane and later saw cutting to the required dimensions. For partial-depth repairs over a transverse crack, it is imperative that the original crack be reestablished in the exact location with the use of a flexible insert at the time of concrete placement.

After placement of the concrete, the surface tolerance is checked, and membrane cure is applied. Due to the high cement content of Mn/DOT's grade 3U18 concrete and the minimum depth of the repairs, it is imperative that the repair have an adequate curing membrane applied as soon as possible; otherwise the repair will fail due to concrete shrinkage.

The final step in the partial-depth repair procedure is the restoration of the joints or cracks. This is accomplished by using the Type A repair techniques of sawcutting the joint or crack, then sandblasting and airblasting. Finally, a closed cell backer rod and the silicone seal are installed.

c. Type C. **Type C** full-depth repairs are localized repairs where the removal of the deteriorated concrete at a joint, crack, mid-panel spot area, or full-panel removal is necessary. In a Type C repair the concrete is removed full depth and a new concrete patch is installed. These removal areas may occur at locations with subsoil problems, where panel replacement is necessary due to thermal expansion, or at locations of major construction difficulty.

It is important to note that if a full-depth repair is proposed for one lane while traffic is on the adjoining lanes of a multi-lane roadway, the adjoining lanes must have a four-inch wide transverse relief cut made prior to the full depth pavement removal. Failure to make this relief cut may cause undue stress in the pavement, and a concrete blow-up could occur. This phenomenon will occur when incompressibles enter the joints and the pavement is under compression.

In Type C full-depth repairs the concrete is sawed full depth and removed. The minimum dimension for the removal operation is a 3.5-foot by 3.5-foot section. For all full-depth repairs, subgrade consistency should be maintained. Therefore, do not disturb the grade any more than necessary during the removal operations. After the debris is removed, holes are drilled in the adjacent slab face for No. 8 tie steel on 12-inch centers. The tie steel is installed at a 20 degree skew, normal to the joint, to help prevent the new slab from separating from the adjacent slab.

The tie steel is 18 inches long and should extend nine inches into the in-place pavement. The steel is placed in the holes, along with a non-shrink grout. The grout placement must be sufficient to bond the total surface of the steel into the holes. If an adequate job is not done grouting in the tie steel, a failure will develop at the tied joint.

A full-depth repair at a joint will involve the removal of the old concrete and the old dowel bar assembly and replacing the area with new concrete and a new load transfer device. In this type of repair, one side of the repair will be tied into the adjacent panel while the other side is doweled. In this fashion, the joint may be moved from its original location, but only two failure planes are built in rather than three. In the tied end, which is usually downstream in traffic flow, the No. 8 rebar is grouted to the old slab, as listed above. On the upstream side of the repair, a straight dowel bar is installed. The epoxy coated dowel bar is a one-inch diameter dowel, 18 inches long and is installed at 12-inch centers parallel to the centerline joint. It is imperative that the dowel bars are not misaligned by more than one-eighth inch or the poor alignment will result in a non-working joint and in future joint spalling.

5-3.06.04 REPAIR BY OVERLAY

Repair by overlay is usually the last measure taken to increase the serviceability of a pavement, without which the entire reconstruction of the pavement might otherwise be required. Mainly, overlays may be justified in two ways. First, an overlay may be required to provide a better riding surface, to improve the friction characteristics of a pavement surface, or to protect the underlying layers of a pavement. In the provision of such overlays, known as functional overlays, consideration must be given to the in-place conditions that caused the decrease in pavement serviceability. Other alternatives must be examined thoroughly prior to the selection of an overlay as the method of repair.

Secondly, an overlay may be necessary to improve the load-carrying capacity of an existing pavement. Past performance, in terms of age or amount of traffic carried, and the present structural condition are key to justifying a structural overlay.

In each of these cases, it is important to ensure that the necessary localized repairs are carried out on areas of the pavement with extensive distress, before the actual overlay of the pavement. Such repairs make overlays more effective. Following are the Department's guidelines for overlays of existing pavements for the purposes stated above.

1. Functional Overlays. These are the overlays for ride improvement, improvement of friction characteristics, or pavement protection. In Minnesota a bituminous functional overlay of a concrete pavement is avoided as much as possible. If the concrete of the pavement is sound, planing is considered. In circumstances such as pavements with D-

cracking, warped and cracked panels, numerous patches, etc., bituminous overlays may be considered. In such cases, the Bituminous Engineering, Concrete Engineering and/or Pavement Design Units should be contacted.

Functional overlays of bituminous pavements with bituminous surfacing are common in Minnesota. Consideration should be given to the in-place conditions, such as cracking, oxidation, polished surfaces, flushing, stripping, raveling, mix deficiencies, and others. The reasons an overlay is required should be clearly defined, and the cause(s) of the problems or distresses warranting the overlay should be thoroughly investigated. These causes must be eliminated before placement of the surfacing. This will prevent or diminish the recurrence of the distresses. Where the pavement ride is poor because of multiple cracking due to age, temperature, or mix deficiencies, consideration should be given to milling, removal, and the recycling or replacement of the existing bituminous surfacing.

2. Structural Overlays. Overlays of both bituminous and concrete pavements for added load-carrying capacity are common in Minnesota. For such overlays, consideration of the past performance of the roadway, age, traffic carried, pavement condition survey, non-destructive load testing at the pavement surface, and properties/characteristics of the underlying soils and pavement materials is essential in the determination of effective rehabilitation measures. Knowledge of the past performance can provide a valuable weighting factor for use on any remedial measure. Borings, cores, and even test pits may be necessary to determine the extent of deterioration of the layers of the pavement.

   a. Bituminous over Bituminous. The thickness of a bituminous overlay over an existing bituminous pavement is determined from a measure of the current structural capacity of the pavement. This capacity is determined either from field deflection measurements or by using Mn/DOT's bituminous design chart.

      In the case where field deflection data is used for a road of less than 9- or 10-ton capacity, the allowable deflection for the desired tonnage is determined as required by the results of the Investigation 603 study (and modified by Technical Memorandum No. 82-26-R-1, "Determination of Allowable Spring Load Limits," July 15, 1982). Where the road capacity is in excess of nine or 10 tons, the following modified AASHO Road Test equation (Equation 5-3.7) is used to determine the allowable deflection for the sum of past and projected, cumulative, 18-kip ESALs.

$$\log \sum ESAL = 11.06 - 3.25 \log d_s \qquad \text{Eq. 5-3.7}$$

$$\text{Log } d_s = \frac{11.06 - \text{Log} \sum ESAL}{3.25}$$

$$d_s = 10^{\left[\frac{11.06 - \log \sum ESAL}{3.25}\right]}$$

where

$\sum$ESAL	=	sum of the 18-kip axle loads carried to a psi of 2.5 and
d_s	=	spring deflection, in.

If the results indicate the pavement is strong, except at transverse cracks, consideration should be given to special crack rehabilitation. The required thickness of overlay is determined using either the Investigation 603 relationship that one inch of overlay on plastic soils reduces deflection by about 11 percent or the following Equation 5-3.8 from Mn/DOT's Investigation 183.

$$\log(d_s + 2s) = 2.728 - 0.0175(G.E.) - 0.525 \log R \qquad \text{Eq. 5-3.8}$$

where

$(d_s + 2s)$	=	average peak spring Benkleman beam deflection plus two standard deviations at 80° F mat temperature,
G.E.	=	total granular equivalency, and
R	=	R value.

The overlay thickness is based on the difference between the G.E. derived from the Inv. No. 183 design chart (Figure 5-3.6) using the design R-value and the projected cumulative design lane ESALs and the reduced G.E. for the existing pavement structure. The reduced G.E. is normally taken to be between two-thirds and three-quarters of the G.E. computed for the existing pavement. It is assumed that the G.E. factors of the inplace pavement structure is the same as the G.E. factors of the original design, unless available samples indicate otherwise.

Either of the two methods described above, or a combination thereof, are acceptable for overlay design in Minnesota.

b. Bituminous over Concrete. The general Mn/DOT bituminous overlay of concrete pavements policy is to provide four to 4-1/2 inches minimum overlay on D-cracked concrete pavements and three to 3-1/2 inches minimum overlay on all other concrete pavements. The sum of the effective concrete thickness, usually two thirds to three quarters of the original thickness, and the proposed overlay thickness are checked against the full-depth design chart (Figure 5-3.7) to determine if the total thickness is adequate. The 15- to 20-year projected traffic and design R value are required for this check. If the concrete pavement is extremely deteriorated, additional overlay thickness is provided.

The promotion of better drainage, especially in the case of pavements carrying high traffic volumes, in excess of one million design lane ESALs, via improved drainage ditches or the installation of edge drains is recommended.

One serious problem which develops in most bituminous over concrete overlays is that of reflection cracking. Possible techniques for minimizing and/or reducing the severity of reflection cracking in the overlays are as follows:

- Sawing and sealing joints in the bituminous overlay at locations coinciding with joints and cracks in the underlying concrete slab. However, if the joints are badly deteriorated, the panels badly cracked and the cracks are working cracks; routing and sealing the overlay after the cracks develop may be a more appropriate technique.

- Breaking (for JRCP) or cracking (for JPCP) and seating the concrete slab before placement of the bituminous overlay. These techniques are used to reduce the size of the concrete slab pieces to minimize the differential movements at existing cracks and joints, thereby minimizing the occurrence and severity of reflection cracks. (The breaking technique is not recommended)

- Rubblizing the concrete slab before placement of the bituminous overlay. This technique results in the complete destruction of the slab section and reduces the size of the concrete pieces to a maximum of about 12 inches. The product of the rubblization process leaves an in-site layer of material similar to the appearance of unbound base layers. However, the inherent strength of the rubblized layer is several times as effective in load distributing characteristics compared with a dense graded aggregate base.

- Increasing the bituminous thickness. Reflection cracks will take more time to migrate through a thicker overlay and will deteriorate at a slower rate.

Reducing the length of the concrete slabs helps to minimize the amount and severity of reflection cracking. The smaller the cracked pieces the greater the potential for reducing/minimizing reflective cracking. On the other hand, there is also a greater reduction in the structural strength of the concrete slab. There is research ongoing by Mn/DOT to determine the optimum cracking spacing as well as the effectiveness of the cracking, breaking and rubblizing techniques as compared to other rehabilitation techniques.

The required overlay thickness is based on the difference between a new pavement design to accommodate anticipated future traffic and subgrade soil conditions and the effective thickness of the existing pavement after consideration of rehabilitation treatments of cracking, breaking and rubblizing.

If the existing concrete pavement is in relatively good structural condition (does not have a significant number of working cracks and joints are badly deteriorated, i.e., D-Cracked, spalled, etc.) the saw and seal concept should be given first consideration before cracking the pavement.

c. Concrete Overlays. Concrete overlays can be placed on existing bituminous or concrete pavements as part of a rehabilitation strategy. The design of the overlays are based on providing an overlaid pavement that is structurally equivalent to a new or reconstructed pavement designed for the anticipated future traffic. There are basically four types of concrete overlays.

(1) Bonded Overlays. With fully bonded overlays, measures are taken to ensure a complete bond with the existing concrete pavement surface so that the overlay becomes an integral part of the base slab.

(2) Unbonded Overlays. With unbonded overlays, a separation course is used to prevent the overlay from bonding to the existing pavement.

(3) Partially Bonded Overlay. For partially bonded overlay, the new overlay of concrete is placed directly on the old pavement. No special attempt is made either to obtain or to break bond.

(4) Concrete Overlay over an Asphalt Pavement (Whitetopping). Concrete overlay is placed directly over an existing asphalt pavement.

At the present time, Mn/DOT has constructed several bonded overlay projects and a whitetopping project. The bonded overlay projects have only been constructed on continuously reinforced concrete pavements (CRCP). The designs are currently being evaluated. The latest project (which is on T.H. 94 in District 4) was included in the SHRP (Strategic Highway Research Program) Program. The whitetopping project (which is located on T.H. 30 in District 7) is part of the ongoing research to evaluate various types of overlay strategies. The Pavement Engineering and Physical Research Sections (Office of Materials Research & Engineering) should be contacted for determination of the feasibility of either of these types of overlay designs for a given pavement site.

Mn/DOT has not constructed a partially bonded overlay.

Mn/DOT has constructed a number of unbonded overlay projects. The first project was constructed in 1976 on T.H. 71 in the Willmar District (District 8). To date, the unbonded overlay has been a most effective method for rehabilitating existing concrete pavements. This type of pavement design is included in the pavement selection process as an alternate consideration to the reconstruction of an existing concrete pavement. The unbonded concrete overlay design procedures and criteria are described and illustrated in Appendix D.

5-3.07 WIDENINGS

Mn/DOT has a number of guidelines for widening existing narrow concrete pavements. Widening is often required before overlaying the pavement.

Full-depth bituminous widenings for this use are normally of a width which will provide a 14-foot wide base on each side of centerline (e.g., 20- and 22-foot wide pavements widened eight and six feet, respectively). Using the 15- to 20-year cumulative traffic and the subgrade design R value, the thickness of the widening is determined from Figure 5-3.7 and increased by 20 percent to compensate for edge effects.

Aggregate base widenings should be the same width as indicated for the full-depth bituminous widenings. The thickness is determined from the 15- to 20-year cumulative ESALs and design R value using the bituminous pavement with aggregate base design chart shown in Figure 5-3.6, and again increased by 20 percent, to compensate for edge effects. This design is not recommended if the potential for severe frost differential exists. Such potential may exist if the resulting slab thickness is appreciably thicker than the in-place slab.

Concrete widenings of existing, narrow, concrete pavements are usually a minimum of six inches thick and 2-1/2 feet wide. The concrete widenings are tied to the in-place slab.

It is essential to promote lateral drainage of widened pavements, especially in cases of high traffic, in excess of one million 18-kip ESALs, with improved ditches, subsurface drains, or the replacement of the in-place shoulder with clean shoulder subbase material.

Currently, it is recommended that a drainage layer of permeable asphalt-stabilized base or open-graded base be placed beneath the bituminous and concrete widening, with a geotextile providing separation between the base and the subgrade soil. This layer drains the infiltrated surface water to a pavement edge drain system. The concept is illustrated in Figure 5-4.2C.

5-3.08 SHOULDERS

Shoulders are provided for the accommodation of stopped vehicles, for emergency use, and for lateral support of base and surface courses. It can be shown, especially for rigid pavements, that the stresses, strains, and deflections at the edge of a pavement without a shoulder far exceed those for a similar pavement with edge support provided by a shoulder. A shoulder will, therefore, increase the life of a pavement and is recommended.

Shoulders also provide recovery for errant vehicles, accident avoidance areas, lateral clearance for signs and guardrails, improved sight distance in cuts, and space for maintenance operations.

The four types of shoulders used in Minnesota are aggregate, bituminous, concrete, and composite shoulders. A composite shoulder is one partially paved with bituminous material, with the remaining portion to the outside of the bituminous shoulder constructed as a gravel shoulder. The design of these shoulders for pavements is influenced by the following broad factors: mainline pavement type, traffic, environment, safety, planned maintenance strategy, and thickness design considerations.

5-3.08.01 GEOMETRIC CONSIDERATIONS

By Mn/DOT standards the shoulder width provided should be enough to allow a stopped vehicle to clear the driving lane by at least two feet for high-volume highways. The shoulder width for a two-lane rural highway is a function of the ADT and the functional class of the highway. Refer to Section 2-5.0 of Mn/DOT's Road Design Manual for the shoulder width standards for two-lane rural highways and Section 2-6.0 for shoulder width standards for rural expressways.

For aggregate shoulders, the width is provided from the mainline/shoulder interface to the shoulder P.I. with the cut or fill slope. For bituminous and concrete shoulders, an additional width of 1-1/2 feet is provided outside of the shoulder proper to the P.I. of the shoulder, to ensure stability and provide support to the edge of the shoulder. The total width from the edge of the traveled lane to the shoulder P.I. is considered as the usable shoulder width for rural designs, i.e., sections without curbs. For urban designs, the shoulder width is the distance from the edge of the driving lane to the gutter or curb line.

On two-way roadway, the full standard shoulder is normally provided on both sides of the roadway. For four-lane divided roadways, a 10-foot paved shoulder on the right and a four-foot paved shoulder on the left is the normal design. On six or eight-lane divided roadway, 10 foot paved shoulders on both sides are provided. The shoulders should be continuous and without variation in width or elevation, with respect to the mainline pavement. Chapter 2, 4 and 7 of Mn/DOT's Road Design Manual should be referred to for specific design details for the various shoulder elements.

5-3.08.02 STRUCTURAL DESIGN

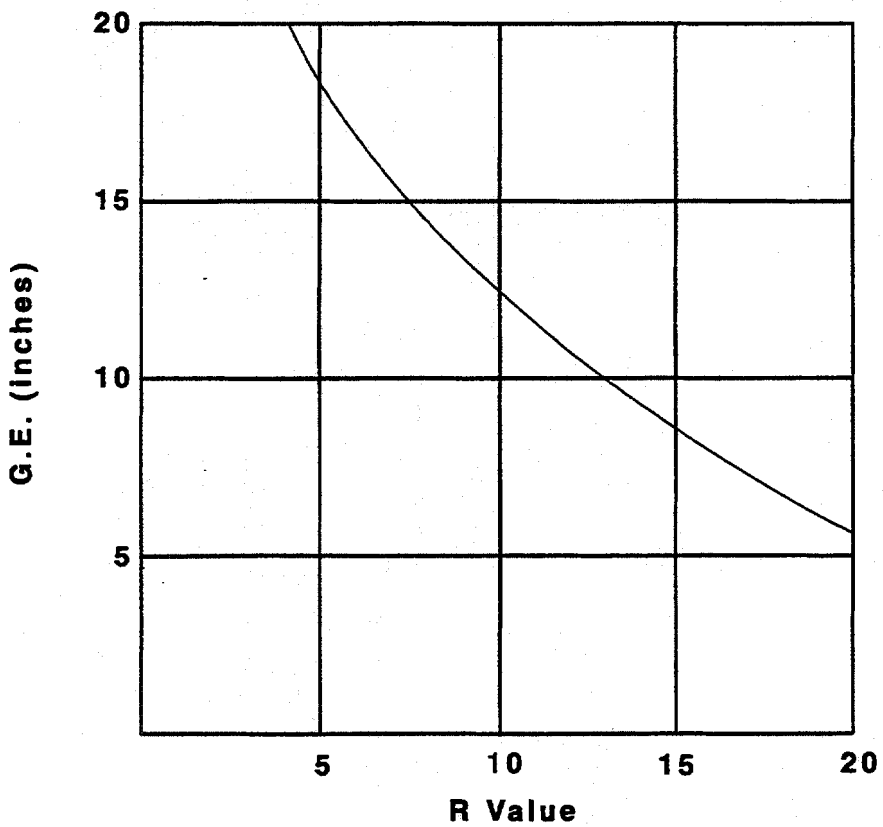
Structural Design. Standard shoulder designs for both rigid and flexible pavements are illustrated in Chapter 7 of Mn/DOT's Road Design Manual.

The G.E. thickness for the bituminous shoulders shown in Chapter 7 should be determined and evaluated relative to the minimum G.E. thickness obtained from Figure 5-3.10. When using Figure 5-3.10., the same design R value as selected for the mainline (mean minus one standard deviation) is normally used. However, there may be projects where, due to a combination of low to moderate traffic and a high degree of scatter among the actual R values (large standard deviation), a shoulder design R value somewhat greater than the mainline design R value should be used to avoid an overly conservative shoulder design. If the G.E. required from Figure 5-3.10. is greater than the G.E. resulting from the standard shoulder design (outside edge), the shoulder is lacking in structural capacity. The shoulder should then be redesigned to the G.E. determined from Figure 5-3.10. The design may include strategies such as the use of higher G.E. materials (Class 5 instead of Class 3), varying the subgrade cross slope under the shoulder, or using 2331 Type 31 bituminous base in place of a portion of the aggregate base. These alternatives would allow the design of adequate shoulders without excessive cost. The above information should be supplied to the designer by the District Soils and/or Materials Engineer in conjunction with the Pavement Design Engineer. (When bituminous shoulders are provided, the mixture type should be in accordance with Table 5-3.4.)

Also, consideration should be given to making the shoulder thickness equal to the driving lane thickness where a shoulder is expected to be used as a driving lane during period of congestion, or at a later time when traffic volumes

increase, as happen in certain urban areas. This consideration should be applied to both bituminous and concrete shoulders.

Figure 5-3.10. Minimum shoulder design.



NOTE: Based on an 0.150 inch spring Benkelman Beam Deflection (average plus two standard deviations) for a 10-ton axle load with edge effect.

Although the structural design of an aggregate shoulder is important, it is not as critical as a bituminous or concrete shoulder. The aggregate shoulder can be maintained by the addition of more aggregate while a bituminous or concrete shoulder requires extensive repairs when problems develop. Thus, the aggregate structural design may be based on a shoulder design R value somewhat greater than the mainline design R value. The design R value will

be selected by the District Soils and/or Materials Engineer in conjunction with the Pavement Design Engineer.

5-3.09 RAMPS AND LOOPS

The type of pavement materials (bituminous or concrete) used in construction of ramps and loops, as well as that used in construction of acceleration lanes, deceleration lanes, escape lanes, auxiliary lanes, directional connections, and gore areas adjacent to the through lanes should, in general, be the same materials used for the mainline roadways.

Section 6-3.0 in Chapter 6 of Mn/DOT's Road Design Manual contains typical ramp and loop cross-sections and also identifies the basic criteria used for the design of urban and rural ramps and loops. (Rural design is identified by the absence of curbs and the urban design contains curbs.)

5-3.09.01 STRUCTURAL DESIGN

1. Concrete Design

The pavement design should be in accordance with the following:

- The thickness design should be based on the procedures described in Section 5-3.05.03, Portland Cement Concrete Pavement Design, using the estimated design lane ESALs (for concrete pavement design) expected to be applied during the design period to the particular ramp, loop, etc. under consideration.

- The joint and dowel design should be based on the procedures described in Section 5-3.05.03. The transverse joints in the ramp pavement to the gore point should coincide with the joints in the mainline pavement. The transverse joints should be spaced on a 15-foot effective pattern (13, 16, 14, 17 feet) and should be skewed.

- Supplemental reinforcement should be placed in all panels with pavement widths between 16 and 22 feet. Requirements for joints and reinforcement can be found in Mn/DOT's Standard Plan 5-297.216 & .217, Concrete Pavement Without Longitudinal Joints, and Mn/DOT's Standard Plans 5-297.200 series.

- Contraction joints for ramps and loops should be specified in accordance with the details shown in Mn/DOT's Standard Plans Sheet 5-297.221.

- Concrete pavement sections of ramps with storage area should be in accordance with the detailing shown in Chapter 7 of Mn/DOT's road Design Manual.

- The normal rural ramp width of 16 feet is tapered out to two 12-foot lanes at the intersection of the crossroad. For urban loops the normal width is 18 feet. The center joint should be an LIT for rural design. For urban design the joint should be an L2KT. Curb design should be D 424 and will be carried through the tapered section. In tapered sections the joint can be terminated at a point where the taper width equals 10 feet or extended to a point near the end of the taper reaching four feet, as noted in Chapter 7, Mn/DOT's Road Design Manual. This area shall contain supplement reinforcement.

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

2. Bituminous Design

- The pavement thickness design should be based on the procedures described in Section 5-3.05.02, Bituminous Pavement Design, using the estimated design lane ESALs expected to be applied during the design period to the particular ramp, loop, etc. and the associated design R-value(s).

5-3.10 BITUMINOUS MIXTURE COMPACTION GUIDELINES

Compaction is the final stage in the placement of the bituminous mixture during the paving operation. It is the stage at which the full constructed strength of the mixture is achieved and the smoothness and texture of the mat are established. The final compaction effort plays an important role in the overall pavement performance/service life. Inadequate compaction effort results in a shorter pavement life.

It is important that the proper method of compaction is selected for use on a given project so as to ensure that the maximum pavement service life is obtained and contract administration problems are avoided.

The Engineer has a choice of several different compaction control methods to select from and recommend for use on a given project. The various compaction methods are described in detail in the Bituminous Manual, Standard Specifications and Supplemental Specifications. A brief description of the methods are provided for review as follows:

A. Control Strip Method (2331.3H1).
This process calls for the construction of a test strip and the monitoring of the increasing level of compaction with the nuclear device after each pass. (A pass is defined as one coverage over a given test location). The requirements for establishing this test strip are included in the specification. After compaction is completed the mean density of the control strip will be determined by averaging the results of ten nuclear density tests taken at randomly selected sites within the control strip. The mean density of the control strip shall be the target density for the remainder of the pavement course it represents. If the mean density of the control strip determined is less than 96 percent of the density of a specimen of the bituminous mixture compacted by the Marshall compaction method as described in the Department's Bituminous Manual, the Engineer will order construction of a new control strip. The higher control strip is used for density control.

Mix placed on the project is divided into five sub-lots per day and the density determined at a randomly selected site in each sub-lot by the Nuclear Density Device. Both the mean and range of the Relative Density results between sub-lots are used in determining a "Quality Level" which is used for determining acceptance or Pay Reduction Factors (Table 2331-4 Specification Book).

The specifications required that the nuclear testing device be furnished and operated by Department personnel.

B. Ordinary Compaction Method (2331.3H3).

1. Supplemental Specifications (dated January 2, 1991) to the 1988 Standard Specification For Construction (one pay item).

This process involves the construction of a control strip (growth curve) where the degree of compaction is monitored by a nuclear density device after each roller pass. Compaction is continued until the nuclear device shows that further compaction effort would cause a reduction in inplace density. The rolling pattern (number of passes by each roller) used in the control

strip is used on the remainder of the project unless changes dictate a new "control strip" (growth curve) is necessary.

The specifications require that the nuclear testing device be furnished and operated by the Contractor.

2. 1988 Standard Specifications for Construction (two pay items). This process involves compacting each course until there is no further evidence of consolidation and all roller marks are eliminated. When this method is employed and the amount of mixture placed by the paver exceeds 100 tons per hour at least 3 rollers will be required for compacting the mixture placed by that paver. A vibratory roller may be used on a performance basis in lieu of the above roller requirements.

C. Specified Density Method (2331.3H2).
This process involves comparing the Bulk Specific Gravity of a core obtained from the roadway with the Bulk Specific Gravity of a sample obtained from the same material prior to compaction and then compacted by the "Field Marshall Method." Relative Density at 95% and higher are acceptable, levels below are subject to penalty based on a "Pay Factor" (Table 2331-5 Specification Book).

D. Modified Specified Density Method (2340.3H).
Each day's production is divided into five sub-lots, two cores are taken from each sub-lot and the Bulk Specific Gravity of each core is determined. Using the appropriate average of the Maximum Specific Gravities determined on the mixture produced for this lot and the average Bulk Specific Gravity for each sub-lot, the "Percent Maximum Theoretical Density" (100 - % in-place Voids) is calculated for each sub-lot. Acceptance is based on the "Percent Maximum Theoretical Density" for each lot (91% minimum) and the lowest sub-lot (89% minimum). A penalty table (2340-3) is included for values less than the minimum allowed (Pay Factor A & B).

In selecting the compaction control methods to be used on a project, it should be kept in mind that each method has certain advantages and disadvantages depending upon factors such as underlying support, lift thickness and uniformity, project size, traffic control and construction schedule. Therefore, in order to assist in the selection of the proper method for each specific situation the following set of guidelines is provided:

a. Control Strip
   1. Used For:
      a. QM or Non-QM
      b. Uniform thickness

c. Uniform support
d. Sufficient tonnage of material
e. Areas where adequate density is necessary but time for testing is limited

2. Not Used For:
a. Variable thickness and support
b. Small projects, intermittent paving
c. Where gauge interference may occur (i.e. reinforcing steel, steel slag, etc.)

b. Ordinary Compaction (with Control Strip growth curve analysis*)
1. Used For:
a. QM or Non-QM
b. Shoulders
c. Resurfacing (over weak grade/support)
d. First lift of Full Depth bituminous pavement structure constructed on a plastic subgrade.
e. Small projects
f. Thin lifts (less than 1 1/2 inches)
g. Irregular paving areas
h. Miscellaneous paving (i.e. tight blading, trenches, rumble strips, etc.)

* In accordance with Supplemental Specifications to the 1988 Standard Specification for Construction, dated January 2, 1991.

2. Not Used For:
Heavy traffic where conditions permit other methods.

c. Specified Density
1. Used For:
a. QM and Non-QM projects
b. Uniform thickness
c. Small quantity, intermittent paving or irregular paving areas where assurance of density is critical.
d. Adequate paving structure/platform

2. Not Used For:
a. Resurfacing (weak pavement structure).
b. Courses less than 1 1/2 inches plan thickness

d. Modified Specified Density

1. Used For:
   - a. QM Projects
   - b. High traffic volume
   - c. Adequate pavement strength
   - d. Uniform thickness
   - e. Sufficient size project to allow adequate sublots

2. Not Used For:
   - a. Non-QM Projects
   - b. First lift of Full Depth bituminous pavement structure constructed on a plastic subgrade
   - c. Resurfacing (weak pavement structure)
   - d. Variable thickness
   - e. Where construction scheduling does not allow for delay in coring by at least 12 hours
   - f. On project where intermittent paving operations produce small or multi-day sublots
   - g. Courses less than 1 1/2 inches plan thickness

5-4.0 DRAINAGE

Highway drainage consists of directing surface and subsurface water away from structural elements of the roadway and toward a safe outfall through the use of ditches, culverts, drains, drainage blankets, etc.

5-4.01 GENERAL CONSIDERATIONS

Most mechanisms of pavement failure are accelerated by surface and subsurface water. Many drainage-related failure mechanisms, such as erosion, uplift between structural elements, frost, and pore pressure-related failures, occur in the presence of free water. Pavements constructed without effective drainage will deteriorate at an accelerated rate relative to a similar well-drained pavement. Free water should be kept, as a minimum, below the depth of frost penetration. If free water is present within the depth of concern, drainage design must either raise the finished grade or lower the local ground water table.

The effect of surface and subsurface water on planned structures and the effect of planned structures on natural drainage patterns should be evaluated. An investigation should be conducted to determine the existing ground water levels below planned highway structures and drainage structures. A boring program should be based on the purpose of the structures and site factors, such as size of structures, soil permeability, and depth of subgrade improvement needed. Locations of potential concern include springs, wet ditches/backslope (cattails), existing pavement cracks which show seepage, and areas that have inadequate surface drainage away from the pavement. any evidence of artesian conditions or soft soils should be noted and further explored. Evaluation should consider special ground water control methods and include filter design, use of geotextiles, etc., to prevent erosion of drained materials, clogging of drains, and inadequate flow capacity of drains. Filter criteria are discussed in Section 5-4.03.02. Design of buried conduit, such as culverts and storm drains, should include an evaluation of proper bedding and cover to prevent settlement and/or heave and potential for pipe breakage and consequent erosion of the surrounding soils into the conduit. Perpetuation of existing agricultural or county ditch drainage must also be evaluated.

5-4.02 SURFACE DRAINAGE

5-4.02.01 DITCHES

Ditches are open channels that collect runoff and seepage water from pavements and slopes and convey it via gravity to a safe outfall. Ditches are the least complicated and most cost-effective means of collecting and conveying water from surface and shallow subsurface flows. In granular, free-draining soils, ditches are typically used to lower the local water table.

In order to determine the proper design elements of a ditch, it is necessary to define the purpose of the ditch. The following is a discussion of some specific uses of ditches in roadway design:

- **Roadside Ditches** are ditches that parallel roadway edges and conduct water longitudinally. The standard geometry for Mn/DOT's ditches is a rounded trapezoidal configuration with a bottom width of about eight feet, as referenced in Mn/DOT's Road Design Manual, Section 4-6.0. Also specified is a desired longitudinal ditch gradient of 0.005 feet per lineal foot, to prevent ponding water in localized low areas. A typical ditch depth is about four to six feet, but design depth should be equal to or greater than the depth of subgrade correction necessary to support traffic.

- **Median Ditches** are a variation of roadside ditches used to conduct water longitudinally within median areas to median collection points. Median ditch grades may vary from the roadway grades as necessary to handle the runoff from lanes between the divided roadway crowns, as well as from the median area itself.

- **Offtake Ditches** are more or less perpendicular to roadway alignments and conduct flows to outfalls which parallel roadways, such as rivers or shorelines.

- **Intercepting Channels** are located at the top of or along intermediate elevations of cut slopes to prevent excessive flows down cut slopes. The use of these channels must be approved by the Foundations Unit (Office of Materials Research & Engineering) due to the potential for slope instability resulting from raising the ground water level in the slope.

- **Toe-of-fill Channels** are similar to intercepting channels and are used along the toe of fill slopes as control at the base of slopes.

Ditch slopes and bottoms are typically lined with three to 12 inches of topsoil or other surfacing materials. Ditch design should account for thickness and roughness of lining materials when sizing for depth of subgrade improvement and maximum slope.

Aside from the initial design and construction of a ditch, the most important element affecting ditch performance is the maintenance of the proper ditch depth. As a part of routine roadway maintenance, inspection for excessive siltation or vegetation which would impair the function of the ditch should be accomplished.

Additional policy and design information is addressed in Mn/DOT's Road Design Manual, Sections 4-6.02, 4-6.03, 8-3.02, 8-4.01, and 8-5.01.

5-4.02.02 CURB AND GUTTER/STORM DRAINS

Gutters are formed by using a curb to contain pavement surface drainage within a roadway, often to direct it toward storm-drain systems. They are typically used in relatively urban areas where right-of-way is limited and/or roadway channels are undesirable for other reasons. Design for curb and gutter systems should include a hydraulic analysis to determine the extent of flooding in the roadway lanes during the design storm event. Storm drains are used to conduct water from within the curb and gutter system to a safe outfall. The presence of free water poses specific installation problems and the potential for buoyancy of the installed pipe. Additional policy and design information is addressed in Mn/DOT's Road Design Manual, Sections 4-4.04, 8-3.04 and 8-4.04.

5-4.02.03 CULVERTS

Culverts are open-ended conduits used to carry runoff under roadways. Culverts are typically based on natural or improved natural drainageways to limit grading and disruption of the existing subsurface ground water regime. Analysis and design of culvert systems should include investigation to determine the existing ground water levels and soil types/conditions at the site. Policy has been to investigate proposed culvert sites with at least three, augered, soil borings; one boring near each end of the structure and one boring near the center of the structure. Extension of satisfactorily performing culverts should utilize bedding material similar to that of the in-place culvert. Additional policy and design information is addressed in Mn/DOT's Road Design Manual, Sections 4-4.04, 8-4.02, 8-4.03, 8-6.02, 9-3.02.

5-4.03 SUBSURFACE DRAINAGE

Subsurface drainage, also called subdrainage or underdrainage, is employed when ground water or infiltrating surface water is present, or likely to exist within the frost depth (even deeper in silty soils). To insure that groundwater does not adversely affect long term roadway performance, grades should be separated from the water table by a depth roughly similar to the depth of frost penetration. (This depth is generally considered to be about five feet as a state-wide average.)

Every effort should be made to satisfy this grade-water criteria. If grades cannot be kept at least five feet above the water table, then special ground water control designs will be required. Water levels can be permanently lowered by the use of deep longitudinal drains, blanket drainage layers in the bottom of subcuts or special drainage arrays. In most cases such drains require project specific designs that relate to parameters such as soil type(s) permeability(s), depth of lowering required, depth to confining layer, etc. Permanent lowering of the water table should not be undertaken without consideration of possible

adverse consequences, such as construction expense, longevity of the design, impacts on adjacent wells or wetlands, and potential of settlement of any structures overlying organic soils or loose sands where water is withdrawn. Assistance with special ground water control designs can be obtained from the Geology Unit (Office of Materials Research & Engineering). It is extremely important that all ground water data be accurately summarized and tabulated in the Design Recommendation Report and also be made available to prospective bidders/contractors.

Subsurface drains should comply with Mn/DOT's Specification 2502, Subsurface Drains, and any appropriate Special Provision(s). Additional policy and design information is addressed in Mn/DOT's Road Design Manual, Sections 8-3.06 and 8-4.05. (Note that Figure 8-3.06A, Road Design Manual, Jan 31, 1982, does not reflect current drainage design practice.) Inclusion of subsurface drainage in any design should be based on current Mn/DOT design guidelines, which guidelines would include the following elements: Roadway functional classification, traffic, subgrade soil type(s), and condition/performance of the inplace pavement (if not a new alignment). The Geology Unit (Office of Materials Research & Engineering) should be contacted for assistance with subsurface drainage designs.

5-4.03.01 SUBSURFACE DRAIN TYPES

The following various types of drains are used by the Department:

- **Subcut drains** are typically longitudinal perforated pipe placed in the bottom outside corners of a granular backfill subcut. These drains collect infiltration water that may pond in the bottom of the subcut (bath tub). Such drains are sometimes used to collect ground water and are occasionally used in conjunction with a blanket-type drainage layer at the bottom of the subcut. They generally discharge via non-perforated pipe to the ditch. Additional guidance is found in Mn/DOT's "Design Guidelines for Subcut Drainage."

- **Pavement edge drains** are narrow trench and perforated pipe systems which parallel roadway edges at depths of approximately 12 to 24 inches to drain water infiltrating the pavement structure. These drains are typically used on new or rehabilitated concrete or full-depth bituminous pavements on non-granular soils but other uses may be recommended by the Soils Engineer. Drain grades should generally be at least 0.5 percent, but grades as low as 0.2 percent are acceptable when necessary to minimize trench depths or to insure that headwall outlets are 12" (6" min.) above ditch grades. Additional guidance is found in Mn/DOT's "Design Guidelines for Pavement Edge Drains."

- **Permeable aggregate base drains** are used to collect water from permeable bases, either the stabilized (PASB) or unstabilized (OGAB)

type. As is the case for other drain systems, these drains should be properly designed to pass the design flows and properly filtered to prevent infiltration of adjacent materials into the permeable trench backfill. Additional guidance is found in Mn/DOT's "Guideline for the Application and Design of Permeable Aggregate Base PAB and Associated Drainage Systems Under Highway Pavements."

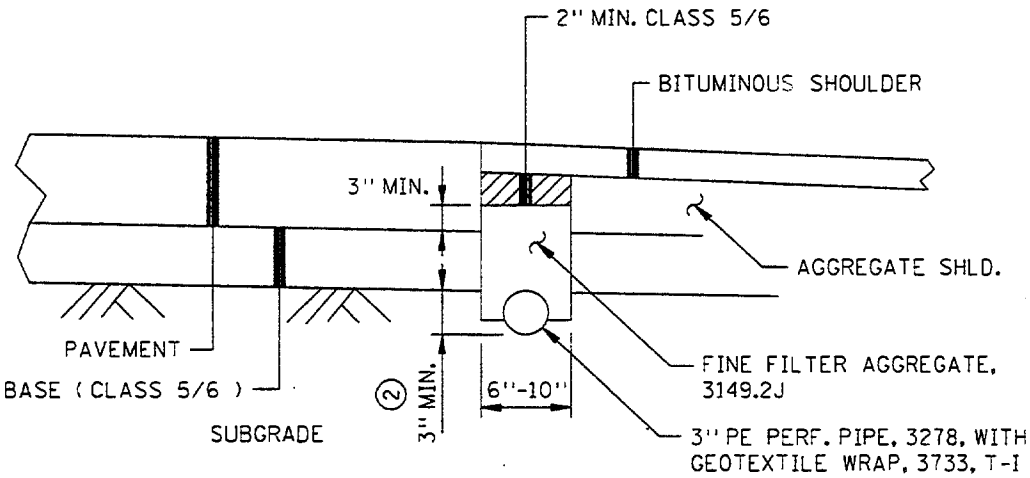
- **Interceptor drains** (mini weeps) are utilized with unbonded concrete pavement overlay designs to collect water from joints and major cracks in PCC pavements to remain in place. These interceptor drains typically connect into a standard permeable base drain for discharge.

- **Horizontal drains** consist of perforated pipe placed in bored holes into soil or rock cut slopes, more or less perpendicular to roadway centerline, to lower ground water pressures, and prevent slope failures.

- **Special drains** such as those associated with pavement widenings, permeable stress relief layers for unbonded concrete overlays, or ground water control, are typically modifications of other types of drains.

Typical sections for various types of subsurface drains are shown in Figures 5-4.1(a) through (f). It should be noted that these are "typicals" and will likely need to be modified slightly for specific project use. The most current version of these, as well as other drainage typicals, are available in the Central Office CADD design file. Also, appropriate special provisions for each type need to be included in the contract. Standard special provisions are available in a special provisions package distributed by the Special Provisions Unit.

Figure 5-4.1(a) SUBSURFACE DRAIN, PAVEMENT EDGE DRAIN TYPE ①

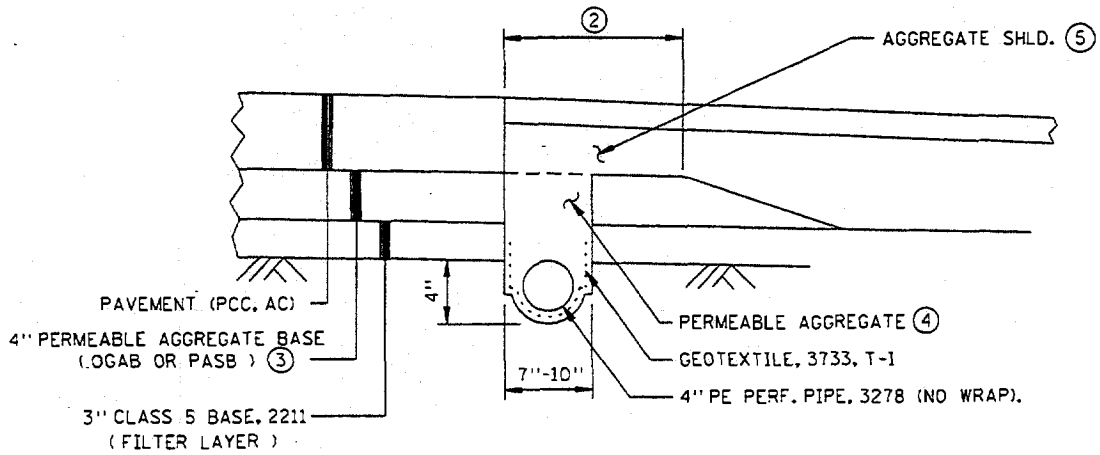


NOTES:

① SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS.

② DESIGN FOR 15" COVER FROM TOP OF PIPE TO TOP OF SHOULDER (12" MINIMUM).

Figure 5-4.1(b) SUBSURFACE DRAIN, PERMEABLE AGGREGATE BASE TYPE ①



NOTES:

① SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS. TYPICAL SECTION SHOWN IS FOR PERMEABLE ASPHALT STABILIZED BASE (PASB). DRAIN TRENCH FOR OPEN GRADED AGGREGATE BASE (OGAB) IS MOVED 6" AWAY FROM THE PAVEMENT EDGE.

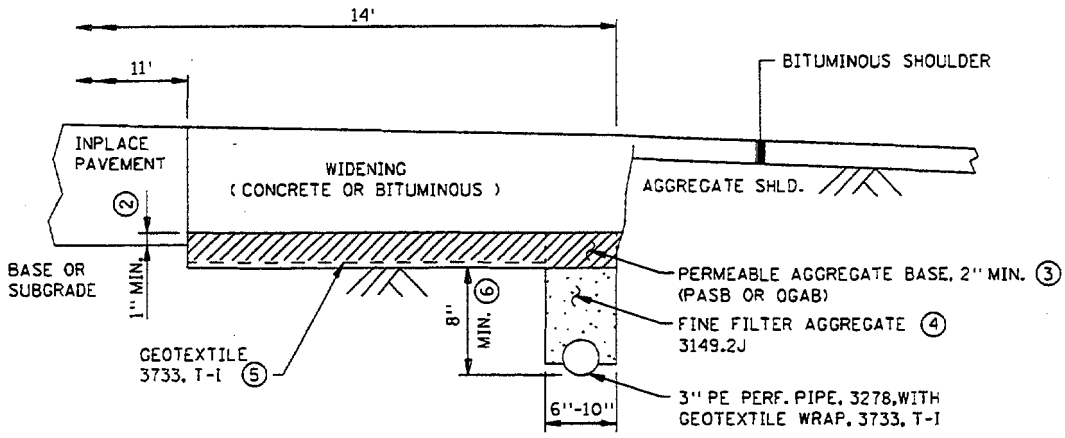
② USE 36" FOR EITHER PASB OR OGAB UNDER PCC PAVEMENT
USE 12" FOR PASB UNDER AC PAVEMENT

③ OGAB - OPEN GRADED AGGREGATE BASE.
PASB - PERMEABLE ASPHALT STABILIZED BASE.
USE PASB WITH AC PAVEMENTS.
USE PASB OR OGAB WITH PCC PAVEMENTS.

④ SEE SPECIAL PROVISIONS FOR AGGREGATE SPECIFICATIONS

⑤ CLASS 3, 5 OR 6, AS SPECIFIED

Figure 5-4.1(c) SUBSURFACE DRAIN, WIDENED PAVEMENT DESIGN WITH PAVEMENT EDGE DRAIN ①



NOTES:

① SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS.

② PERMEABLE BASE SHOULD OVERLAP PAVEMENT MAXIMUM AMOUNT PERMITTED BY STRUCTURAL DESIGN, BUT BOTTOM SHOULD NOT BE ABOVE THE BOTTOM OF INPLACE PAVEMENT.

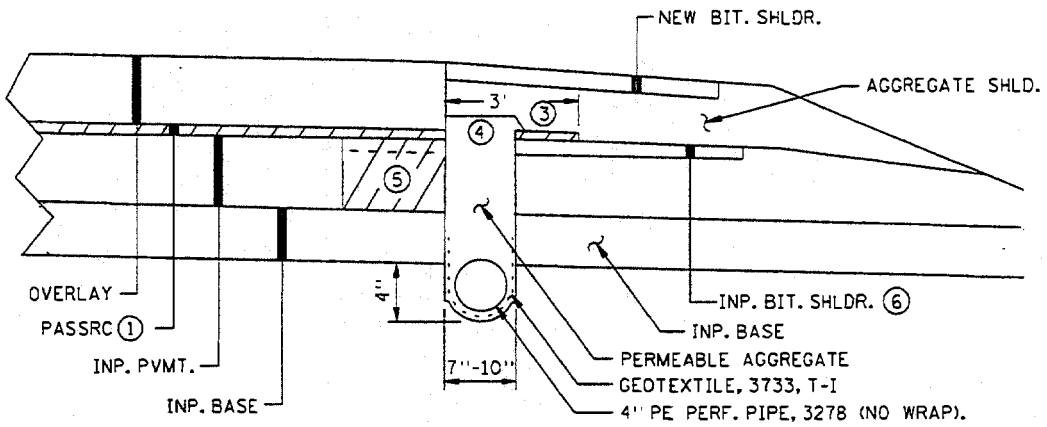
③ AS REQUIRED BY DESIGN STANDARDS.
PASB - PERMEABLE ASPHALT STABILIZED BASE.
OGAB - OPEN GRADED AGGREGATE BASE.
PAB - OPTION

④ DRAIN SHALL BE PAVEMENT EDGE DRAIN TYPE.
AFTER COMPACTION, FINE FILTER AGGREGATE IN DRAIN SHALL EXTEND AT LEAST 4" ABOVE THE BOTTOM OF THE FUTURE PERMEABLE AGGREGATE BASE.

⑤ GEOTEXTILE MAY BE DELETED IF CLASS 5 OR 6 BASE EXISTS INPLACE UNDER PERMEABLE BASE.

⑥ IF CLASS 5 OR 6 BASE IS INPLACE BELOW THE PAB, BOTTOM OF PIPE SHOULD BE A MINIMUM OF 3" BELOW BASE/SUBGRADE INTERFACE OR A MINIMUM OF 8", WHICHEVER IS DEEPER.

Figure 5-4.1(d) SUBSURFACE DRAIN, PERMEABLE BASE & DRAIN USED WITH PASSRC ① ②



NOTES:

① PASSRC - PERMEABLE ASPHALT STABILIZED STRESS RELIEF COURSE

② SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS

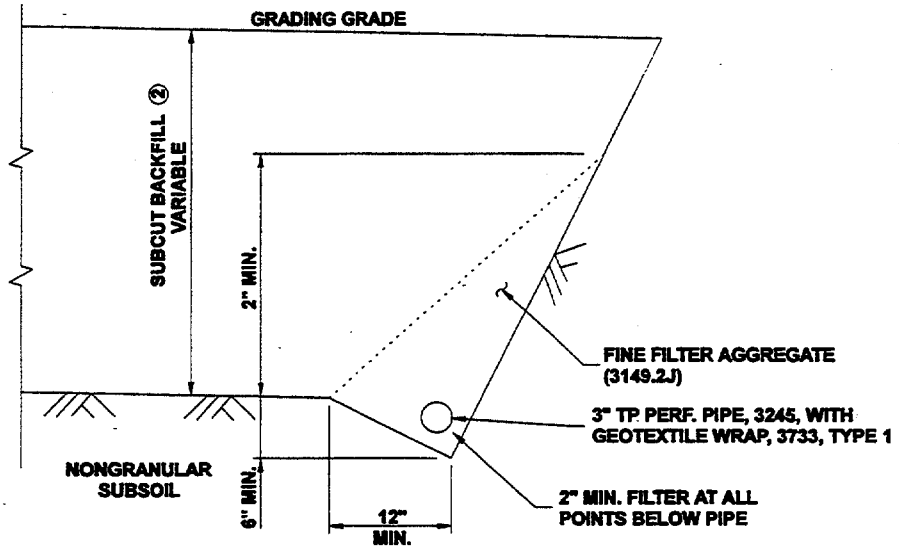
③ WIDTH AS NEEDED TO SUPPORT PAVER TRACK

④ PERMEABLE AGGREGATE TO BE HEAPED 2" ABOVE TOP OF PASSRC AFTER COMPACTION

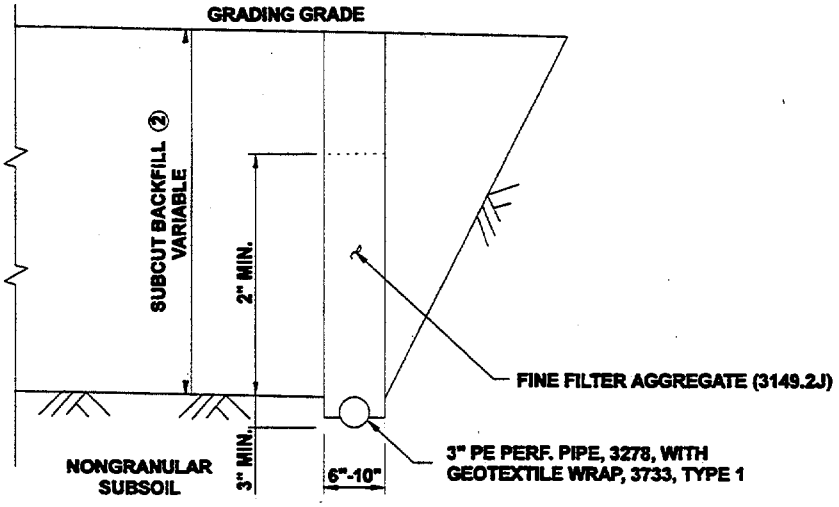
⑤ INTERCEPTOR DRAINS TYPICALLY USED AT THIS LOCATION. SEE DETAIL & SPEC. PROVISIONS IF APPLICABLE

⑥ IF THE BIT. SHLDR. REMAINS INPLACE, THE PASSRC AND SHLDR. CAN BE REMOVED BY MILLING, TRENCHING, OR OTHER METHOD, PROVIDED THE REMAINING BIT. SHLDR. IS NOT DISTURBED/DISPLACED

Figure 5-4.1(e) SUBSURFACE DRAIN, SUBCUT DRAIN TYPE



TYPEICAL SECTION (OPTION NO. 1) ①

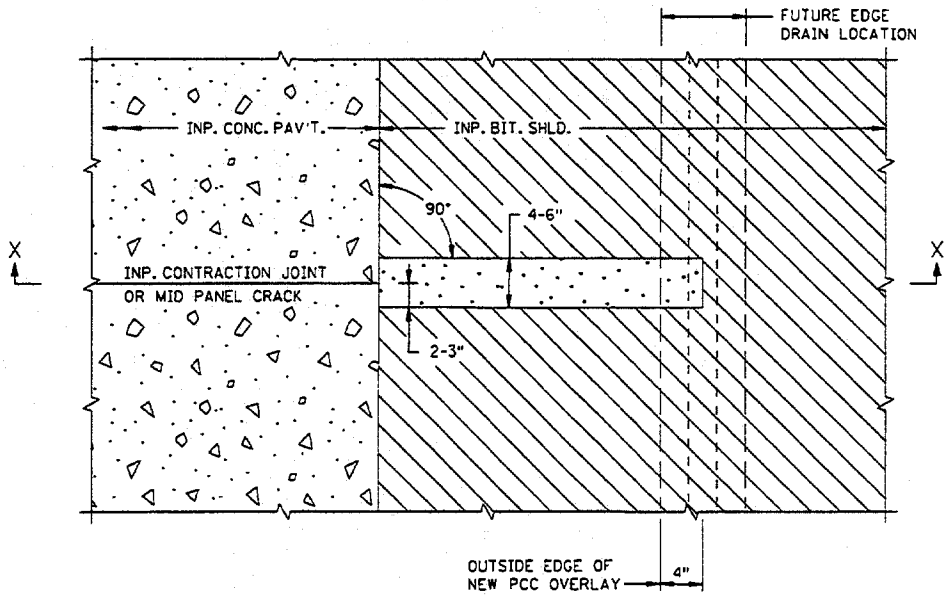


TYPEICAL SECTION (OPTION NO. 2) ②

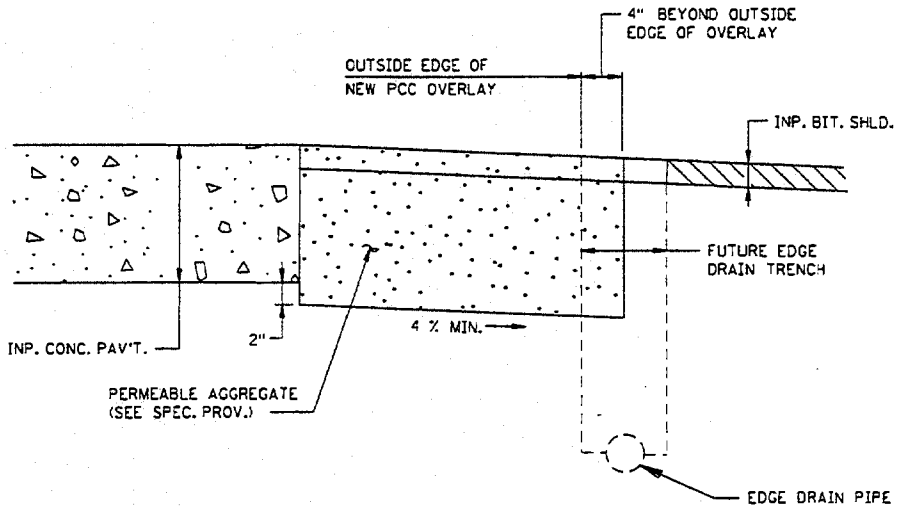
① MAY NEED TO BE MODIFIED FOR SPECIFIC PROJECTS. SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS. OPTION NO. 2 MAY ONLY BE USED WHEN PIPE IS TO BE PLACED BY MACHINE TRENCHER.

② GRANULAR, SELECT GRANULAR, OR SELECT GRANULAR MODIFIED. (AS SHOWN IN DESIGN RECOMMENDATION LETTER.)

Figure 5-4.1(f) INTERCEPTOR DRAIN DETAIL ①



PLAN VIEW



SECTION X-X

NOTE:

① SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS

5-4.03.02 SUBSURFACE DRAIN MATERIALS

1. Pipe. Pipe should always be used to collect and/or convey subsurface water from a drained material to a safe outfall. The pipe may be perforated (to allow infiltration) or non-perforated. If pipes are used only to convey water, such as from a drain to an outlet, the pipes should be non-perforated. Pipe type and size should generally be as shown in the typical sections in Figure 5-4.1. Standardized pipe sizes and materials are as follows:

   - Plastic.

     - Four- to 15-inch diameter, rigid thermoplastic (TP) - Mn/DOT 3245.

     - Three- to 10-inch diameter, flexible, corrugated, polyethylene (PE), drain tubing - Mn/DOT 3278.

   - Clay or Concrete. Four- to 10-inch diameter (only used to extend/perpetuate agricultural drainage) - Mn/DOT 3276.

   As a general rule, three- and four-inch diameter pipes are used to collect water infiltrating from rain or snow, while six-inch diameter or larger sizes are used for ground water control designs or unusually long pipe runs.

   Both TP and PE pipe may be used anywhere in the roadway system, provided they have at least 15 inches of cover. Polyethylene pipe is generally only used when it can be placed by machine trencher in a narrow trench with controlled backfill and compaction. Thermoplastic pipe should be used in all other applications, e.g. wide trenches, bridge approaches, behind retaining walls, etc. Thermoplastic discharge pipe and concrete headwalls should always be used to convey drained water to the ditch.

   Thermoplastic (TP) pipe should generally be installed with the perforations or slots located on the bottom. Perforations or slots must be compatible with the surrounding filter aggregate or wrapped with geotextile to prevent clogging of the perforations or loss of filter and/or surrounding soil into the pipe (piping). Hole sizes for perforated pipe are governed by the applicable specifications.

   Pipe slopes in subsurface drainage applications should be as appropriate for the design, but a minimum slope of 0.2 percent is recommended and 0.5 percent is preferred. An exception to this guideline is pipe placed for permeable aggregate base drains, which will always have a grade that parallels the pavement grade. Drainage pipe

is typically paid by the installed footage, to include trenching, pipe, filter and compaction. Therefore, when pipe grades are not at a uniform depth, the designer should indicate the range of depths by run(s) to enable the contractor to accurately calculate a bid price per foot.

The longitudinal grade of pipes should be checked often during installation. The grade should be kept uniform with continuous positive drainage in the direction of the flow of water. Care should be taken to prevent intrusion of the surrounding materials into the pipe and to prevent breaking or collapse of the pipe. Continuity of the drain and discharge pipe, after installation, should be checked both by visual inspection and by pushing a probe into the drain as required by the special provisions. The Department currently has two, 2" diameter color video cameras which can be used to inspect up to 500 feet of drainage installation (or for other uses as appropriate). Discharge trenches are typically backfilled with a lower permeability soil, or "plug," which serves to prevent piping and erosion along the outside of the pipe.

Geocomposite drains (combination of a formed plastic core with geotextile wrap) have been used for edge drains, drains behind retaining structures, under roads, in landslide repairs, and as vertical drains to increase the rate of consolidation under fills. (Geocomposite drains are not recommended for use as pavement edge drains at this time.) The use of geocomposites is very specialized and should be discussed with the Foundation Unit and/or Geology Unit (Office of Materials Research & Engineering).

2. Granular filters. Granular filters are used to allow unrestricted drainage of a saturated soil, while preventing piping/intrusion of the drained soil into the filter or even worse, into the perforated pipe or drainage layer. The filter layer must be graded (designed) to prevent piping and also to have adequate permeability. Filter layers are always coarser in grain size than the soil they protect (drain). Filter design criteria are well established in many references, and are only briefly discussed below. It should be noted that these same criteria/equations are also widely used to design or to verify the potential for a finer soil/material to migrate into a coarser soil/material even in the absence of water, e.g. under the influence of gravity or traffic.

The following equations characterize the filter criteria.

Filter Criteria (soils):

Piping $$\frac{D_{15Cr}}{D_{85F}} \leq 5$$ (Eq. 5-4.1)

Uniformity $$\frac{D_{50Cr}}{D_{50F}} \leq 25$$ (Eq. 5-4.2)

Permeability $$\frac{D_{15Cr}}{D_{15F}} \geq 5$$ (Eq.5-4.3)

Designs should satisfy all three soil filter criteria, but the most important is Eq. 5-4.1.

Filter criteria (pipe perforations):

Round Holes $D_{85m} \geq 1.2 \times \text{hole diameter}$ (Eq. 5-4.4)

Slots $D_{85m} \geq 1.4 \times \text{width of slot}$ (Eq. 5-4.5)

Where:

D_{85}	=	size of particle (based on percent passing graph of gradation) of which 85% of the material is smaller.. Similarly for D_{50} and D_{15}.
Cr	=	coarsest material.
F	=	finest material.
m	=	material next to pipe (typically filter aggregate.

<u>Filter Design Example</u> - Fig. 5-4.2

Design a granular filter to protect a fine sand with the gradation shown in Fig 5-4.2. The filter should be designed so as to prevent the filter material from entering a perforated pipe with either ⅛" round holes or slots.

For the fine sand: $D_{85} = 0.012$ in
$D_{50} = 0.006$ in
$D_{15} = 0.0033$ in

(Since a filter is always be coarser than the material it protects, the filter is the coarse (Cr) material and the sand is the fine (F) material.)

Example Calculations:

Piping

$$\frac{D_{15Cr}}{D_{85F}} < 5, \quad \frac{D_{15Cr}}{0.012} < 5, \qquad \text{(Eq. 5-4.1)}$$

$$\therefore D_{15Cr} < 0.06''$$

Uniformity

$$\frac{D_{50Cr}}{D_{50F}} < 25, \quad \frac{D_{50Cr}}{0.006} < 25, \qquad \text{(Eq. 5-4.2)}$$

$$\therefore D_{50Cr} < 0.15''$$

Permeability

$$\frac{D_{15Cr}}{D_{15F}} > 5, \quad \frac{D_{15Cr}}{0.0033} > 5, \qquad \text{(Eq. 5-4.3)}$$

$$\therefore D_{15Cr} > 0.0165''$$

Round Hole

$$D_{85m} > 1.2 \text{ x hole diameter } (0.125''), \qquad \text{(Eq. 5-4.4)}$$

$$\therefore D_{85m} > 0.15''$$

Slot

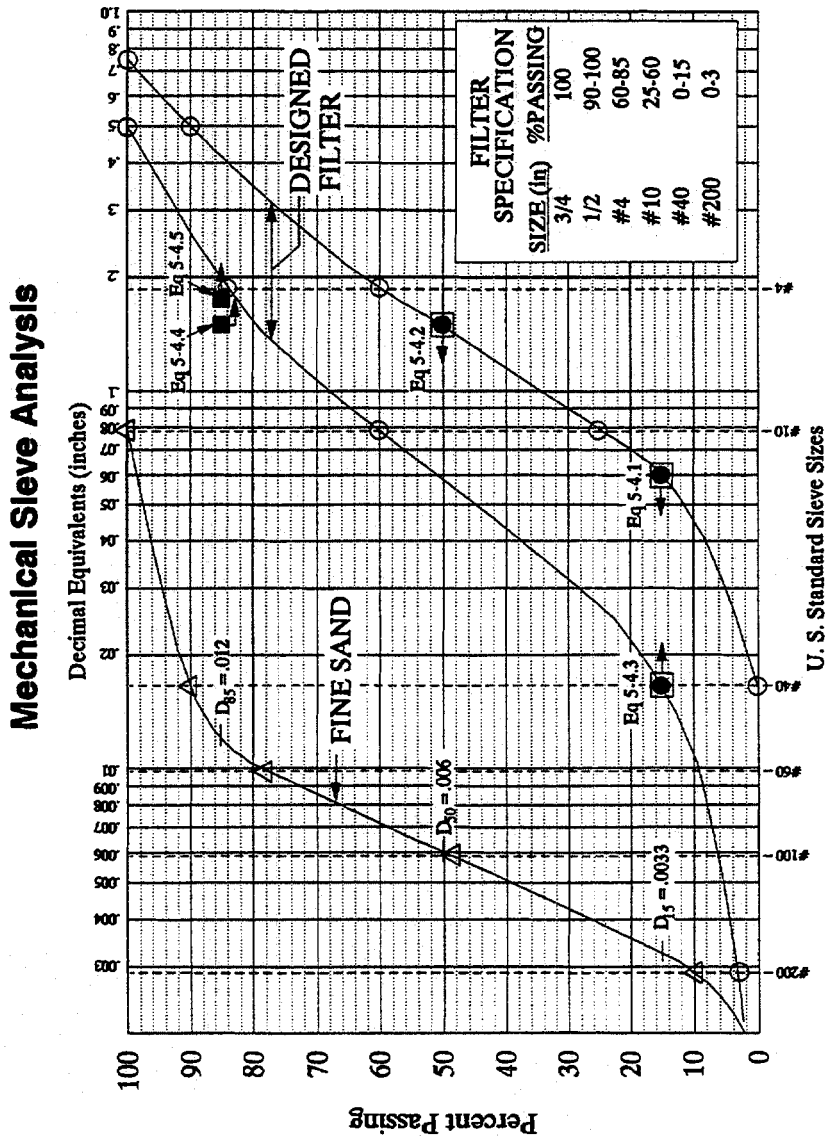
$$D_{85m} > 1.4 \text{ x slot width } (0.125''), \qquad \text{(Eq. 5-4.5)}$$

$$\therefore D_{85m} > 0.175''$$

The resulting design for the filter envelope is shown on Fig. 5-4.2. Note that there are only four "control" points (only three if pipe is not involved). The coarse and fine sides of the filter envelope should roughly be parallel and should generally be drawn so that specified sieve sizes are in multiples of five, e.g. 60-85 or 25-60, <u>not</u> 62-81 or 24-58. Note also that the resultant designed filter is quite coarse compared to the protected fine sand and also that <u>no</u> geotextile is required (or

should be used) around the pipe. If the fine side of the designed filter envelope will not satisfy the perforation criteria then the pipe must be wrapped with geotextile, but this should be avoided if possible. For specification purposes, the designed filter should be specified as either Mn/DOT 3149.2J (Fine Filter Aggregate, Mod.) or 3149.2H (Coarse Filter Aggregate, Mod.), depending on which of the two gradations is closest to the designed gradation.

Figure 5-4.2 Granular Filter Calculations



3. Geotextiles Filters. The use of geotextiles, in lieu of an aggregate filter in subsurface drainage situations, can greatly reduce the amount of time necessary to make a drain installation; and, their thinness, relative to an equivalent, properly graded, aggregate filter, saves space and allows water to pass without piping (erosion) of adjacent soil fines. Mn/DOT 3733, Type 1 geotextile is used for most subsurface drainage applications. Fine filter aggregate, Mn/DOT 3149.2J, is typically used in standard pavement edge drain designs, as well as for many other uses. (see Figure 5-4.1(1).) This granular filter will protect most soils from "piping" into the perforated drain pipe. In this design, the geotextile on the pipe keeps the filter aggregate from falling through the pipe perforations and is necessary because the filter aggregate does not meet the perforation criteria as required by Eq. 5-4.4 or 5-4.5. Special purpose geotextile and drainage designs should be discussed with the Geology Unit.

5-5.0 AGGREGATE SURFACED ROAD DESIGN

The structural requirements for aggregate surfaced roads are a function of the subgrade soil strength, expected traffic load applications and the strength provided by the various types of surfacing materials.

The design procedure consists of the determination of the total thickness of aggregate material required to be placed above the subgrade, as well as the determination of the thicknesses of the different aggregate layers required for the given traffic and subgrade conditions. A design guide for determining the respective thicknesses was developed for the counties, townships and municipalities entitled "Minnesota's Design Guide for Low Volume Aggregate Surfaced Road." This guide can be found in Appendix B.

A reference to an FHWA publication on the use of geotextiles in unpaved roads is included at the end of this chapter.

REFERENCES

Ahlborn, G., ELSYM5, Computer Program for Determining Stresses and Deformations in Five Layer Elastic System, University of California, Berkeley.

American Association of State Highway and Transportation Officials, AASHTO Interim Guide for Design of Pavement Structures, Washington, D.C., 1981.

American Association of State Highway and Transportation Officials, AASHTO Guide for Design of Pavement Structures, Washington, D.C., 1986.

Asphalt Institute, Thickness Design - Asphalt Pavements for Highways and Streets, Manual Series No. 1, Maryland, September 1981.

Cedergren, Harry R., Drainage of Highway and Airfield Pavements, John Wiley and Sons, Inc., New York, 1974.

Dehlen, G. L., The Effects of Non-linear Response on the Behavior of Pavements Subjected to Traffic Loads, Ph.D. Dissertation, University of California, Berkeley, 1969.

De Jong, D. L., M. G. F. Peutz, and A. R. Korswagen, Computer Program BISAR, Layered Systems Under Normal and Tangential Loads, External Report AMSR.0006.73, Koninklijke Shell-Laboratorium, Amsterdam, 1973.

Duluth Technical College, Civil Engineering Technology and Minnesota Department of Transportation, Grading and Base II Course Book, 1992-93.

Duncan, J. M., C. L. Monismith, and E. L. Wilson, "Finite Element Analyses of Pavements," Highway Research Record No. 228, Highway Research Board, Washington, D.C., 1968, pp. 18 - 33.

ERES Consultants Inc., Pavement Design, Principles and Practices, U.S. Department of Transportation, Federal Highway Administration, National Highway Institute, September 1987.

ERES Consultants Inc., Techniques for Pavement Rehabilitation, U.S. Department of Transportation, Federal Highway Administration, National Highway Institute, October 1987.

Ioannides A. M. and J. P. Donnelly, "Three-Dimensional Analysis of Slab on Stress-Dependent Foundation," Transportation Research Record 1196, Transportation Research Board, Washington, D.C., 1988.

Jorenby, B. N. and R. G. Hicks, "Base Course Contamination Limits" Transportation Research Record 1095, Transportation Research Board, Washington, D.C., 1986.

Koerner, Robert M., Designing with Geosynthetics, Prentice Hall, New Jersey, 1990.

NCHRP, Design and Use of Highway Shoulders, NCHRP Synthesis of Highway Practice, Report No. 63, 1979.

United States Army Corps of Engineers, EM 1110-2-1901, "Engineering and Design, Seepage Analysis and Control for Dams," 30 September 1986.

Tayabji, S.D. and P.A. Okamoto, "Thickness Design of Concrete Reinforcing," 3rd International Conference Concrete Pavement Design and Rehabilitation, Purdue Indiana, 1985.

United States Department of Transportation, Federal Highway Administration, Highway Subdrainage Design, Report No. FHWA-TS-80-224, 1980.

United States Department of Transportation, Federal Highway Administration, Geotextile Engineering Manual, Publication No. FHWA-HI-89-050, 1989.

United States Department of Transportation, Federal Highway Administration, Geotextile Design and Construction Guidelines, Report No. FHWA-HI-90-001, 1990. (Edited version of above, with construction-related additions.)

United States Department of Transportation, Federal Highway Administration, Geotextile Selection and Installation Manual for Rural Unpaved Roads, Report No. FHWA-RT-89-050, 1989.

Chapter Six
DESIGN RECOMMENDATIONS REPORT

6-1.0 GENERAL

The Design Recommendations Report is a summary of the project geotechnical and pavement survey information, as well as the information used to develop the project design. It includes all of the design and construction recommendations developed for the project, including any alternatives. A Design Recommendations Report will be prepared for each project by the District Soils and/or Materials Engineer and submitted to the District Engineer and Office of Materials Research & Engineering for review and comment. The review by the Office of Materials Research & Engineering will be in accordance with the recommendations of the Soils Letter Resolution Relationship Task Force Report as implemented by Technical Memorandum 91-43-ME-20. This Memorandum can be found in Appendix C.

Foundation surveys, analysis and recommendations required for proposed bridges, culverts, buildings, retaining structures, embankment fills, swamp crossings, etc. will be prepared by the Foundations Unit (Office of Materials Research & Engineering) and submitted to the requesting Division/District Engineer for incorporation into the project design.

6-2.0 GUIDELINES

An outline of a Design Recommendations Report is included on the following pages. This outline is intended to provide a checklist for the District Soils Engineer to prepare the Design Recommendations Report. It includes recommended major headings (underlined), as well as recommended subtopics for each major heading.
The objective of preparing a Design Recommendations Report is to summarize *all* of the pertinent information that is used in the development of a design, so the outline covers many topics that may not need to be addressed on each and every individual project. In providing this outline, it is hoped to develop a degree of uniformity in the presentation of the Design Recommendations Reports, while retaining enough latitude in the contents to reflect the individuality of each reporting individual and each project.

6-3.0 REPORT OUTLINE

Following is the recommended outline for the Design Recommendations Report.

STATE OF MINNESOTA

DEPARTMENT: _____

Office Memorandum

_____

DATE: _____

TO: _____
District Engineer

FROM: _____
Materials/Soils Engineer

PHONE: _____

SUBJECT: Design Recommendation Letter

S.P. _____ (T.H. _____)

F.A.I. _____ (if applicable)

Geographical limits/location (termini)

Station limits (major equations only)

Length of project, in miles

Type of construction project, e.g., overlay, resurface, reconstruction, etc.

PROGRAMMED LETTING DATE

_____

DESIGNER

_____

I. GENERAL PROJECT INFORMATION

A. Project Scope
Proposed Construction, Reconstruction, Resurfacing and Rehabilitation

1. Project Termini
   - Termini (or limits) of various construction operations (milling, resurfacing, etc.)
2. Proposed Alignment Changes
3. Proposed Geometric Design Features (No. of lanes, widths, turn lanes, bypass lanes)
4. Proposed Structures (bridges, culverts, etc.)
   a. Type
   b. Number
   c. Locations
   d. Describe proposed rehabilitation
      Remove & reconstruct, widening, realigning, etc.
5. Proposed Interchanges - Type and Location
6. Describe Proposed Work

B. Project Background Information

1. Existing/Inplace Facility
   a. Geometric Features (no. of lanes, widths, mainline and shoulders)
   b. Existing Pavement Structure
      (Bituminous, Concrete, Base)
      (1) Bituminous Pavement
         (a) Thickness and types of various courses (including thicknesses of aggregate bases)
         (b) Pavement width (mainline)
      (2) Concrete Pavement
         (a) Thickness (concrete and bases)
         (b) Width (mainline)
         (c) Reinforced (JRCP), Non-reinforced (JPCP), Continuous Reinforced (CRCP)
         (d) Joint spacing
         (e) Dowelled/non-dowelled
         (f) Joint seal material
      (3) Shoulder
         Material/Width/Thickness
      (4) Subsurface Drainage
      (5) Construction History
      (Include dates, and refer to Typical Sections)
   c. Past Traffic (ESALs)
   d. Previous Reports and Correspondence, by date

2. Site Topography
   a. Type and Amount of Relief
   b. Land use
   c. Water Courses, Ponding Areas and Wetland Locations (Natural and man-made)

C. Geologic Information/Survey

1. Surface Geology and Features of Interest
   a. Pleistocene (Glacial) Soil Features
      (1) Glacial landforms/terrain
      (2) General soil types
      (3) Extent/Uniformity
   b. Holocene (recent) Soil Features
      (1) Recent landforms/terrain
      (2) General soil types
      (3) Extent/Uniformity
   c. Bedrock Features
      (1) Rock type/formation
      (2) Extent/outcrops
      (3) Weathering/Condition
2. General Subsurface Soil/Rock Conditions (Depth to Rock, refusals other than bedrock - boulders, etc.)
3. Data Presentation
   a. Drilling Field Logs/Observations
   b. Plot Soil and Bedrock Profile, Topsoil Horizons and Water Levels (with descriptive adjectives)
   c. Ground Water Data
      (1) Levels (including piezometers) and wet ditches
      (2) Springs
      (3) Nearby wells
   d. Field and Laboratory Test Data
      (1) Gradations
      (2) Proctor's
         (a) Maximum densities
         (b) Optimum moistures
      (3) Atterberg Limits
      (4) Textural Classifications
      (5) Percent Passing No. 200 Sieve
      (6) R-Values
      (7) etc.
   e. Interpretation and Analysis of Data
      (1) Design R-value
      (2) Modulus
      (3) Undesirable soils
      (4) High water table
      (5) Rock cuts
      (6) Etc.

D. Pavement Field and Laboratory Data

1. Deflection Measurements (FWD)
2. Test Data Results (from cores, test pits, soil borings, etc.)
   a. Core Thickness/Density/Air Void, etc.
   b. Extraction Test Results
   c. Gradations
   d. Etc.

E. Project Evaluation (Rehabilitation, etc.)

1. Pavement Management Data
   Pavement Historical Condition (PQI, PSR, SR)
2. Identify Pavement Distresses
3. Deflection Measurement Analysis/Structural Capacity Analysis
4. Surface and Subsurface Drainage Analysis
5. Laboratory Test Data Analysis
6. Identify Factors Causing Distresses
7. Identify Feasible Alternatives that will:
   a. Address causes of deterioration
   b. Be effective in repairing existing distress(es).
   c. Prevent the premature reoccurrence of the distress(es).

F. Project Analysis

1. Perform Necessary Engineering and Economic Analysis.
2. Select Best Rehabilitation Alternatives.
3. Provide Necessary Recommendations Under II. Design Recommendations.

G. Traffic Data Collection and Analysis
(Present and 20-year design lane ADT and ESALs)

H. Turf Establishment Soils

1. Inplace Topsoil Type (Textural Classification)
2. Inplace Topsoil Depth/Extent

II. DESIGN RECOMMENDATIONS

A. Removal, Salvage and Disposition

1. Existing Structures - Culvert, Etc.
2. Pavements (Bituminous and Concrete)
3. Aggregate Base
4. Topsoil

B. Foundation Surveys, Analysis and Recommendations

1. Recommendations
   a. Bridge Design & Construction
   b. Embankment Design and Construction
   c. Etc.

C. Excavation and Embankment Construction

1. Grading Soil Evaluation
   a. Define Suitable/Unsuitable Grading Materials
   b. Define Top of Grading Grade
   c. Borrow
   d. Shrinkage Factors (soil and rock)
2. Subgrade Corrections/Compaction Subcuts
   a. Subcuts, (length, depth, tapers, drainage, backfill material, typical section)
      (1) Mainline
      (2) Widened section
      (3) Turn and bypass lanes
   b. Special Treatments
      (1) Rock Excavation (backslopes, tapers, depth, etc)
         (a) Blasting, ripping
         (b) Special uses (riprap on aggregate, etc.)
         (c) Vibration impacts on wells, structures, people
      (2) Swamp Treatments
         (a) Excavation (depth and width)
         (b) Environmental (disposal, haul, backfill, borrow, etc)
         (c) Floated embankment (Geosynthetics)
         (d) Embankment widening
      (3) Frost heaves
      (4) Drainage/dewatering (Temporary or permanent)
         (a) Surface
         (b) Subsurface

3. Embankment and Backfill Material Placement
   a. Transitions
   b. Slope Preparation - Embankment Widening
   c. Backfill Materials (granular, selected grading, etc.)
   d. Geotextiles
   e. Compaction Requirements
   f. Control Rate/Surchange
   g. Test Rolling
4. Culvert Treatments

D. Pavement Structure

1. New Construction/Reconstruction
   a. Pavement Design - Mainline
      (1) Bituminous Surfacing Designs (full depth and aggregate base)
         (a) Aggregate base/permeable base types and thicknesses
         (b) Bituminous layers (courses, thickness and mix types)
         (c) Drainage (edge drains)
         (d) Saw and seal
         (e) etc.
      (2) Concrete Surfacing Design
         (a) Base/sub-base/permeable base types and thicknesses
         (b) Pavement thickness
         (c) Pavement type (JPCP, JRCP)
         (d) Joint design
            i) Transverse joint spacing, dowel size and joint type
            ii) Longitudinal joint type (tied or untied)
            iii) Joint sizes and sealant material
         (e) Drainage (edge drains)
   b. Shoulder/Turnlane/Bypass Lane Design
      (As above for mainline pavement, but using appropriate traffic numbers.)
   c. Ramps, Loops, Climbing Lane, etc.
      (As above for mainline pavement, but using appropriate traffic numbers.)

2. Rehabilitation
   a. Bituminous and Bituminous over Concrete Pavements
      (1) Pre-overlay repairs (crackfilling, patching, milling, etc.
      (2) Overlay types (functional or structural)
      (3) Milling
         (a) Depth
         (b) Width

(4) Overlay
(a) Courses and mixture types
(b) Thicknesses
(5) Drainage (Edge drains)
(6) Saw and Seal
(7) Etc.

b. Concrete
(1) Repair type (A, B, & C)
(2) Drainage (Edge drains)

c. Concrete Overlays
(1) Unbonded Overlays
(a) Pre-overlay repairs
(b) Stress relief layer (thickness & material type)
(c) Thickness, joints, etc. refer to new construction
(d) Drainage (edge drains, interceptor drains, permeable stress relief layer)
(2) Bonded Overlays
(3) Concrete over existing bituminous pavement (white topping)

d. Widenings
(1) Courses and thicknesses (permeable base)
(2) Mixture types
(3) Drainage (edge drains)

e. Shoulder thickness Design (base and surfacing)

3. Miscellaneous Bituminous Items
a. Mix Design Requirements
b. Quality Control Requirements (i.e. 2340 QM and 2331 - for non-QM)
c. Density Control
(Based on QC choices)
d. Tack coat uses and application rates
E. Pre-overlay repairs
(1) Patching requirements
(2) Patching mixture (type, placement and compaction requirements, etc.)

4. Aggregate Base Compaction Requirements

E. Erosion Control and Turf Establishment

1. Topsoil
2. Lime
3. Fertilizer
4. Mulch
5. Seed Mixtures

F. Aggregate and Borrow Sources

1. Possible Sources of Aggregate
a. Source number
b. Legal descriptions
c. Owner's name

d. Material available (class, use)
e. Lease expiration date

2. Granular and Common Borrow Sources
   a. Non-exclusive lease designation
   b. Cost of borrow per cu. yd (1 yd.^{3} or lump sum)
   c. Legal Description
   d. Duration of lease
   e. Disposition of topsoil and final shaping of pit

G. Miscellaneous

We concur with the foregoing recommendations, except as noted by insert on the attached sheet.

	DATE	INSERT
Pavement Design Engineer		
*Bituminous Engineer		
*Concrete Engineer		
*Aggregate Engineer		
*Agricultural Engineer		
*Foundations Engineer		
*Geologist		

* NOTE: District Soils/Materials should be included only when relevant to the recommendations contained in the body of the report.)

cc: Pavement Engineer/Geotechnical Engineer
Pavement Design Engineer
Bituminous Engineer
Concrete Engineer
Special Provisions Engineer
Estimating Engineer
Preliminary Design Engineer
Central Office Pit Lease Specialist
District Personnel

Plus other appropriate individuals as determined in the District and/or by the Pavement Design Engineer. If it is necessary to send a copy to a Consultant, another Agency, or a non-Mn/DOT individual, provide a complete address or include other instructions.

APPENDIX A

AGGREGATE SOURCES

GRAVEL EXPLORATION AND LEASING

You have been contacted today by an employee of the Minnesota Department of Transportation (Mn/DOT) for permission to test drill your property for possible sources of sand and gravel. Mn/DOT routinely searches the state for new sources of gravel to supply the large quantities required for highway construction and maintenance. Locating and leasing gravel deposits near future highway projects enables Mn/DOT to make gravel sources available to all contractors who are bidding on state highway projects, thus lowering construction costs.

With your permission, a Mn/DOT drilling crew will be scheduled to test your property in the near future. They will drill a number of widely spaced holes in selected areas to check for the presence of suitable gravel. All drill holes will be backfilled immediately after drilling, and will be marked with wooden stakes for future reference. However, these stakes may be removed by you if your field work makes it necessary. Mn/DOT will mail you a copy of the drilling results several months after the drilling is completed. This length of time is necessary for Mn/DOT to process and analyze laboratory samples and determine if your property contains a suitable gravel deposit.

If you do have a gravel deposit that meets our needs, a Mn/DOT Right-of-Way agent will contact you to discuss details of leasing. There are several advantages a property owner has when leasing a gravel deposit to Mn/DOT. First of all, the usual type of lease that Mn/DOT uses is a non-exclusive lease. That is, you will be free to sell gravel to other buyers even if Mn/DOT's lease is in effect at the time. You may also ask Mn/DOT's agent to include clauses in the lease to require that slope dressing, topsoil replacement, and other pit cleanup work be done after gravel has been excavated by a contractor working under the terms of Mn/DOT's lease.

The lease price you and Mn/DOT agree upon will be a fair one, as it will be based upon the current market price for gravel in your area. If gravel from your property is used on a Mn/DOT project under terms of Mn/DOT's lease, Mn/DOT will keep an accurate accounting of all material taken and you will receive prompt payment by State warrant.

If you have any questions regarding the above information, you may contact the Mn/DOT Aggregate Engineer or one of his assistants at (612) 779-5611.

AGGREGATE SOURCE LEASING REQUEST

DATE: _____

TO: _____
District R/W/Land Management Engineer

FROM: Warren Pladsen, Aggregate Engineer
1400 Gervais Avenue, Maplewood, Mn 55109

VIA: _____
District Materials Engineer

SUBJECT: AGGREGATE PIT/PROSPECT - _____

_____. _____County

We are considering using the subject aggregate property as a possible source of aggregates for
S.P. _____, T.H. _____, Location

_____Letting Date: _____
PLEASE ARRANGE FOR LEASING THE FOLLOWING DESCRIBED TRACT(S):

_____
Sec. _____, T _____, R _____
OWNER: _____
_____
CITY: _____ PHONE: _____
OTHER: _____
_____
_____

COMMENTS BY MATERIALS ENGINEER:
_____
_____

STATUS: (Please notify Materials Engineer and Aggregate Engineer within 60 days.)
_____
_____
By: _____ Date: _____

Enclosures: (Drilling notes, sketch, lab reports, other: _____)

NONEXCLUSIVE
MATERIAL PIT LEASE

Pit No. _____
County _____
District _____

Date:_____, 19_____

For and in consideration of the agreements set forth herein, _____
_____, as the Owners (whether singular or plural), hereby grant to the State of Minnesota, acting through the Commissioner of Transportation and its assigns, the right to enter upon and use the following described real property in _____ County, Minnesota, hereinafter referred to as the Pit:

of Section _____, T._____ N., R._____ W., of the _____th P.M.

This right of entry and use is for the purpose of investigating, testing, and removing therefrom earth, gravel and any or all materials required for use in the construction and maintenance of public highways for a period of two (2) years from the date stated above. At the option of the State of Minnesota, this lease may be renewed for ____ additional two (2) year period(s) after the termination date, by giving written notice to the Owners at the above address, mailed not less than thirty days prior to the termination date.

It is understood between the parties hereto that the State of Minnesota may assign the rights herein without prior consent of the Owners. The Owners agree to permit such assignees to remove materials from the Pit in accordance with the terms and provisions of this agreement.

The Owners hereby grant to the State of Minnesota and its assigns, the rights of ingress to and egress from the Pit by right of way across other lands of the Owners. Said right of way shall follow such course as is reasonably necessary for convenient access to the Pit. Said course may be improved and maintained by the State of Minnesota or its assigns in such a manner and at such times as may be reasonably necessary for the transportation of equipment and materials to and from the Pit.

Payments for materials taken under this agreement shall be based on records kept by the Commissioner of Transportation. Payment shall be made by the State of Minnesota, upon satisfactory proof of marketable title, free and clear of all encumbrances, at the following rates:

Common Borrow	Granular Borrow	Aggregate	Topsoil	Rock	Measure
					per cubic yard, loose volume
					per cubic yard, excavated volume
					per cubic yard, compacted volume
					per ton

The State of Minnesota, or its assigns, shall dispose of all waste materials accumulated by reason of its operations in removing the materials from the Pit as follows:

_____

_____

The Owners hereby release the State of Minnesota and its assigns from all claims whatsoever in the use of or damage to other lands and property of the owner resulting from the use of the Pit and the right of way when such use has been in accordance with the terms of this agreement.

OWNERS

_____

_____

_____

By:_____

_____

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

COMMISSIONER OF TRANSPORTATION

Director
Office of Right of Way and Surveys

STATE OF MINNESOTA)
) ss.
COUNTY OF)

The foregoing was acknowledged before me this _____ day of _____, 19_____, by _____.

_____
NOTARY PUBLIC

My commission expires: _____

STATE OF MINNESOTA)
) ss.
COUNTY OF RAMSEY)

On this _____ day of _____, 19_____, before me personally appeared _____ to me known to be the _____ Director of Office of Right of Way and Surveys acting for the Commissioner of Transportation of the State of Minnesota, and the person described in and who executed the foregoing instrument in behalf of the State of Minnesota and the Commissioner of Transportation, and acknowledged that he executed the same as the free act and deed of the State of Minnesota.

_____
NOTARY PUBLIC

My commission expires: _____

Approved as to form and execution
Office of the Attorney General

By:_____
For Attorney General

This instrument was drafted by the State of Minnesota Department of Transportation, Office of Right of Way and Surveys, St. Paul, Minnesota 55155

REQUEST FOR GRAVEL PIT SURVEY

Date: _____

TO: _____, District Surveys Engineer

FROM: Warren Pladsen, Aggregate Engineer
1400 Gervais Avenue, Maplewood, MN 55109

VIA: _____, District Materials Engineer

SUBJECT: SURVEY REQUEST - TYPE:

☐ New Source ☐ New Borings ☐ Condition

☐ R/W Corners ☐ Other _____

_____

SOURCE/PIT NO. _____, _____ County.

LOCATION: _____

Sec. _____ T _____ R _____

_____

REMARKS: _____

_____

_____

_____

ENCLOSURES: Drillers Sketch, gravel pit sheet, other _____

_____

==========

STATUS: _____

_____

_____

(Please notify Materials Engineer and Aggregate Engineer within 60 days if survey cannot be completed.)

MN/DOT AGGREGATE UNIT
ASIS Data Entry Form

Date Submitted: _____

ACTION:
___ Insert new record
___ Update old record
___ Delete old record

Source Number: _____ Status (A,I,N,Q,C): ___ Old Reference Number: _____

County: _____ District: _____ th Principal Meridian

Legal Desc.: _____

Sec.: _____ T _____ N R _____ W State Plane Coord.: X: _____ Y: _____

Owner/Operator: _____ Lease Exp. Date: _____

Concrete Source? (Y/N): _____ Date Computer File Revised: _____

Date Last Condition Survey: _____ Nearest City: _____

QUALITY INFORMATION

Shale -4; Lo: Hi: ___ Avg: ___ of: ___ LAR; Lo: ___ Hi: ___ Avg: ___ of: ___

Shale +4; Lo: Hi: ___ Avg: ___ of: ___ Litho Information (Y/N): _____

Total Shale: _____ Total Spall: _____ Absorption; Lo: ____ Hi: ___

Unsound Chert (Avg.): _____ of: _____ Iron Oxide (Avg.): _____ of: ___

QUANTITY INFORMATION

Mat. Class: _____ Quan. Est. (cu.yds.): _____ % Below Water: _____

Mat. Class: _____ Quan. Est. (cu.yds.): _____ % Below Water: _____

Stripping Ratio: _____

Geological Comments: _____

_____

_____

Source Name: _____

Process date: _____

Effective Date:
August 1, 1985
Revised May 31, 1994

AGGREGATE SOURCE INFORMATION SYSTEM (COMPUTER DATABASE)

- ASIS USERS GUIDE -

AGGREGATE SOURCES ARE ADDED TO THE ASIS DATABASE BY THE M&R AGGREGATE UNIT, IN COOPERATION WITH THE CENTRAL OFFICE R/W PROPERTY MANAGEMENT UNIT AS FOLLOWS:

I. Mn/DOT LEASED AGGREGATE SOURCES

A. New Source with New Lease

1. District R/W negotiates lease.
2. District R/W requests assignment of source number* from C.O. R/W (612-296-6891), or sends completed lease to C.O. R/W for source number assignment.
3. C.O. R/W checks records to ensure parcel is not presently assigned a number and then assigns a new source number.
4. C.O. R/W sends copy of lease to MR & E Aggregate.
5. MR & E Aggregate creates aggregate source record and enterslease information into ASIS database. (Additional information such as gradations, quality tests, etc., is entered into ASIS database by MR & E Aggregate when information is acquired.)

NOTE: Newly purchased parcels (new Mn/DOT-owned pits) and parcels with exclusive leases will have source numbers assigned, as above.

B. Previously Leased Sources

1. District R/W renegotiates lease and sends to C.O.R/W.
2. C.O. R/W sends copy of lease to MR & E Aggregate.
3. MR & E Aggregate enters revised lease expiration data into ASIS database.

- Since the inception of the ASIS, 5 digit numbering system in 1985, the Property Management Unit at Central Office R/W has had the sole responsibility for assigning new aggregate source numbers.

II. CONCRETE/COMMERICAL AGGREGATE SOURCES

A. Requests for assignment of source numbers to "new" concrete aggregate or commerical sources and rock quarries, which are not presently contained in the ASIS database, (or the COPES database), must be made to the Concrete Office at Maplewood MR & E. Contact Steve Babcock at (612) 779-5572.

B. Requests for concrete source numbers should include the following information:

1) County Name & nearest City
2) Owner's or operator's last name and first initial (or Company name)
3) Legal description
4) Pit or quarry name, if applicable

C. MR & E Concrete checks records to ensure commerical source or quarry is not presently assigned a number and then contacts C.O. R/W for new source number.

D. MR & E Concrete gives new source number to person making the request.

E. C.O. R/W sends information to MR & E Aggregate on "ASIS Data Entry Form."

F. MR & E Aggregate creates aggregate source record and enters commerical source number and information into ASIS database.

III. BORROW PITS

A. Borrow pits are assigned unique reference number by District R/W office at the time borrow pit lease is finalized.

B. Borrow pits are never assigned aggregate source numbers under the ASIS system.

C. Borrow pit numbers usually include the S.P. number for reference, followed by a number or letter designation (e.g., "S.P. 6953-00, Borrow Pit A.")

D. C.O. R/W provides copies of borrow pit leases to MR & E Aggregate on a routine basis, but no entry is made in the ASIS database by MR & E Aggregate.

E. If a borrow pit is subsequently reclassified as an aggregate source (by virture of type of material present), C.O. R/W then assigns a source number, and MR & E Aggregate enters information in ASIS database.

IV. ASIS DATABASE INPUT AND UPDATE

A. The ASIS database master file is maintained by MR & E Aggregate on an IBM PS/2, Mod. 60, w/hard disk.

Appendix A-6 (cont.)

B. Additions, revisions, and deletions to master file are made by MR & E Aggregate when new information becomes available, such as:

1) Aggregate source lease (newly leased)
2) Aggregate source purchase
3) Concrete aggregate source assigned a source number
4) Lease renewal (revised expiration data)
5) Condition survey
6) Test data - quality, quantity, etc.
7) Requests from database user (See D below)

C. MR & E Aggregate provides updated ASIS data files to District Materials/Soils Engineers, MR & E Concrete and Bituminous Offices, C.O. R/W and other active database users on a regular basis, or when requested, by network transfer of data files and/or ASIS source "log" printout. The ASIS source log is a summary printout, by County, of all aggregate source files in ASIS. Sources are listed numerically by source number, and information includes old reference number, legal description, last known source owner, lease expiration data and source status category.

D. District Materials/Soils and other database users may request additions or changes in the database by filling out the "ASIS Data Entry Form" and sending to MR & E Aggregate, or by sending a printout of the modified record with the changes 'hi-lited'.

A newer and faster method is to make a text file listing the pertinent data and copying it into the \HOME\GUEST\ASISDATA directory on the DOT-MRLAB server and notifying this office by phone. Minor changes in the data can also be relayed to MR & E Aggregate by a short note or phone call (612) 779-5611.

V. ADDITIONAL NOTES

A. The Concrete Office COPES - PIT database and the MR & E Aggregate ASIS database use similar, unique numbering systems. To convert a COPES number to an ASIS source number, merely drop the first digit, which is a one, from the COPES number and the numbers are then identical and refer to the same aggregate source.

B. County map sheets available from MR & E Aggregate show aggregate source (gravel pit) locations and numbers. Five (5)-digit aggregate source numbers replaced old reference numbers for gravel pits. If maps in your possession do not contain 5-digit source numbers, call MR & E Aggregate (612-779-5611) for replacements.

W. Pladsen

APPENDIX B

MINNESOTA'S DESIGN GUIDE FOR LOW VOLUME AGGREGATE SURFACED ROADS

MINNESOTA'S DESIGN GUIDE FOR LOW VOLUME AGGREGATE SURFACED ROADS

INVESTIGATION NO. 667

By
Terrance M. Beaudry

Minnesota Department of Transportation

July, 1992

Prepared for the

MINNESOTA LOCAL ROAD RESEARCH BOARD
OFFICE OF RESEARCH ADMINISTRATION
117 UNIVERSITY 2ND FLOOR
ST. PAUL, MINNESOTA 55155

The opinions, findings and conclusions expressed in this publication are those of the author and not necessarily those of the Minnesota Local Road Research Board of the Minnesota Department of Transportation.

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Executive Summary

The following Soil Factor Design Method was written for counties, townships, and municipalities for use in designing aggregate roads. Although many design methods for aggregate roads exist, most require thorough soil identification and soil strength testing. Since most local agencies do not have the means to perform elaborate testing of soils, and because they are familiar with soil factors, this design method should be helpful. This procedure requires soil identification, traffic counts, and rational judgement.

At present MN/DOT does not have a design method for aggregate roads. It is not the intention of this manual to change the present legal load limits of aggregate roads.

For those who would like a more thorough design method, or more background information on aggregate roads, this manual includes information on the use of the U.S. Forest Service Aggregate Surface Design Guide (ref. 1).

Also included is information regarding compaction, drainage, frost action, geosynthetic use, lime stabilization, and an appendix with sample problems based on the Soil Factor and Forest Service design methods.

Abstract

This Soil Factor Design Method was developed to assist counties, municipalities, and townships in designing aggregate roads. It was written for those who do not have the resources to use other more costly design methods. For those who would wish to use a more elaborate method, or would like more background information on aggregate roads, information on the use of the U.S. Forest Service design method has been included.

Introduction

This study was funded by the Local Road Research Board (LRRB). It is intended to be used as a guide for Minnesota counties and municipalities for the design of gravel roads. Included in this paper are:

* a Soil Factor Thickness Design Guide,
* background information on the use of the US Forest Service Aggregate Road Design Guide,
* information on compaction, drainage, geosynthetic use, and lime stabilization,
* an appendix with sample problems for the Soil Factor and Forest Service designs.

The purpose of this Soil Factor Design procedure is to give Minnesota counties, townships, and municipalities a simple method to calculate the required aggregate thickness for gravel roads, when it is not practical to perform soil strength tests. This design was developed by averaging thickness results and rational judgement from four design methods. These methods were the U.S. Forest Service (ref. 1), Mn/DOT Soil Factor (ref. 2), AASHTO (ref. 3), and Corps of Engineers (ref. 4). The results of these trials were then correlated to soil factors.

If possible, soil strength tests, such as California Bearing Ratio (CBR), Resistance value (R-value), pavement Dynamic Cone Penetrometer (DCP), or Falling Weight Deflectometer (FWD), should be performed prior to construction to determine the road carrying capacity of the subsoil (Appendix A contains a diagram of the pavement DCP apparatus). If tests are done, it is recommended that one use the US Forest Service (USFS) design method, because it should provide an economically efficient design. Even if one does not use this procedure, the USFS report contains considerable background information on aggregate roads, and therefore it is an excellent reference guide. To obtain this guide see the address found in the reference section.

At present MN/DOT does not have a design method for aggregate roads. It is not the intention of this manual to change the present legal load limits of aggregate roads.

Soil Factor Aggregate Thickness Design

The thickness requirement of this design is based on soil factors and traffic. Table 1 lists soil factors for three different soil classification systems: Mn/DOT, AASHTO, and Unified. Table 2 lists thickness requirements for different soil factors and traffic using two way ADT and HCADT.

To calculate the aggregate thickness, one first determines the soil type. The preferred method is to sample the soil. The sampling frequency should be as shown on Appendix B (ref. 5). If sampling is not practical, USGS maps or information from the SCS may be used to determine soil types. Local Mn/DOT materials units often have data on local Soil Factors and R-values.

Appendices C-G (ref. 5-8) are charts that will help define the soil type and relate strength parameters, for the Mn/DOT, AASHTO, and Unified soil classification systems.

Note: The USDA soil classification system is similar but different from the Mn/DOT soil system.

The next step is to determine the soil factor for the soil type under consideration. Table 1 lists soil factors for each soil classification system and soil type.

Once the soil factor and two way traffic data (ADT or HCADT) have been determined, use Table 2 to determine the thickness requirement. Thicknesses for a specific soil may vary because of the range in subsoil strength within a specific soil class. This variance is accounted for by the soil factors within this soil class. If one cannot obtain the strength of the subsoil and only has a thickness range given by soil factors, use the average thickness, as a minimum, from these soil factors for the design of the aggregate road.

This design recommends that the top layer should be Surface Class 1 material. For thicknesses of five inches or less, the entire lift should be Class 1 material. For thicknesses greater than five inches, use Class 5 or 6 material as a base for the Class 1 material. If Class 3 or 4 material is used as a base, the base thickness requirement should be increased by 33% over the Class 5 or 6 thickness. If Select Granular is used, the base thickness should be increased by 100% over the Class 5 or 6 thickness.

Note: Surface Class 1 requires 10-15% passing the No. 200 sieve.

Table 1

Soil Factors For Different Soil Classification Systems

Mn/DOT Soil Class	Soil Factor
Gravel (G)	50
Fine Gravel (FG)	50
Sand (S)	50
Coarse Sand (CS)	50
Loamy Sand (LS)	50–75
Fine Sand (FS)	50
Sandy Loam (SL) With 0–10% clay	50–75
Sandy Loam (SL) With 10–20% clay	100–130
Loam (L)	100–130
Silty Loam (SiL)	100–130
Clay Loam (CL)	100
Silty Clay Loam (SiCL)	100–130
Sandy Clay Loam (SCL)	100–130
Clay (C)	120–130
Silty Clay (SiC)	120–130
Sandy Clay (SC)	120–130

AASHTO Soil Class	Soil Factor
A–1–a	50
A–1–b	50–75
A–2–4	50–115*
A–2–5	50–115*
A–2–6	75–115*
A–2–7	75–115*
A–3	50
A–4	100–130
A–5	130+
A–6	100
A–7–5	120
A–7–6	130

*They are commonly mixed with A–4's in the field, and are difficult to isolate.

Unified Soil Class	Soil Factor
GW	50
GP	50
GM	50–75
GC	50–75
SW	50–75
SP	50–100
SM	75–100
SC	75–110
ML	100–130
CL	100–130
OL	130+
MH	110–130
CH	130
OH	130+

Notes: Soil strength tests should be performed and sampling should be done in accordance with Appendix B. Soil factors may be modified based on local experience.

Table 2
Thickness Design – Using Soil Factors

Soil Factor	Two Way Traffic (ADT)	Two Way Traffic (HCADT)	Surface Class 1 Thickness (inches)	Base Class 5 or 6 Thickness (inches)	Alt. Base Class 3 Thickness (inches)	Alt. Base Sel. Gran. Thickness (inches)
50	25	2	3	---	---	---
	50	4	4	---	---	---
	150	12	4	2	3	---
	400	32	4	3	4	---
	750	60	4	4	5	---
75	25	2	4	---	---	---
	50	4	5	---	---	---
	150	12	4	3	4	6
	400	32	4	4	5	8
	750	60	4	6	8	12
100	25	2	5	---	---	---
	50	4	4	2	3	4
	150	12	4	4	5	8
	400	32	4	5	7	10
	750	60	4	7	9	14
110	25	2	4	2	3	4
	50	4	4	3	4	6
	150	12	4	5	7	10
	400	32	4	6	8	12
	750	60	4	8	11	16
120	25	2	4	3	4	6
	50	4	4	4	5	8
	150	12	4	6	8	12
	400	32	4	7	9	14
	750	60	4	9	12	18
130	25	2	4	4	5	8
	50	4	4	5	7	10
	150	12	4	8	11	16
	400	32	4	9	12	18
	750	60	4	10	13	20

Notes: The suggested thicknesses found on this table assume that the soil is uniform and the road will be constucted at a moisture content in accordance with Mn/DOT Spec. 2105. The current (1991) Mn/DOT State Aid standards indicate that all C.S.A.H.'s with ADT's less than 150 should be aggregate surfaced.

If both ADT and HCADT are known, design the roadway by using the traffic parameter that gives the greater thickness.

Care should be taken when designing roadways with subsoil strengths that are highly variable. For example, from Table 1, the soil factor range of a Mn/DOT silty loam (SiL) subsoil is 100-130. From Table 2, for a two way ADT of 150, the total thickness requirements using Class 5 and Class 1 aggregate are: 8, 9, 10, and 12 inches for soil factors of 100, 110, 120, and 130 respectively. If feasible, the soil strength should be determined and a more thorough design procedure used. If soil strength data are not available, the thickness should be at least the average thickness within this range, 10 inches.

Note: The suggested thicknesses found on Table 2 assume that the soil is uniform, and the roadway will be constructed at a moisture content and density in accordance with Mn/DOT spec. 2105.

Note: Because of eventual rutting, loss of crown, and aggregate loss, periodic blading and additional aggregate placement is required. These maintenance activities are best determined at the local level.

USFS Design Using Subsoil Strengths

Several design methods exist for the aggregate thickness of gravel roads. Most methods require that one determine the subsoil strength. If one has access to soil strengths, it is recommended that one use the Forest Service method (ref 1). It is a simple to follow and an uncomplicated method, and there is a user friendly computer program included with the design. Inputs needed for the Forest Service method include:

- traffic (one way),
- subsoil and aggregate strength,
- rut depth,
- and aggregate loss.

Traffic is input in design lane (one way) Equivalent Single Axle Loads (ESAL's). ESAL's are based on the number of repetitions of a 18-kip axle load with a tire pressure of 80 psi. It is recommended that five to ten years of traffic be imputed into this program. If one does not have detailed traffic information to calculate the number of ESAL's, Equation 1, which relates design lane Average Daily Traffic (ADT) and design lane Heavy Commercial Average Daily Traffic (HCADT) to design lane yearly ESAL's, may be used. The distribution of traffic used to derive

Equation 1 came from the tables found in Appendices H.1 and H.2 (ref. 5). The traffic distribution used was for local rural CSAH and municipal roads. To obtain the total traffic multiply the yearly ESAL's by the design life and by the yearly growth factor.

Equation 1

100 Design Lane ADT = 8.3 Design Lane HCADT = 882 Design Lane Yearly ESAL's.

The next information needed for the USFS method is the California Bearing Ratio (CBR) of the aggregate and the subsoil. Because the Forest Service method uses CBR to rate the soil and aggregate, one should perform this test as opposed to other strength tests. However, for those performing other strength tests, Appendix I contains equations relating resilient modulus to CBR and R-value, and equations relating CBR to pavement DCP (ref. 3 and 9-11).

A soil's CBR value is defined as the force required to cause a standard cylindrical piston to penetrate at a given rate into the soil. The CBR is the ratio between this measured force and a standard force. Standardized procedures for conducting the CBR test are described in ASTM Standard D 1883 or AASHTO Standard T 193.

The pavement DCP is a simple to use soil strength testing instrument. The initial cost of the pavement DCP is reasonable, and one can quickly evaluate a section of roadway. Because the DCP is an economical test to perform and its data correlates well with other soil strength tests, its use should increase.

The other two parameters to set are the aggregate loss and the rut depth. According to the Forest Service report, the aggregate loss is usually set at one inch and the rut depth at two inches.

Appendix J contains sample problems using the Forest Service and Soil Factor Design methods. The first two examples compare the required thicknesses using the Soil Factor and Forest Service methods with the same input.

Note: The confidence level of using the equations in Appendix I to obtain CBR values for use in the Forest Service design is unknown.

Note: When performing field soil strength tests, it is critical that they are analyzed at times when the soil is at its lowest strength. This will usually occur in the spring, when the subgrade is saturated. If tests are run at other times of the year, one must compensate for the higher values.

Compaction

Good compaction is the most important requirement when building an aggregate road. The aggregate should be placed and compacted in accordance with Mn/DOT spec. 2211. This guide assumes that the subsoil and aggregate are well compacted. The CBR will decrease from 25 to 50 percent for each 5 percent decrease in T-99 compaction below 95% (ref. 12). The decrease is greatest for cohesive soils and least for granular soils.

The following example was obtained by imputing the following parameters into the Forest Service design program, and is an illustration of the benefits of compacting subsoils. It shows a three inch increase in the required aggregate for compaction decreasing from T-99 = 95% to T-99 = 85%.

Example:

A subsoil is cohesive and has a CBR of 10 at 95% T-99. Design lane traffic is 500 ADT or 4410 yearly design lane ESAL's. Five years of traffic are input, for a total of 22,050 ESAL's. The CBR of the aggregate is 40. Aggregate loss is one inch, and rut depth is two inches.

Compaction (% T-99)	Soil CBR	Thickness Required (inches)
95	10	7.3
90	5.0	8.6
85	2.5	10.3

Drainage

Another important design attribute is drainage. Whenever possible, ditches should be designed 4-6 feet below the roadway crown, with the crown at least five feet above the normal water table.

Although not intended to be covered in detail herein, wet subgrade soils, spring areas, or frost heave areas should generally be corrected by the appropriate use of perforated subsurface drains, proper grade and crown, deeper ditches, or replacement with non-frost susceptible soils.

Frost Action

To minimize frost heaving and thaw weakening, it is essential that the finished grade elevation is placed an adequate distance above the water table. This distance should be at least equal to

the depth of frost penetration. For silty soils, the distance should be significantly greater (Table 7-503H Mn/DOT Road Design Manual).

Geosynthetic use in Aggregate Roads

Current practice suggests that one may decrease aggregate thickness through the judicial use of geosynthetics (geotextiles and geogrids). Many publications exist on the use of geosynthetics in the design of aggregate roads, and four are included in the reference section (ref. 13-16). While most geotextile suppliers have "designs" for aggregate reduction, these should always be independently verified.

Note: All aggregate reduction designs require that the geotextile be "rutted" to develop tension in the fabric and that the ruts be subsequently filled with new aggregate, not regraded aggregate.

Lime Stabilization

Lime stabilization may be used to improve a soil's plasticity and workability. Lime stabilization is most beneficial on clay soils, but its success is dependent on many variables including: soil and lime types, lime percentage, temperature, and moisture. More detailed information on lime stabilization may be found from references 17 and 18.

Using lime stabilization to enhance a soil's strength should be approached with some caution. One Minnesota study on clay soils found that the R-Value rose sharply after an initial treatment with lime, but after being subjected to freeze-thaw cycles, the soil's strength decreased (ref. 19 and 20). Consequently, the pavement cracked severely, which increased maintenance costs. Another unpublished study seems to indicate an accelerated ride deterioration with lime treated subgrade on a bituminous road (ref. 21).

Conclusions

The Soil Factor design method included in this report was written for counties, townships, and municipalities for use in designing aggregate roads. Although this Soil Factor Design Method will be helpful to these agencies, rational judgement and experience in identifying soil types, determining soil strengths, and thickness requirements should not be discounted. The required thicknesses assume that the soil is uniform and constructed at a moisture content in accordance with Mn/DOT spec. 2105. In subgrades that

have variable soils, or where soil strengths of the same soil type vary, one should use particular design judgement.

For those who would like a more thorough design method, or more background information on aggregate roads, it is recommended that one obtain a copy of the U.S. Forest Service Aggregate Surface Design Guide.

By designing for drainage and against the effects of frost action, the performance of aggregate roads will be enhanced.

Acknowledgments

The following people helped structure and critique this manual.

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Gerald Teig (Mn/DOT Physical Research)

Mark Gieseke (Mn/DOT State Aid)

Tim Petersen (Minnesota DNR Bureau of Engineering)

Tom Burnham (Mn/DOT Physical Research)

References

1) Whitcomb, W.G., Yapp, M.T., and Myers, M., Aggregate Surface Design Guide, U.S. Department of Agriculture Forest Service.

APPENDICES

14) Geotextile Design and Construction Guideline, FHWA, Pub. # FHWA-HI-90-001, 1989 (Edited version of ref. 13, with some construction related additions.

15) Steward, Williamson, and Mohney, Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads, FHWA, Report No. FHWA-TS-78-205, 1977.

16) Koerner, R.M., Designing with Geosynthetics, Prentice Hall, second edition, 1990.

17) National Lime Association, Lime Stabilization Construction Manual, Bulletin 326, 1987.

18) Transportation Research Board, Lime Stabilization, State of the Art Report 5, 1987.

19) Wolfe, R.E. and Allen J.R., Laboratory Evaluation of Lime Treatment of Typical Minnesota Soils, Minnesota Department of Highways, Investigation No. 601, 1964.

20) Mikkelsen, V.V., Hale, J.C., and Holm, L.J., Experimental Lime Stabilization Project -Norman County-, Minnesota Department of Highways, Investigation No. 611, 1971.

21) Baker, H.B., unpublished report titled "Subgrade with Organic Material".

To Obtain a Copy of the Forest Service Design Write to:

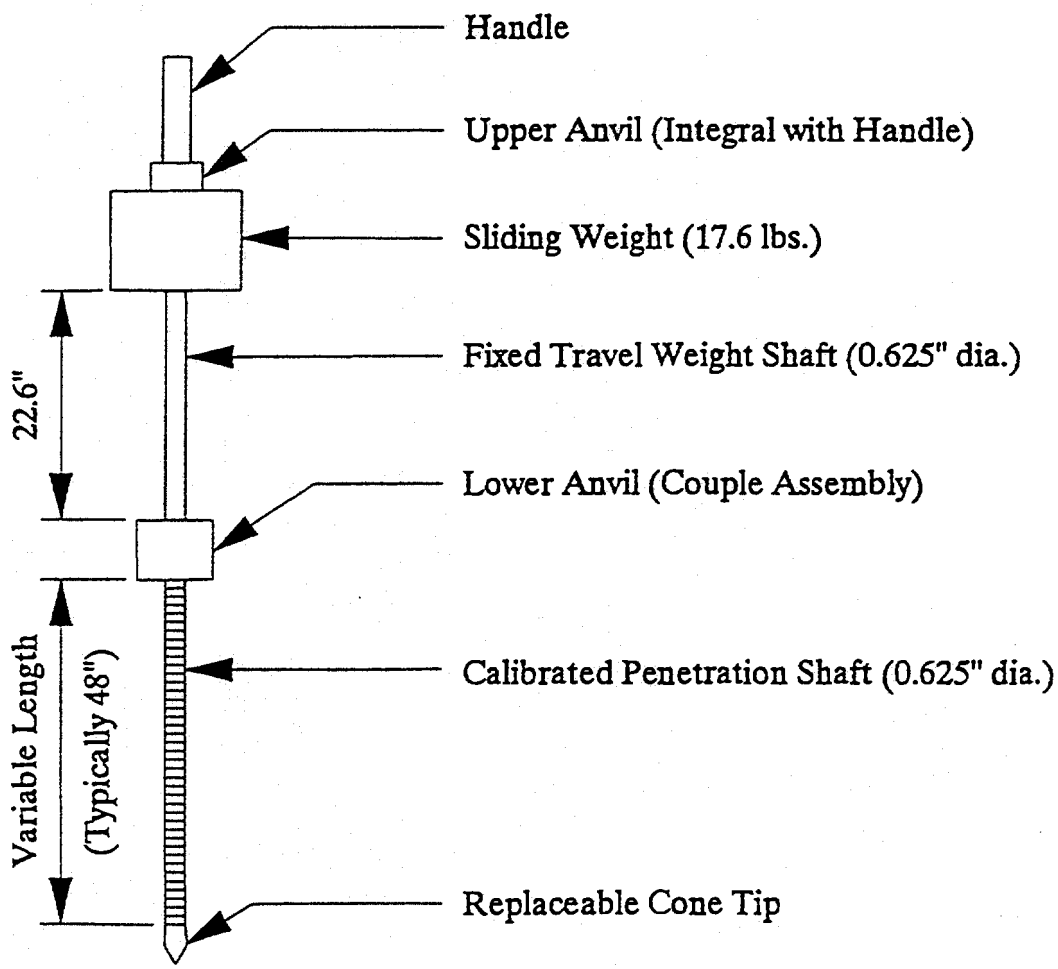
USDA - Forest Service
333 SW 1st Avenue
P.O. Box 3623
Portland, Oregon 97208.

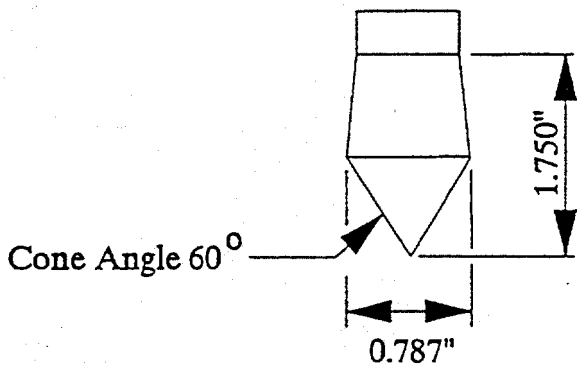
Ask for: Whitcomb, W.G., Yapp, M.T., and Myers, M., Aggregate Surface Design Guide, U.S. Department of Agriculture Forest Service, J669-Final Report, February 1990.

APPENDICES

Appendix A

"Pavement" Dynamic Cone Penetrometer





Cone

Appendix B*

Stabilometer R–Value Sampling Frequency Guidelines for Mn/DOT Soil Types

For cases where R–value samples will be obtained,
the following table may be used as a guide in sample frequency.

Major Soil Texture**	Recommended Minimum Sampling Rate	Minimum Number of Samples
Sands	0 (assume a value of 70 or 75)***	0***
Clays, Clay Loams	1 every two miles	3****
Sandy Loams (nonplastic to slightly plastic)	3 per mile	5
Silt Loams	3 per mile	5
Silty Clay Loams	3 per mile	5
Plastic Sandy Loams	3 per mile	5
Sandy Clay Loams	3 per mile	5

*Table 7–5.03G from Mn/DOT Road Design Manual, January 31, 1982.

**Major soil texture refers to a soil texture significant enough in areal extent to economically justify a change in pavement design.

*** If % passing #200 exceeds 15%, then sample and select a design R–value in the same manner as for clay, clay loams. This means that a sufficient number of gradation checks of the sand areas will have to be made to determine if Stabilometer tests are required.

**** Given significant local experience, this may be reduced to 1 or 2 samples.

NOTE: Samples should be representative of the upper 4 feet of the proposed road grade as much as possible. In other words, in unbalanced jobs concentrate on the borrow sources; in balanced jobs concentate on the cuts. If practical, resample the embankment after construction.

Appendix C*

Mn/DOT SOIL Classification Identification

Textural Class	Identification by "Feel"	Ribbon Length	AASHTO Group (H.R.B. Class)	Group Index	Rating For Upper Emb.
Gravel (G)	Stones: Pass 3" sieve, Retained on #10	0	A-1-a	0	Excellent
Fine Gravel (FG)	Stones: Pass 3/8" sieve, Retained on #10	0	A-1-a	0	Excellent
Sand (S)	100% pass #10. Less than 10% silt & clay.	0	A-1-a	0	Excellent
Coarse Sand (CS)	Pass #10, Retained on #40	0	A-1-a or A-1-b	0	Excellent
Fine Sand (FS)	Most will pass #40. Gritty-non plastic	0	A-1-b or A-3	0	Excellent to Good
Loamy Sand (LS)	Grains can be felt, Forms a cast.	0	A-2-4 or A-2-5	0	Excellent to Good
Sandy Loam (SL)					
a. slightly plastic	0-10% clay. Gritty	0-1/2"	A-2-6 or A-2-7	0	Excellent to Good
b. Plastic	10-20% clay. Gritty	1/2"-1"	A-4	0-4	Excellent to Good
Loam (L)	Gritty, but smoother than SL.	1/4"-1"	A-4	0-4	Excellent to Good
Silt Loam (SiL)	Smooth, slippery or velvety. Little resistance.	0-1"	A-4	0-4	Fair to Poor
Clay Loam (CL)	Smooth, Shiny, considerable resistance.	1"-2"	A-6	0-16	Good to Fair
Silty Clay Loam (SiCL)	Dull appearance, slippery, less resistance.	1"-2"	A-6 or A-5	0-16	Fair to Poor
Sandy Clay Loam (SCL)	Some what gritty. Considerable resistance.	1"-2"	A-6 or A-5	0-16	Good to Fair
Clay (C)	Smooth, shiny, long thin ribbon.	2"+	A-7	0-20	Fair to Poor
Silty Clay (SiC)	Buttery, smooth, slippery.	2"+	A-7 or A-7-5	0-20	Poor
Sandy Clay (SC)	Very plastic but gritty. Long thin ribbon.	2"+	A-7 or A-7-6	0-20	Fair to Poor

Note: Where the group index is expressed as a range, such as 0-16, the lower values are the better foundation soils.

*Table A 5-692.607 from Mn/DOT Grading & Base Manual, April 7, 1978.

Appendix D*

Mn/DOT Soil Classification System

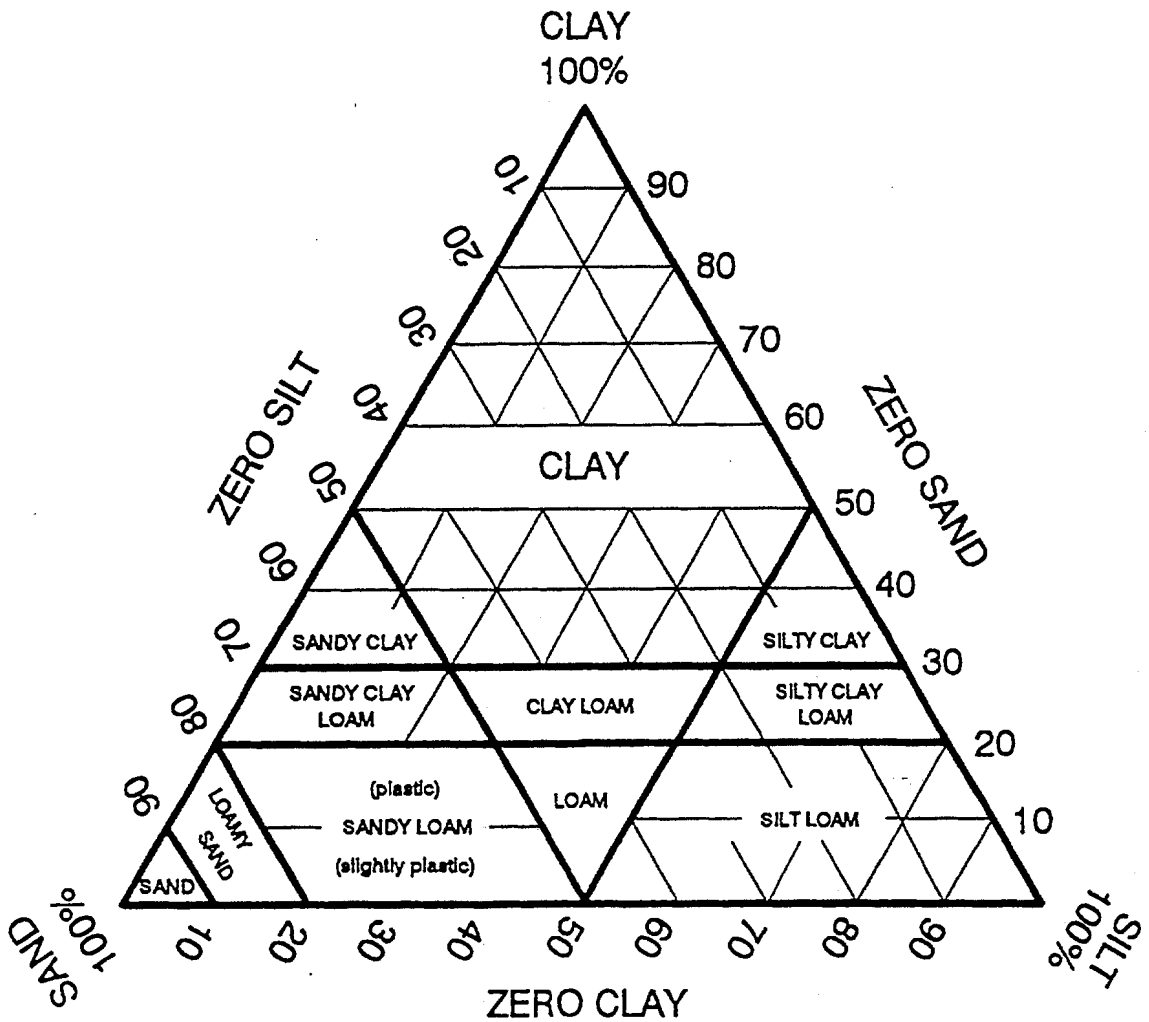


*Figure A 5-692.611 from Mn/DOT Grading & Base Manual. May 1, 1959.

Appendix E

AASHTO Classification of Soils and Soil–Aggregate Mixtures

General Classification	Granular Materials (35% or less passing No. 200)							Silt–Clay Materials (more than 35% passing No. 200)			
	A–1		A–3*	A–2				A–4	A–5	A–6	A–7
Group Classification	A–1–a	A–1–b		A–2–4	A–2–5	A–2–6	A–2–7				A–7–5 A–7–6
Sieve Analysis:											
Percent Passing:											
No. 10	50 max.	----	----	----	----	----	----	----	----	----	----
No. 40	30 max.	50 max.	51 max.	----	----	----	----	----	----	----	----
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of Fraction passing No. 40:											
Liquid Limit	----	----	----	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity Index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.**
Group Index	0		0	0		4 Max.		8 Max.	12 Max.	16 Max.	20 Max.
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

*The placing of A–3 before A–2 is necessary in the "left to right elimination process", and does not indicate superiority of A–3 over A–2.

** Plasticity index of A–7–5 subgroup is equal to or less than LL minus 30.
Plasticity index of A–7–6 subgroup is greater than LL minus 30.

Stabilometer R–Values for AASHTO Soil Types

AASHTO Soil Type	Textural	Assumed R–Value	Comments
A–1–a	Sands Gravels	75	Excellent confidence in using assumed value.
A–1–b	Sands Sandy Loams (non–plastic	70	If percent passing number 200 sieve is 15 to 25 percent, R–value may be as low as 25. In such cases, it is highly desirable to obtain laboratory R–values.
A–2–4 & A–2–6	Sandy Loams (non–plastic, slightly plastic, or plastic)	30 (70 for LS and LFS)	Loamy Sands and Loamy Fine Sands commonly have R–values of 70. Laboratory R–values range from 10–80 for the entire A–2 classification. It is highly desirable to obtain laboratory R–values, for the Sandy Loams.
A–3	Fine Sands	70	Excellent confidence in using assumed value.
A–4	Sandy Loams (plastic) Silt Loams Silty Clay Loams Loams Clay Loams Sandy Clay Loams	20	Laboratory R–values range from 10 to 75. It is highly desirable to obtain laboratory R–values.
A–6	Clay Loams Clays Silty Clay Loams	12	Laboratory R–values commonly occur between 8 and 20.
A–7–5	Clays Silty Clays	12	Data available is limited.
A–7–6	Clays	10	Laboratory R–values commonly occur between 6 and 18.

Note: In using the above assumed R–value for bituminous pavement design, it is essential that the subgrade be constructed of uniform soil, and at a moisture content and density in accordance with Mn/DOT Spec. 2105, and be capable of passing test rolling, Mn/DOT Spec. 2111. To minimize frost heaving and thaw weakening, it is also essential that finished grade elevation be placed an adequate distance above the water table. This distance should be at least equal to the depth of frost penetration. For silty soils, this distance should be significantly greater.

*Table 7–5.03H from Mn/DOT Road Design Manual, January 31, 1982.

Appendix G*

Laboratory Criteria for Unified Soil Classification System

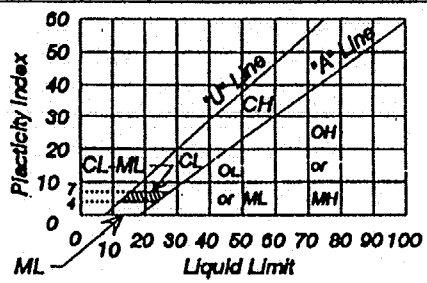
					Laboratory Classification Criteria		GROUP SYMBOLS	TYPICAL NAMES
Coarse Grained Soils Less than 50% passing No. 200 sieve	Sands and Gravels	Gravels (Less than half of the coarse fraction passes the No. 4 sieve size)	Less than 5% passes the No. 200 sieve size	Borderline cases requiring the use of dual symbols (i.e. SW-SC)	$C_u = \frac{D_{60}}{D_{10}} > 4$; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between 1 & 3		GW	Well graded gravels, gravel-sand mixtures, little or no fines
					Not meeting both C_u and C_c requirements above		GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
			More than 12% passes the No. 200 sieve size		Above "A" line with PI between 4 and 7 requires the use of dual symbols (GC-GM)	Atterberg limits below "A" line, or PI<4	GM	Silty gravels, gravel-sand-silt mixtures
						Atterberg limits above "A" line, and PI>7	GC	Clayey gravels, gravel-sand-clay mixtures
		Sands (More than half of the coarse fraction passes the No. 4 sieve size)	Less than 5% passes the No. 200 sieve size	5% to 12% passing the No. 200 sieve size	$C_u = \frac{D_{60}}{D_{10}} > 6$; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between 1 & 3		SW	Well graded sands, gravelly sands, little or no fines
					Not meeting both C_u and C_c requirements above		SP	Poorly graded sands, gravelly sands, little or no fines
			More than 12% passes the No. 200 sieve size		Above "A" line with PI between 4 and 7 requires the use of dual symbols (SC-SM)	Atterberg limits below "A" line, or PI<4	SM	Silty sands, sand-silt mixtures
						Atterberg limits above "A" line, and PI>7	SC	Clayey sands, sand-clay mixtures
Fine Grained Soils More than 50% passing No. 200 sieve	Silts and Clays	Go to Plasticity Chart	 Plasticity Chart				ML	Inorganic silts & v fine sands, rock flour, silty or clayey f sands w/ slight plasticity
							CL	Inorg clays of low to med plasticity gravelly, sandy, silty, or lean clays
							MH	Inorg silts, micaceous or diatomaceous, fine sandy or silty soils, elastic silts
							CH	Inorg clays of high plasticity, fat clays
							OL	Organic silts & organic silt-clays of low plasticity. LL<50
			Below "A" line and $\frac{\text{LL (oven dry soil)}}{\text{LL (air dry soil)}} < 0.75$				OH	Organic clays of med to high plasticity LL>50
			Visual Identification only				Pt	Peat or other highly organic soils.

*Table 2-1 from "Basic Soil Mechanics Workbook", USDA SCS, January 1981

Appendix H.1

Assumed Distribution Factors By Vehicle Type (Table 7–5.03B)

Vehicle Type	Description	Rural Truck Highway % of AADT	Metro % of AADT	Local Rural and CSAH % of AADT**
1	Passenger Cars	78.1	83.5	75.7
2	Panels and Pickups (under 1 ton)	10.0	9.0	16.0
3	Single Unit – 2 axle, 4 tire	1.4	1.6	2.4
4	Single Unit – 2 axle, 6 tire	3.9	1.8	2.6
5	Single Unit – 3 axle & 4 axle	1.3	0.5	1.7
6	Tractor Semitrailer Combination – 3 axle	0.3	0.3	– –
7	Tractor Semitrailer Combination – 4 axle	0.5	0.4	0.1
8	Tractor Semitrailer Combination – 5 axle	3.0	2.4	0.5
9	Tractor Semitrailer Combination – 6 axle	*	*	*
10	Trucks with Trailers and Buses	1.5	0.5	1.0

* Too few to establish a value at this time.
** Data for local roads is from 1975 and 1977 County Roads Pilot Project, and these should not be used in preference to current seasonally adjusted classification counts.

This is Table 7–5.03B From Mn/DOT Road Design Manual January 31, 1982, reference 5.

Appendix H.2

Average N18 Factors By Vehicle Type (Table 7–5.03D)

Vehicle Type	Description	Rural T.H. N18 Factor	Metro N18 Factor	Local Rural CSAH and Municipal N18 Factors	Range		
					Max. Legal 10–Ton	Measured	
						Max.	Min.
1	Passenger Cars	0.0004	0.0004	0.0004	––	0.0008	0.0003
2	Panels and Pickups (under 1 ton)	0.007	0.007	0.007	3.0	0.012	0.0006
3	Single Unit – 2 axle, 4 tire	0.01	0.01	0.01	3.0	0.070	0.003
4	Single Unit – 2 axle, 6 tire*	0.24	0.22	0.21	3.0	0.61	0.019
5	Single Unit – 3 axle & 4 axle****	0.41	0.57	0.45	2.61	1.40	0.015
6	Tractor Semitrailer Combination – 3 axle	0.58	0.21	0.15	2.20	2.45	0.028
7	Tractor Semitrailer Combination – 4 axle	0.53	0.41	0.30	2.62	3.91	0.060
8	Tractor Semitrailer Combination – 5 axle	0.88	0.63	0.59	2.20	4.10	0.028
9	Tractor Semitrailer Combination – 6 axle	***	***	***	––	––	––
10	Trucks with Trailers and Buses**	0.42	0.42	0.34	––	––	––

* Use 0.60 for 2 axle garbage trucks.
** Use 1.25 for MTC buses.
*** Too few to establish a value at this time.
**** Use 0.91 for sugar beet trucks.

This is Table 7–5.03D from Mn/DOT Road Design Manual January 31, 1982, reference 5.

Appendix I

Note: The confidence level of using these equations to obtain CBR values for use in the Forest Service design is unknown to the author.

Equations Relating Resilient Modulus to R-value and CBR

1) M_R (psi) = 1500 x CBR.

2) M_R (psi) = A + (B x R-value).

3) M_R (psi) = 1000 + (555 x R-value).

Equation 1 is from Heukelom and Klomp. This correlation was made using dynamic compaction for CBR, and the insitu modulus of the soil. The data from which this correlation was developed ranged from 750 to 3000 times CBR.

Equation 2 is from the Asphalt Institute, where A = 772 to 1155 and B = 369 to 555.

Equation 3 is from AASHTO, and they state that this equation can be used for fine grained soils (R-values less than or equal to 20).

Equations Relating DCP to CBR*

1) Log CBR = 2.56 - 1.16 Log DCP, for cohesive soils and DCP > 10mm/blow.

2) Log CBR = 2.70 - 1.12 Log DCP, for granular soils and DCP < 10mm/blow.

3) Log CBR = 2.81 - 1.32 Log DCP, for combined data.

* From Harison, J.A., Correlation of CBR and Dynamic Cone Penetrometer Strength Measurement of Soils. Australian Road Research 16 (2), June 1986.

Appendix J

USFS and Soil Factor Design Examples

Note: Examples 1 and 2 use both methods with the same input. Examples 3 and 4 use the Soil Factor Method; Examples 5 and 6 use the Forest Service Method.

Example 1: Using Soil Factor Design

Subsoil is Mn/DOT Clay Loam, R = 12, CBR = 2.5. The two way measured traffic is 40 ADT and 4 HCADT. Since 4 HCADT gives a greater thickness, the HCADT should be the traffic parameter used. From Table 1, the soil factor is 100.

Using Table 2:

Thickness (Inches)		
Surface Class 1		**Base Type**
4.0"	+	2.0" Class 5 or 6 = 6.0" Total Aggregate, or
4.0"	+	3.0" Class 3 or 4 = 7.0" Total Aggregate, or
4.0"	+	4.0" Select Granular = 8.0" Total Aggregate.

In all cases a surface of 4 inches of Surface Class 1 should be used. If Class 5 or 6 is used, the base should be 2 inches. If Class 3 or 4 is used, the base should be 3 inches. If Select Granular is used, the base should be 4 inches.

Example 1: Using Forest Service Design

Subsoil is Mn/DOT Clay Loam, R = 12, CBR = 2.5, Soil Factor = 100. Two way traffic is 40 ADT and 4 HCADT. The Forest Service method uses one way traffic, therefore one way traffic is 20 ADT and 2 HCADT. Again the traffic parameter one should use is HCADT. 2 HCADT = 213 yearly ESAL's (from equation 1).
Use 10 Year design = 10 years x 213 ESAL's/Year = 2130 ESAL's.
Base = Class 5 or 6 (CBR = 80), or Class 3 or 4 (CBR = 45), or Select Granular (CBR = 30).
Surface = Surface Class 1.
Rut Depth = 2 inches.
Aggregate loss = 1 inch.

Thickness (Inches)		
Surface Class 1		**Base Type**
4.6"	+	0.0" Class 5 or 6 = 4.6" Total Aggregate, or
3.0"	+	3.3" Class 3 or 4 = 6.3" Total Aggregate, or
4.0"	+	4.5" Select Granular = 8.5" Total Aggregate.

Appendix J-2

Example 2: Using Soil Factor Design

Subsoil is granular, AASHTO Class A-1-a, R = 60, CBR = 25.
Measured two way traffic is 150 ADT.
From Table 1, the soil factor is 50.

Using Table 2:

Thickness (Inches)		
Surface Class 1		Base Type
4.0"	+	2.0" Class 5 or 6 = 6.0" Total Aggregate, or
4.0"	+	3.0" Class 3 or 4 = 7.0" Total Aggregate.

In both cases a surface of 4 inches of Surface Class 1 should be used. If Class 5 or 6 is used, the base should be 2 inches. If Class 3 or 4 is used, the base should be 3 inches.

Example 2: Using Forest Service Design

Subsoil is granular R = 60, CBR = 25, Soil Factor = 50.
Two way traffic is 150 ADT, therefore one way traffic is 75 ADT = 662 yearly ESAL's. Use 10 Year design = 6,620 ESAL's.
Base = Class 5 or 6 (CBR = 80), or Class 3 or 4 (CBR = 45).
Aggregate loss = 1 inch.

Thickness (Inches)		
Surface Class 1		**Base Type**
4.0"	+	0.0" Class 5 or 6 = 4.0" Total Aggregate, or
4.7"	+	0.0" Class 3 or 4 = 4.7" Total Aggregate.

Example 3: Using Soil Factor Design

Subsoil is AASHTO Class A-4.
Measured two way traffic is 75 ADT and 12 HCADT. Since 12 HCADT gives a greater thickness, the HCADT should be the traffic parameter used. From Table 1, the soil factor varies from 100-130, therefore, if one does not have any soil strength information, use the average thickness for soil factors 100, 110, 120, and 130.

Appendix J-3

Using Table 2:

	Thickness (Inches)			
	Surface	Base Type		
Soil Factor	Class 1 +	Class 5 or 6 or	Class 3 or 4 or	Sel Gran.
100	4	4	5	8
110	4	5	7	10
120	4	6	8	12
130	4	8	11	16
Average	**4**	**6**	**8**	**12**

In all cases a surface of 4 inches of Surface Class 1 should be used. If Class 5 or 6 is used, the base should be 6 inches. If Class 3 or 4 is used, the base should be 8 inches. If Select Granular is used, the base should be 12 inches.

Example 4: Using Soil Factor Design

Subsoil is Mn/DOT Sandy Loam (SL) with 0-10% clay.
Two way traffic is 750 ADT and 50 HCADT. Since 750 ADT gives a greater thickness, the ADT should be the traffic parameter used. From Table 1, the soil factor varies from 50-75, therefore one should take the average thickness for soil factors 50 and 75.

Using Table 2:

	Thickness (Inches)			
	Surface	Base Type		
Soil Factor	Class 1 +	Class 5 or 6 or	Class 3 or 4 or	Sel Gran.
50	4	4	5	-
75	4	6	8	12
Average	**4**	**5**	**7**	**10**

In all cases a surface of 4 inches of Surface Class 1 should be used. If Class 5 or 6 is used, the base should be 5 inches. If Class 3 or 4 is used, the base should be 7 inches. If Select Granular is used, the base should be 10 inches.

Example 5: Using Forest Service Design

Base = Class 5 or 6 (use CBR = 60).
One way traffic = 500 ADT = 4410 Yearly ESAL's.
Use 5 Year Design = 22,050 ESAL's.
Subsoil CBR = 15.
Rut Depth = 2 inches.
Aggregate loss = 1 inch.
Thickness = 5.2 inches (use 5.2 inches of Class 1 only).

Appendix J-4

Using 10 Year Design = 44,100 ESAL's
Thickness = 5.6 inches (use 3 inches of Class 1 with 2.6 inches of Class 5 or 6).

Example 6: Using Forest Service Design

Base = Class 3 or 4 (use CBR = 45).
One way traffic = 8 HCADT = 882 Yearly ESAL's.
Use 5 Year Design = 4410 ESAL's.
Subsoil CBR = 12.
Rut Depth = 2 inches.
Aggregate loss = 1 inch.
Thickness = 5.0 inches (use 5 inches of Class 1 only).

Using 10 Year Design = 8820 ESAL's
Thickness = 5.5 inches (use 3 inches of Class 1 with 2.5 inches of Class 5 or 6).

APPENDIX C

DESIGN RECOMMENDATION/SOIL LETTER RESOLUTION

MINNESOTA DEPARTMENT OF TRANSPORTATION
Operations and Technical Services Divisions
Technical Memorandum 91-43-ME-20
October 17, 1991

TO: Distribution 57

FROM: P. C. Hughes
Acting Assistant Commissioner
Operations

E. E. Ofstead
Assistant Commissioner
Technical Services

SUBJECT: Design Recommendation/Soil Letter Resolution

I. INTRODUCTION:

An AOD/ADE Subcommittee of the Relationship Committee met with its Materials Engineers' Group to identify issues which needed clarification. The committee felt that an issue that needed to be addressed was that the recommendations which are provided by the Technical Service Divisions' materials specialists, by means of inserts to the District Soils Letters, and the final resolution of these recommendations were not being effectively communicated to all parties involved.

In view of these findings, a task force was formed and given the following charges:

1. To recommend a process for resolving any differences between the original District Soils Letter recommendations and the Materials & Research Specialist inserts. In addition, identify a method to clearly communicate that agreement to all involved, i.e., Final Specialists and Design Engineers.

2. To review the policy as to which projects require Soils Letters and recommend appropriate changes and/or clarifications to the same.

The task force, in turn, accomplished the following:

- Developed procedures which will ensure that any Design Recommendation/Soils Letter inserts as provided by the Materials Specialists are agreed upon by the District Soils and/or Materials Engineers to the maximum extent possible and so signified on the letter prior to returning the letter back to the District.

- Provided a vehicle to resolve differences where mutual agreement cannot be achieved between the District and the Materials Specialists.

- Developed a position/policy on which types of projects Soils Letters will be provided for informational purposes only and concurrence by signature will not be required.

PURPOSE:

The purpose of this Technical Memorandum is to implement the recommendations of the task force. Therefore, the following shall be adhered to:

1. Design Recommendation/Soils Letter shall be prepared for all projects, except as noted otherwise, to document the proposed design concepts and construction procedures.

2. To effectively address concerns relative to Soil Letter inserts and to communicate to all parties (i.e., Final Plans, Special Provisions, Material and Research Specialists and Design Engineers) involved in the utilization of the Letter that the inserts are mutually agreed upon; the letters for major construction, reconstruction and/or rehabilitation projects shall be processed as follows:

   a. The District will submit the Design Recommendation/ Soils Letters to the Pavement Design Engineer, Materials & Research Laboratory.

   b. The letter will be reviewed by the respective Materials Specialists. Recommendations or changes/modifications to the letter will be documented by inserts. The District Materials/Soils Engineers will be contacted by the Materials Specialists when major issues are involved.

   c. Except for minor issues, inserts will be sent by mail to the District Soils/Materials Engineer for review, concurrence and resolution of any disagreement.

   d. The District Soils and/or Materials Engineers will place their initial by the inserts they concur with.

   e. In the event the District Soils and/or Materials Engineers do not agree with an insert, they will cross out the insert and contact the appropriate party. Letter g, below, will be followed if an agreement with the Materials Specialist(s) cannot be reached.

   f. The District Soils/Material Engineers will send the inserts back to the Pavement Design Engineer.

   g. In the event that resolution of differences is unlikely, resolution will be achieved in accordance with Technical Memorandum No. 91-18-TS-01 (Inter-Divisional Issue Resolution), dated April 10, 1991.

   h. Upon resolution of differences copies of the signed letter with the initialed and/or crossed out inserts will be sent out by the Pavement Design Engineer to the addressee and individuals on the cc list.

3. That Design Recommendation/Soils Letters should be prepared for minor construction, rehabilitation and maintenance projects. For those minor projects, the letter would be sent to the Materials Specialists for informational purposes only. Concurrence would not be required by the Specialists; however, the Specialists would be free to comment on the letter and make suggestions/recommendations to the Soils/Materials Engineer.

Minor projects would include:

- Concrete rehabilitation projects.
- Surface Treatment Program ($40,000/mile overlay).
- Turn lanes (when only a few are involved).
- Culvert replacement.
- Intersection revisions (other than radii work).
- Other minor projects as agreed upon by the District Materials/Soils Engineer and the Pavement Design Engineer at the Materials & Research Laboratory.

4. The District Soils/Materials Engineer shall submit addendums to document major changes in design concepts and construction methods for projects for which the Design Recommendation/Soils Letters have been submitted previously. Also, when aggregate source prospecting is still unresolved at the time the Letter is written, it may be necessary for the District Soils/Materials Engineer to add sources by addendum.

NOTE: In the future, Soils Letters will be referred to as Design Recommendations Letters.

NOTICE:

TM 91-43-ME-20 is hereby made Appendix C of the Geotechnical and Pavement Manual. Termination dates are no longer in force.

APPENDIX D

UNBONDED CONCRETE OVERLAY

STATE OF MINNESOTA
Office Memorandum

DEPARTMENT : Transportation
Office of Materials Research and Engineering

DATE : January 15, 1993

TO : Darryl E. Durgin
Deputy Commissioner/Chief Engineer
Engineering and Operations Bureau

FROM : Richard H. Sullivan
Director
Office of Materials Research and Engineering

PHONE : (612) 779-5590

SUBJECT : Unbonded Concrete Overlay Policy

Attached for your approval is the "POLICY ON UNBONDED OVERLAYS RELATING TO PAVEMENT SELECTION." In summary, it allows the district to submit an Unbonded Overlay as an alternative to reconstruction, particularly to avoid subgrade problems.

It is the recommendation of our committee that this policy be implemented.

Richard H. Sullivan, Chair

Patrick C. Hughes

John E. Sandahl

Gerald J. Rohrbach

Approved: Darryl E. Durgin

POLICY ON UNBONDED OVERLAYS RELATING TO PAVEMENT SELECTION

The initial step is to determine the feasibility of an Unbonded Overlay. An Unbonded Overlay (new concrete over an existing concrete pavement) is not feasible if one of the following conditions is met:

1) **Short segments** - Projects often have areas which require regrading or pavement removal due to:

   a) Clearance at bridges
   b) Alignment (vertical or horizontal) changes
   c) Subgrade corrections

   When these areas become a significant portion of the overall project, the use of an unbonded overlay between these segments should be discouraged.

2) **Urban design** - In urban areas, with curb and gutter design, unbonded overlays should not be used.

If the Unbonded overlay is feasible and the project is programmed in the Reconstruction or Major Construction category, the district may submit the Unbonded Overlay as an alternate design. The district should state this desire clearly in their submittal.

The Unbonded Overlay and interim fixes will be designed by the Pavement Design Engineer. The interim fixes should be that of a joint seal at year 15, and a thin overlay at year 30. This thin overlay is given a life of 5 years, to get the same 35 year analysis period for all alternatives.

The Estimating unit will then do their analysis of the five alternatives, including both initial costs and interim costs.

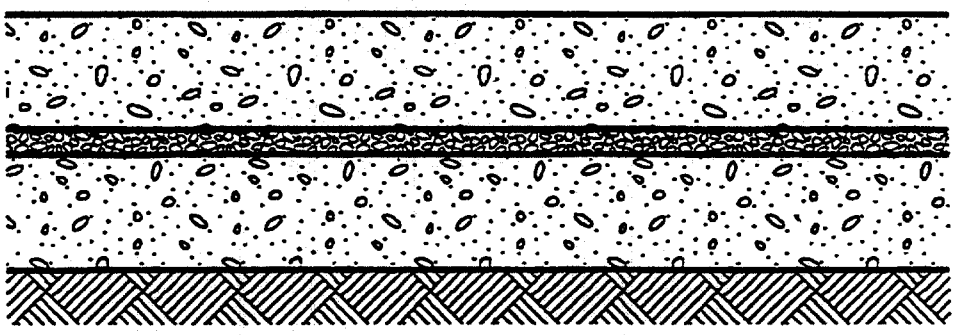
The general policy of the Pavement Selection Committee will be:

- If the low is Unbonded, and the second low is Rigid, the district may be given the option of the Unbonded or the new Rigid.

- If the low is Unbonded, and the second low is one of the bituminous options, the unbonded overlay will be mandatory.

Unbonded Concrete Overlay

Design Procedures



Minnesota Department of Transportation
Office of Materials Research & Engineering
Pavement Engineering Section
October 1993

MINNESOTA DEPARTMENT OF TRANSPORTATION
UNBONDED CONCRETE OVERLAY DESIGN PROCEDURE

10/5/93

An unbonded JPCP (Jointed Plain Concrete Pavement) overlay of an existing JPCP, JRCP (Jointed Reinforced Concrete Pavement) or CRCP (Continuously Reinforced Concrete Pavement) pavement can be placed to improve both structural capacity and functional condition.

Feasibility

An unbonded overlay is a feasible rehabilitation alternative for PCC (Portland Cement Concrete) pavements for practically all conditions. However, a PCC unbonded overlay would not be considered feasible under the following conditions where:

1. The amount of deteriorated slab cracking and joint spalling is not large and other alternatives such as CPR would be much more economical.

2. Vertical clearance of bridges is inadequate for required overlay thickness. This may be addressed by reconstructing the pavement under the overhead bridges or by raising the bridges. Thicker unbonded overlays may also necessitate raising signs and guardrails as well as flattening side slopes and extending the culverts.

3. The existing pavement is susceptible to large heaves or settlements.

4. Alignment changes of significant length are involved, resulting in short segments of unbonded overlays. (However, rather than making an arbitrary decision, a cost analysis should be performed as an aid in determining the feasibility of constructing an unbonded overlay under this type of condition.)

5. The existing pavement is an urban design with curb and gutter.

6. The existing pavement is 20 feet or less in width.

Pre-Overlay Repairs

One major advantage of an unbonded overlay is the amount of repair to the existing pavement prior to overlay is minimized. However, unbonded overlays are not intended to bridge localized areas of nonuniform support. The following distress types should be reviewed and repaired prior to placement of the overlay so as to prevent reflection of cracks that may reduce its service life:

Distress Type	Repair*
Working crack	- No repair needed
Badly broken/shattered slabs/ high deflection (FWD)	- Full-depth concrete repair (Non-reinforced and non-doweled)
Spalled joints	- Patch with bituminous mixture
Pumping	- Edge drains
Settlement	- Level-up with bituminous mixture or mud jack
Poor Joint/Crack Load Transfer	- No repair needed; if pavement has many joints or cracks with poor load transfer, consider a thicker stress relief layer and crack and seat.
Unstable slabs, poor load transfer, extensive D-cracking	- Crack and seat (entire project)
Punchouts, potholes, extensive D-cracking	- Remove loose material and fill with bituminous patching mixture
Severe Frost Heaves	- Subgrade correction and concrete repair
Badly Faulted	- Consider a thicker stress relief layer

- These are suggested guidelines. Each distress type should be evaluated and the appropriateness of the suggested repair should be determined and modified/changed, if necessary, to satisfy the conditions at hand.

Design Methods

The design concept for unbonded overlays is based on providing an overlayed pavement such that the base slab plus the unbonded resurfacing is structurally equivalent to a new concrete pavement designed to carry the anticipated future traffic. This implies that the resurfaced pavement will provide a service life that is equivalent to that provided if a new concrete pavement were to be constructed.

There are basically three widely accepted unbonded overlay design methods. They are the Corps of Engineers' (COE), Portland Cement Association (PCA) and AASHTO methods. The AASHTO method, which is presented in 1993 "AASHTO Guide for Design of Pavement Structures," is not included with these design procedures at this time. The COE and PCA methods are as follows:

A. Corps of Engineers (COE) Design Method
(This method was empirically developed.)

$$D_{OL} = \sqrt{D_N^2 - C(D_E)^2}$$

D_{OL} = required unbonded PCC overlay thickness. (inches)

D_N = required new PCC pavement thickness to carry future traffic. (inches)

D_E = thickness of existing pavement. (inches)

C = Coefficient depending on the structural condition of the existing pavement determined by visual inspection. The practice has been to use the following values for C; however, other values can be used.

C = 1.0 Existing pavement is in good overall structural condition with little or no cracking.

C = 0.75 Existing pavement has initial joint and corner cracking due to loading but no progressive structural distress or recent cracking.

C = 0.35 Existing pavement is badly cracked or shattered.

B. PCA Method

This procedure was mechanistically derived and is based on results of pavement analyses conducted using the finite element computer program JSLAB.

The stress data used to prepare the design charts for determination of unbonded overlay thickness were developed by using the program JSLAB. These charts are applicable to existing concrete pavements that have effective modulus of elasticity values ranging from about 3,000,000 to about 4,000,000 psi.

Design charts are presented for three cases of existing pavement condition. These cases are as follows:

Case 1. Existing pavement exhibits a large amount of midslab and corner cracking with poor load transfer at joints and cracks.

Case 2. Existing pavement exhibits a small amount of midslab and corner cracking. It exhibits reasonably good load transfer at the joints and cracks. Localized repairs were performed to correct distressed slabs.

Case 3. Existing pavement exhibits a small amount of midslab cracking and good load transfer at the cracks and joints.

The design chart for Case 1 was developed using data from analyses of overlay sections containing a crack in the existing pavement directly under an edge load on the overlay. The design chart for Case 3 was developed using data from the analysis of overlay sections with no cracking in the existing pavement. The design chart of Case 2 was developed through interpolation between Case 1 and Case 3 conditions. The design charts are given in Figures 1, 2, and 3 for Cases 1, 2, and 3; respectively.

The first step in the design process involves determination of the thickness of a new concrete pavement that would be needed for the anticipated subgrade soil conditions and future traffic.

Design charts in Figures 1, 2 and 3 are then used to compute the thickness required for the unbonded overlay.

Representative values of unbonded overlay thickness determined from Figures 1, 2, and 3 are summarized in Table 1. These thicknesses are listed for different values of existing pavement thicknesses and equivalent new concrete pavement thickness. The determination of the actual overlay thickness is influenced by the condition and thickness of the existing base pavement and the equivalent new thickness.

To compare results given by the PCA design charts with the Corps of Engineers design equations for unbonded overlays, overlay thicknesses were computed for values of the pavement condition coefficient, C, of 0.35, 0.6, and 0.8. These results are presented in Table 2. It is seen that the overlay thicknesses listed in Table 2 agree closely with those listed in Table 1. This indicates that the characterization of the existing pavement in terms of Cases 1, 2, and 3, as recommended in the design procedure, closely simulates existing pavement condition coefficient, C, values of 0.3 to 0.5, 0.5 to 0.7, and 0.7 to about 0.9, respectively.

In comparing these two design methods, it can be observed that the ratings (Cases 1, 2 and 3 and C factors) involved a certain degree of subjectivity. Because of the subjectivity involved, various engineers may rate a pavement differently and thereby obtain different thickness for the overlay.

Mn/DOT Overlay Design Procedure

The design process will consist of the following:

A. Existing pavement evaluation.

1. Pavement condition survey.
   Utilize the PMS data relative to ride and distresses. The type, extent and severity of the distresses should be identified. The pavement should be divided into analysis sections based on pavement design, construction history, traffic usage and location.

2. Existing Pavement.
   a) Type of pavement (JPCP, JRCP, CRC)
   b) Slab thickness & width
   c) Type of load transfer
   d) Type of shoulder (tied, PCC, other)

3. FWD Testing.
   It is strongly recommended that FWD load testing be performed, if possible, on all existing concrete pavements where an unbonded concrete overlay is considered as a possible rehabilitation alternative.

   FWD load testing should be especially considered where there exists significant load and material related distresses. The load testing should be carried out to determine the severity of the problem and the pavement and subgrade soil materials characteristics/ properties. The load testing should be conducted at joints and working cracks to determine the relative deflection across the joints and cracks and to estimate the joint and crack load transfer efficiency. In addition, FWD loads should be applied to the interior of the concrete panels so as to determine the pavement's elastic modulus and subgrade soils elastic modulus and the modulus of subgrade reaction. Results of the load testing can be used to better determine the case and C factors to be used for unbonded overlay design, verification/backcalculation of layer moduli, and new pavement thickness design.

B. Traffic Analysis

In accordance with current policy, unbonded overlay thickness designs will be based on anticipated cumulative 30-year design lane rigid ESALs.

The 30-year design lane rigid ESALs (CESALs) should be determined as follows:

$$CESAL_{30} = CESAL_{20} + 10[\frac{CESAL_{35} - CESAL_{20}}{15}]$$

$CESAL_{20}$= Rigid ESAL's shown for the Design Year in the Office of Transportation Data Analysis, Traffic Forecast and Analysis Sections "Cumulative ESAL Report."

If the design year is moved back a year or two, a new design year ESALs should be determined for $CESAL_{20}$ and the $CESAL_{35}$ in accordance with the following:

$CESAL_{35R} = CESAL_{35} + (CESAL_{20R} - CESAL_{20o})$

$CESAL_{20R}$ = Revised Design Year Traffic

$CESAL_{20o}$ = Original Design Year Traffic

$$CESAL_{30} = CESAL_{20R} + 10\ [\frac{CESAL_{35R} - CESAL_{20R}}{15}]$$

Example:

Base Year 1993

	Year	CESALs
20 year	2013 (Design Year-Traffic Report)	15,017,000
	2014	15,312,000
	2015	15,608,000
	-	-
	-	-
	-	-
35 year	2028	29,538,000

1. Design Year 2013

$$CESAL_{30} = 15{,}017{,}000 + 10\ \frac{(29{,}538{,}000 - 15{,}017{,}000)}{15}$$

$CESAL_{30} = 24{,}698{,}000$

2. Design Year moved 2013 to 2015

$$CESAL_{35R} = 29{,}538{,}000 + (15{,}608{,}000 - 15{,}017{,}000)$$

$CESAL_{35R} = 30{,}129{,}000$

3. Design Year 2015

$$CESAL_{30} = 15{,}608{,}000 + 10\ \frac{(30{,}129{,}000 - 15{,}608{,}000)}{15}$$

$CESAL_{30} = 25{,}289{,}000$

C. Subgrade Support (Modulus of subgrade reaction - (k.))

1. Design R-Value (Average minus one standard deviation)
2. $k = 1.17 + 63\sqrt{R}$

D. New Concrete Pavement Thickness - D_N

Use PAVE program (based on 1972 AASHO, Revised 1981) with following parameters:

1. Design R-Value
2. 30-year design lane traffic - $CESALS_{30}$ x 0.93 (Modified for frozen subgrade effect.)
3. $P_t = 2.5$
4. Modulus of rupture - 500 psi
5. Modulus of elasticity - 4,200,000 psi
6. J-Factor
   1) J = 2.6 for 27-foot wide pavement
   2) J = 3.2 for 24-foot wide pavement

E. Overlay Thickness

1. COE Method

$$D_{OL} = \sqrt{D_N^2 - C(D_E)^2}$$

D_N - From PAVE program (do not round)
D_E - existing slab thickness
C - Coefficient previously selected (Badly D-Cracked pavements use C = 0.5, unless FWD load testing indicates that the load transfer across joints and cracks is adequate, then use C=0.65)

2. PCA Method

Use design charts in Figures 1, 2, and 3 to determine the required unbonded overlay thickness.
(Badly D-Cracked pavements use Case 1, unless FWD load testing indicates that the load transfer across joints and cracks is adequate, then use Case 2.)

3. Design Thicknesses

a. Use both methods and average the results.
b. Round the average result in accordance with the current rounding procedures* and use this value for the design thickness. However, the minimum design thickness should be in accordance with the following:

MINIMUM UNBONDED CONCRETE OVERLAY THICKNESS

Overlay Width (ft.)

		27	24
Existing Pavement Width (ft.)	27	6"	**
	24	7"	6"
	22	**	7"***
	20	**	**

* Rounding Procedure
Thickness is x.0 to x. 1, round to x.0
x. 2 to x.6, round to x.5
x. 7 to x.9, round to (x+1).0

** Not appropriate design

*** Unless recommended otherwise by MR&E's Pavement Engineering Section.

F. Other Design Appurtenance/Considerations

1. Pavement Type

All unbonded concrete pavement will be designed as jointed plain concrete pavements (JPCP) unless otherwise determined by MR&E's Pavement Engineering Section.

2. Transverse Contraction Joints

All transverse joints shall be as follows:

a. All transverse joints shall be doweled, using 1-inch dowels for unbonded overlays less than 9 inches thick, 1.25 inch dowels for overlay thicknesses of 9 to 10.5 inches and 1.5 inch dowels for overlays greater than 10.5 inches in thickness.
b. Unbonded overlays which are 7 inches but less than 10.5 inches in thickness and overlays that are 10.5 inches or greater in thickness use a 15-foot and 20-foot uniform joint spacing, respectively; except as noted in d.
c. Unbonded overlays less than 7 inches, contact the MR&E's Concrete and Pavement Design Units for the appropriate joint spacing and dowel bar size.
d. Do not locate transverse joints within 3 feet of an existing transverse joint or working crack. The most effective joint location is to place the overlay joint on the approach side of an existing joint or crack. This allows the leave slab to bridge the joint or crack and avoid cantilever deflections and pumping action beneath the existing slab (APCA).
e. Joints in the overlay shall be either skewed or non-skewed in accordance with the joint layout of the underlying PCC pavement. (The joints in the unbonded pavement and the inplace pavements shall be parallel.)
f. All transverse joints should be C4E-D and should conform to Figure 4 ($g = {}^{t}/_{3}$).
g. Dowel basket assembly detail and anchoring procedures should be in accordance with Figure 5.
h. Transition from unbonded overlay to on grade pavements near bridges and other removal areas and associated supplemental steel detail should be in accordance with Figures 6 and 7.

3. Joint Sealant

a. Use silicone or other materials as approved by the Concrete Unit (MR&E) for sealing transverse joints.
b. Use hot pour (3720) for longitudinal joints.

4. Edge Rumble Strips

Provide on 27-foot wide pavement but omit on 24-foot wide pavements.

5. Stress Relief Layer

Provide a 1-inch minimum thickness stress relief layer between the unbonded overlay and the existing pavement. The material used for the layer should be in accordance with (2331) Type 31 BIT MIX FOR PERMEABLE STRESS RELIEF COURSE (SP 4-107). This layer should be 30 and 33 feet wide for 24 and 27 foot wide overlays, respectively; extending out from the concrete pavement edge 3 feet on each side or as needed to support the paver tracks (Figure 9). If the inplace pavement is badly faulted and/or has a rough profile due to cracked panels, D-Cracking, curling, etc., the stress relief layer should be increased in thickness (to 2± inches) and placed in two lifts.

6. Interceptor Drain

Provide interceptor drains when the overlay pavement is wider than the existing underlying pavement. The drains should be in accordance with Figures 8 & 9.

7. Pavement Edge Drain

Provide edge drains for all pavements regardless of subgrade soil type in accordance with Figure 9. Edge drains are to be provided, even for granular subgrade type soils, since the stress relief layer consists of a permeable stabilized material and therefore an outlet for positive drainage must be provided.

8. Extensive D-Cracking

Provide for the removal of unconfined and/or loose deteriorated concrete at the pavement joints, cracks and patches by air blasting and power sweeping. Air blasting shall utilize a nominal 100 psi pressure as directed by the Engineer. Depressions, which result from air blasting and/or power sweeping, at the transverse and longitudinal joints and cracks shall be patched with Bituminous Patching Mixture 2231 and compacted in accordance with the Ordinary Compaction Method (2331) with a pneumatic-tired roller. The patching of these areas shall be accomplished prior to paving the stress relief layer.

9. Shoulder Area Soils

If the overlay design involves cantilevering the pavement over the inplace pavement, the soils in the shoulder area beneath the overhang should be investigated. The investigation should focus on textural classification, density, and moisture and whether the shoulder heaves during the winter months. This area should contain non-frost susceptible soils. If this is not the case, the inplace soils should be excavated and replaced with non-frost susceptible material to the bottom of the inplace concrete pavement or to the bottom of any aggregate base under the concrete pavement, whichever is deeper. If frost susceptible soils are allowed to remain in place and the shoulder heaves, this may cause the pavement to crack. In addition, even if the soil doesn't heave, it should be replaced with non-frost susceptible soil so as to provide firm uniform support.

10. Transition Areas (bridge ends and other removal areas)

In transitional areas, which are located in slow drainage areas (i.e. at bridge ends or pavement ends on steep grades), and which are exposed to a significant number of heavy construction loads; the lower 2-3 inches of the PASSRC shown in Figure 6 (right side) may be replaced with dense graded bituminous mixture (i.e. 31BB50000Y or 41BB50055Y), if necessary in order to achieve satisfactory stability. The thickness of remaining PASSRC should not be less than the PASSRC placed under the adjacent unbonded concrete pavement.

11. Unbonded Overlay Cantilevering

In some cases to avoid of cantilevering the unbonded overlay, it may be desirable to widen the inplace narrow concrete pavement with bituminous or concrete.

The appropriateness of these types of widenings should be discussed with the Pavement Engineering section prior to incorporating into the design.

G. Design Example

1. Design Information

a. Existing Roadway
- Four-lane divided highway
- 22 years old

b. Existing Pavement
- 9.0-inch JRCP
- 5.0-inch granular subbase
- 40-foot joint spacing
- 24-feet wide
- dowelled joints

c. Roadbed Soils
- Clay loam
- Design R-Value = 15
- k = 240 psi

d. Major Distresses
- Mid-panel cracks
- Joint deterioration (severely D-Cracked)
- PSR = 2.7
- PQI = 2.6

e. Traffic
$CESAL_{30}$=24,698,000 from previous example (page 7)

f. Proposed Design
- 15-foot joint spacing
- all joints dowelled
- 27-foot wide pavement

2. New Pavement Design

2. New Pavement Design

- Design R-Value - 15; k = 240 psi
- J = 2.6
- M_r = 500 psi
- E = 4,200,000 psi
- P_t = 2.5
- $CESAL_{30}$ = 24,698,000 x 0.93* = 22,969,000
  - * 0.93 - frozen subgrade effect

PAVE program results in a new full-depth pavement thickness of 9.7 inches - D_N.

3. Overlay Thickness

a. COE Method

$$D_{OL} = \sqrt{D_N^2 - CD_E^2} = \sqrt{9.7^2 - 0.5(9.0)^2} = 7.3 \ inches$$

(D_N, C & D_E, defined on Page 3.)

b. PCA Method
Figure 1 (Case 1), D_{OL} = 8.2 inches

Average of Methods (7.3 + 8.2) ÷ 2 = 7.8 inches

Design Pavement 8.0 inches thick

The above design procedures were derived from:

Tayabji, S.D. and P.A. Okamoto, "Thickness Design of Concrete Resurfacing," present at the Third International Conference on Concrete Pavement Design and Rehabilitation, April 1985.

FHWA's "Techniques for Pavement Rehabilitation" developed by ERES Consultants, Inc.

Hutchinson, R.L. "Resurfacing with Portland Cement Concrete," NCHRP Synthesis of Highway Practice 99, Transportation Research Board, December 1992.

Darter, M.I., R.P. Elliot, K.T. Hall, "Revision of AASHTO Pavement Overlay Design Procedures," April 1992.

"AASHTO Guide For Design of Pavement Structures," 1993

Table 1 Representative Values of Unbonded PCC Overlay Thickness

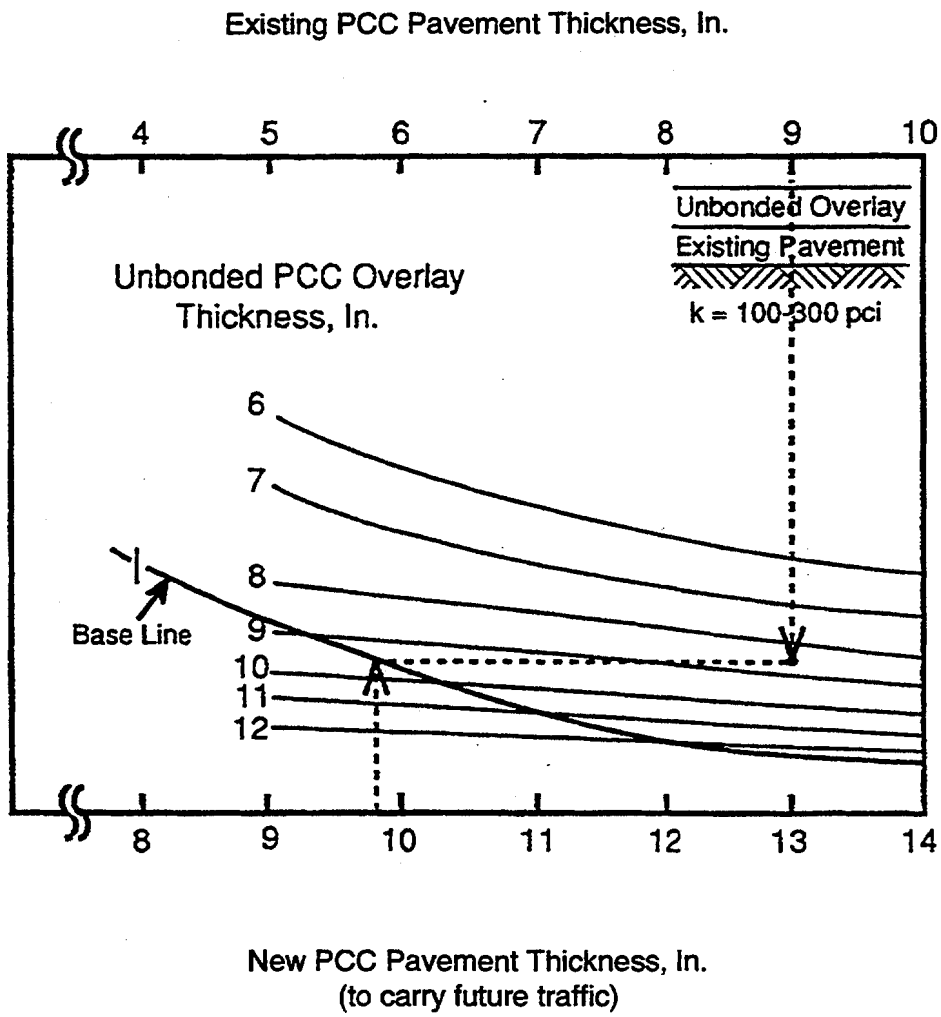
t_n in.	t_e in.	Overlay Thickness, in.		
		Case 1	Case 2	Case 3
8	8	6.8	(6.0)	(6.0)
	7	7.2	6.0	6.0
	6	7.6	6.8	6.0
10	9	9.0	7.0	(6.0)
	8	9.2	7.8	6.0
	7	9.4	8.8	8.0
12	9	11.5	10.4	9.0
	8	11.7	10.8	10.0
	7	11.8	11.2	10.8

Notes: t_n = equivalent new PCC pavement thickness (to carry future traffic)
t_e = existing pavement thickness
Cases 1, 2, and 3 refer to condition of existing pavement described in the text.
Values in parentheses indicate minimum thickness requirement of 6 in.

Table 2 Unbonded PCC Overlay Thickness Determined Using Corps of Engineers' Equation

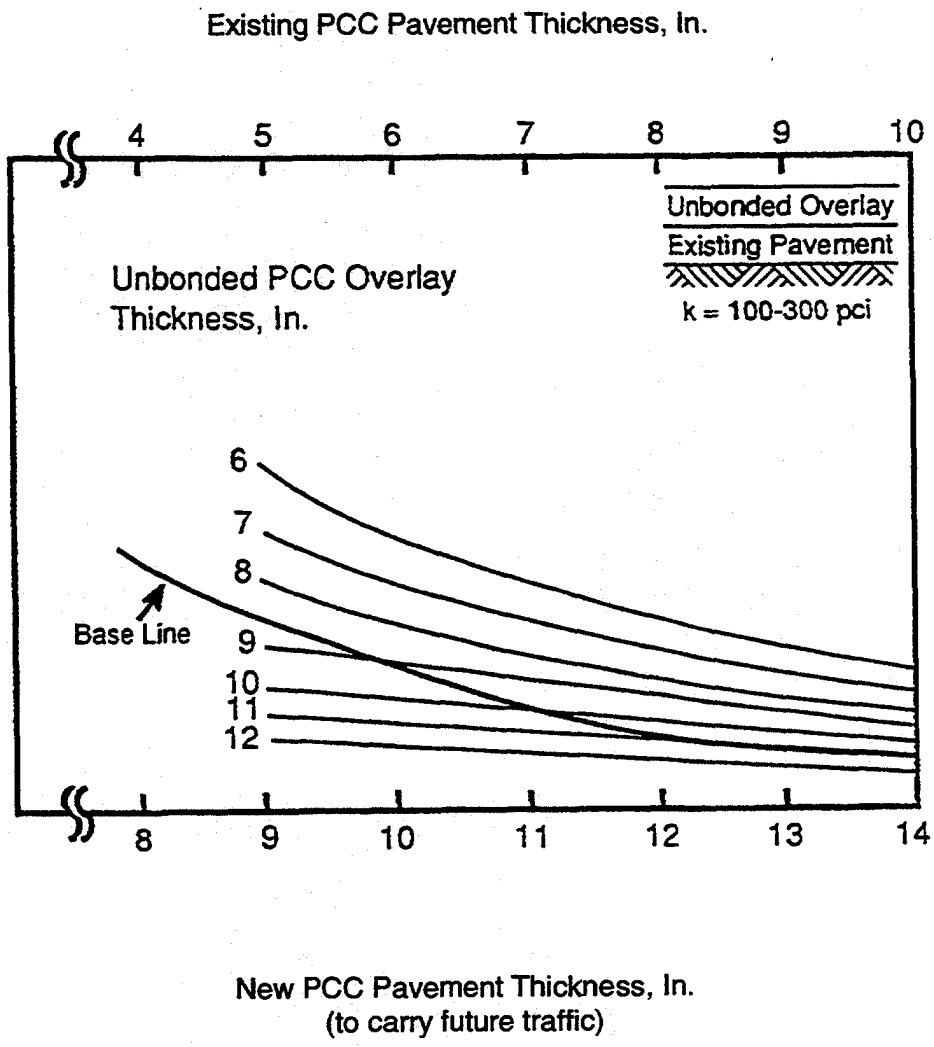
t_n in.	t_e in.	Overlay Thickness, in.		
		C = 0.35	C = 0.6	C = 0.8
8	8	6.5	(6.0)	(6.0)
	7	6.8	(6.0)	(6.0)
	6	7.2	6.5	6.0
10	9	8.5	7.2	6.0
	8	8.5	7.9	7.0
	7	9.1	8.4	7.8
12	9	10.8	9.8	8.9
	8	11.0	10.3	9.6
	7	11.3	10.7	10.2

Notes: t_n = equivalent new PCC pavement thickness (to carry future traffic)
t_e = existing pavement thickness
C = pavement condition coefficient
Values in parentheses indicate minimum thickness requirement of 6 in.



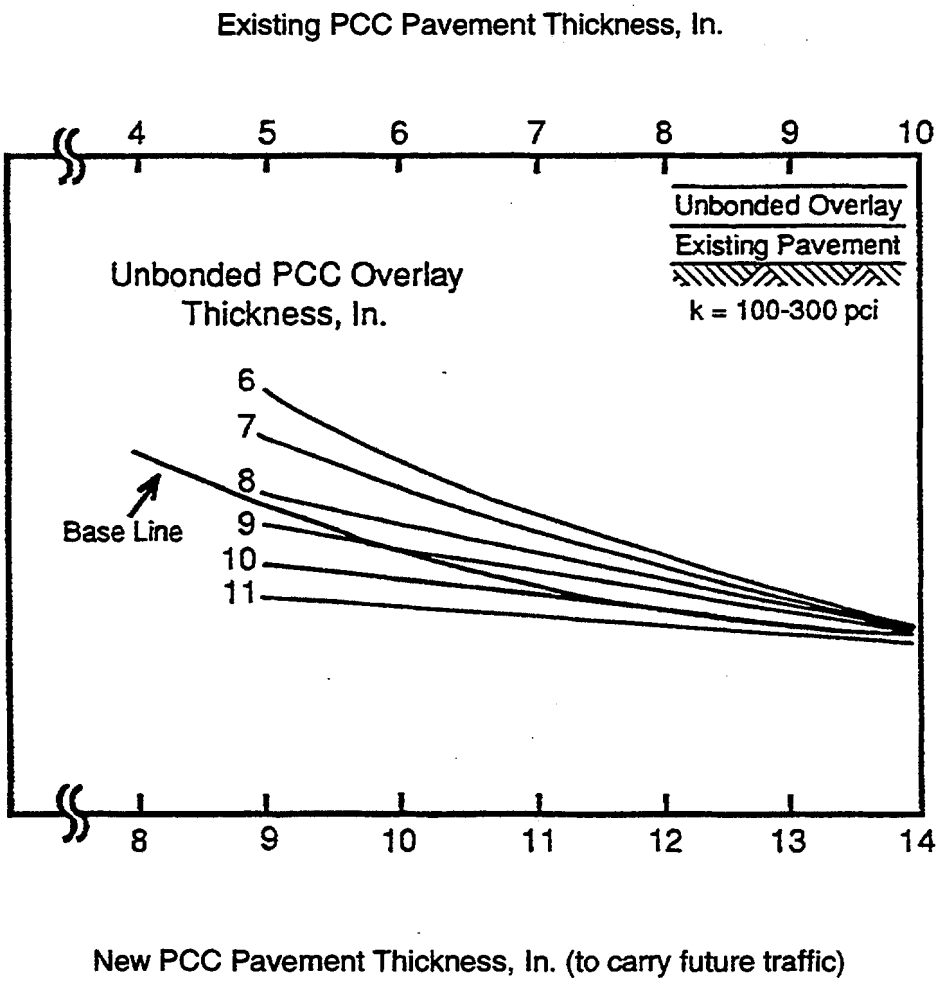
Case 1: Existing pavement exhibits a large amount of midslab and corner cracking with poor load transfer at joints and cracks.

Figure 1. Design Chart for Case 1 Condition of Existing Pavement



Case 2: Existing pavement exhibits a small amount of midslab and corner cracking. It exhibits reasonably good load transfer at the joints and cracks. Localized repairs were performed to correct distressed slabs.

Figure 2. Design Chart for Case 2 Condition of Existing Pavement



Case 3: Existing pavement exhibits a small amount of midslab cracking and good load transfer at the cracks and joints.

Figure 3. Design Chart for Case 3 Condition of Existing Pavement

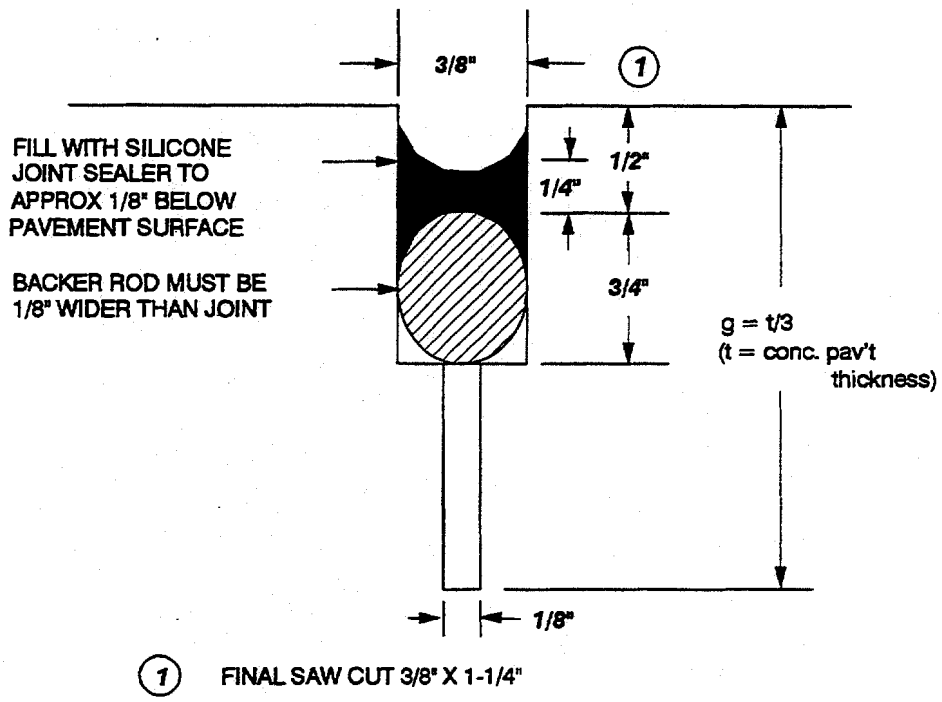
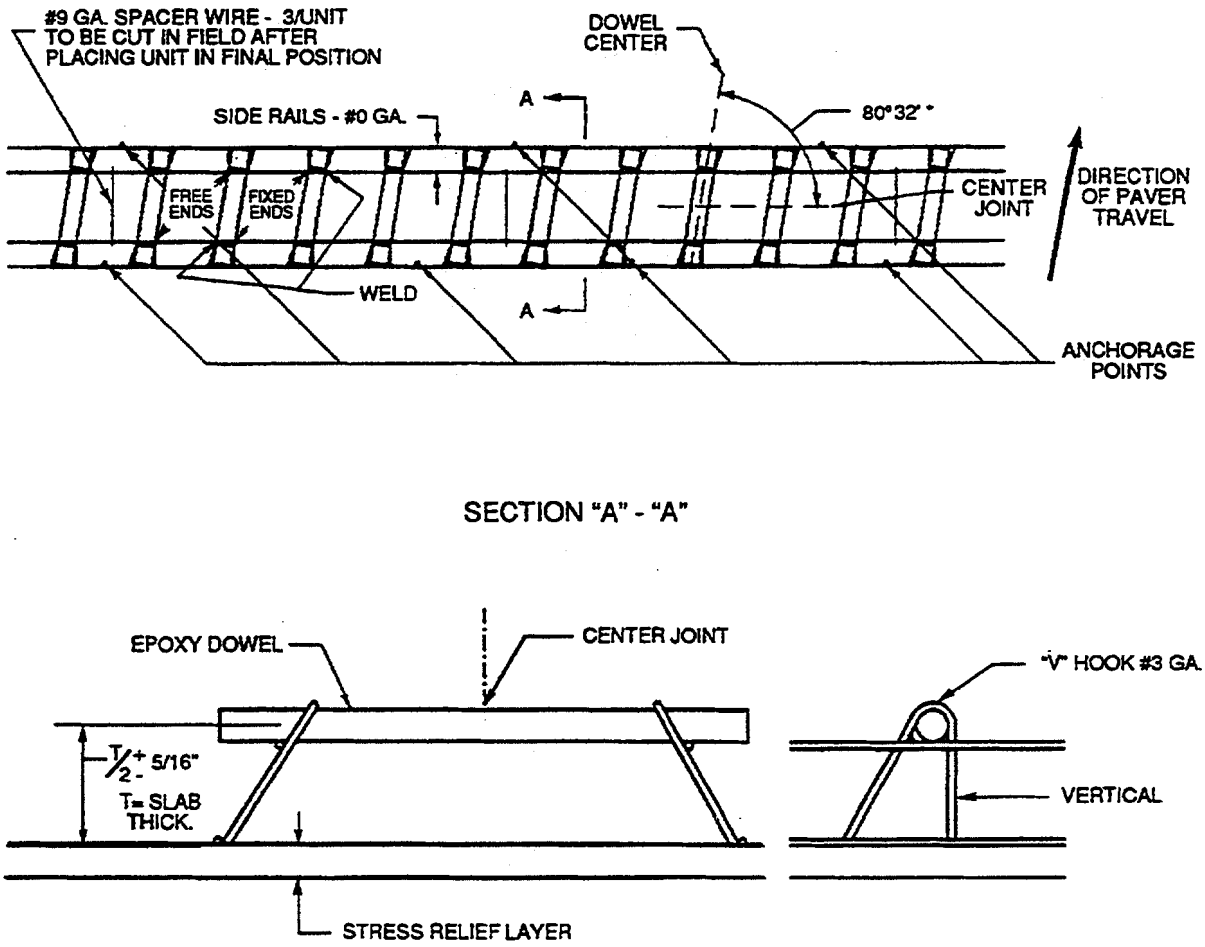


Figure 4. TRANSVERSE PAVEMENT JOINT C4E AND C4E-D MODIFIED



NOTES: Epoxy dowel shall be sheared at both ends.

Steel for dowel shall conform to AASHTO Spec. M31 for grade 40.

Epoxy for dowels shall conform to ASTM spec. M254, type B, (5 to 10 mil. after cure).

A minimum of seven (7) anchorage points are required, four on the side of the basket facing the front of the paver.

The dowel basket assemblies shall be anchored to the underlying concrete pavement to prevent assembly movement. The anchorage system shall be to use pins or fasteners of sufficient length and shank diameter (0.177 inch minimum) to penetrate into the inplace concrete a minimum of 1 inch. The contractor's chosen anchorage procedure must be demonstrated prior to the start of paving to the satisfaction of the engineer.

*If joints in the existing pavement are not skewed, then dowel assemblies for the unbonded overlay shall be aligned so that the dowel baskets are parallel with the existing transverse or contraction joints.

Figure 5. PLAN OF SKEWED* CONTRACTION JOINT DOWEL ASSEMBLY

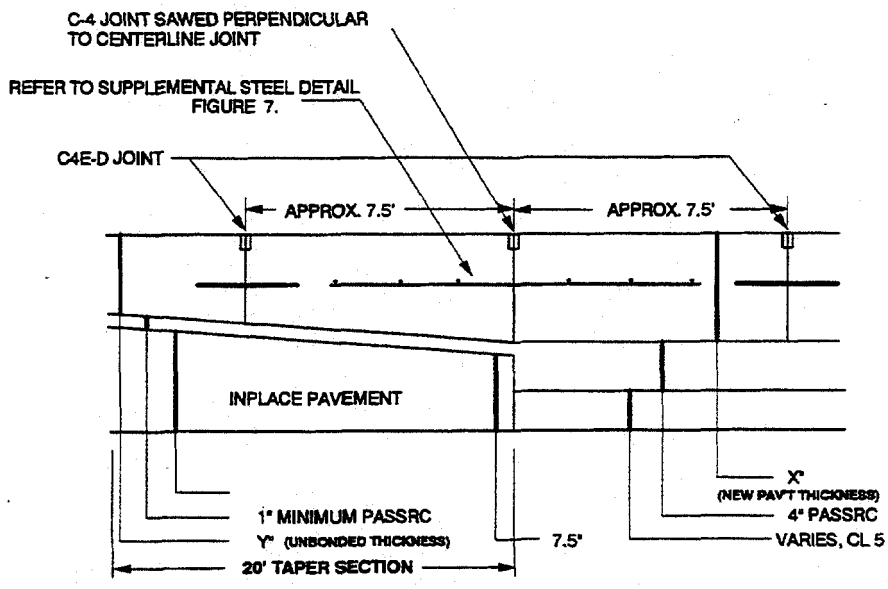


Figure 6. TRANSITION FROM UNBONDED OVERLAY TO PAVING ON GRADE(USE NEAR BRIDGES AND OTHER REMOVAL AREAS)

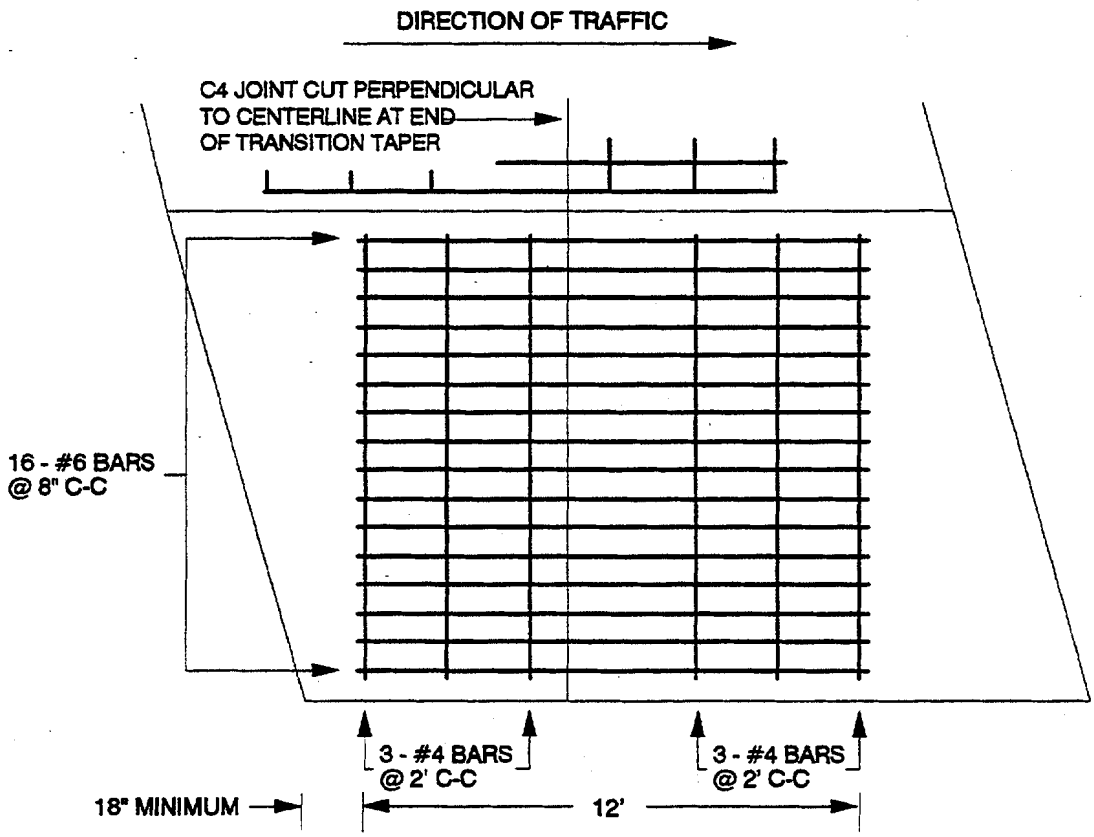
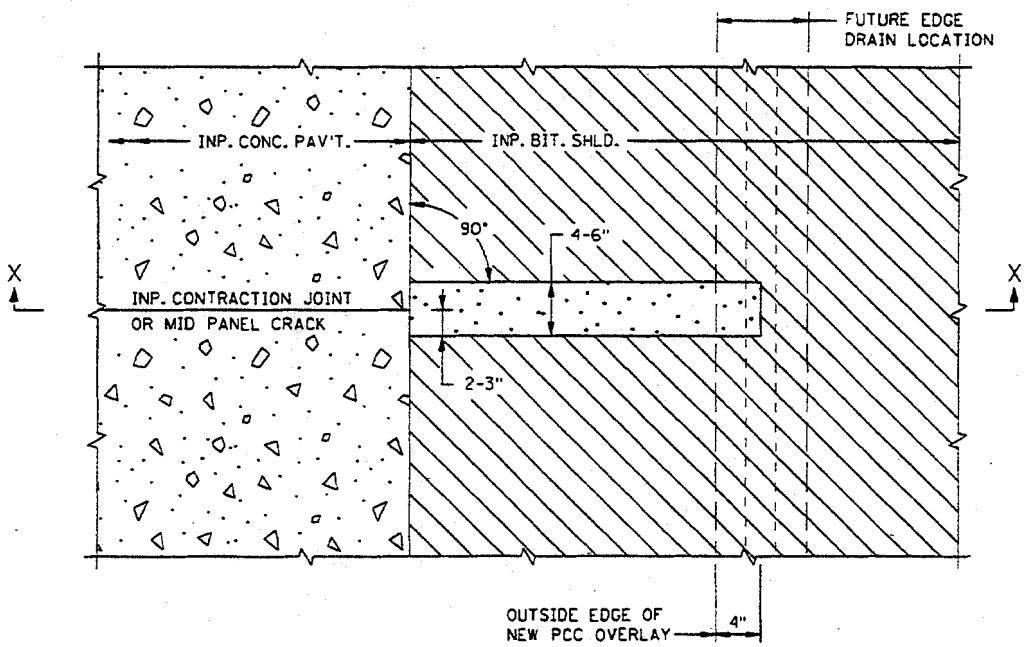
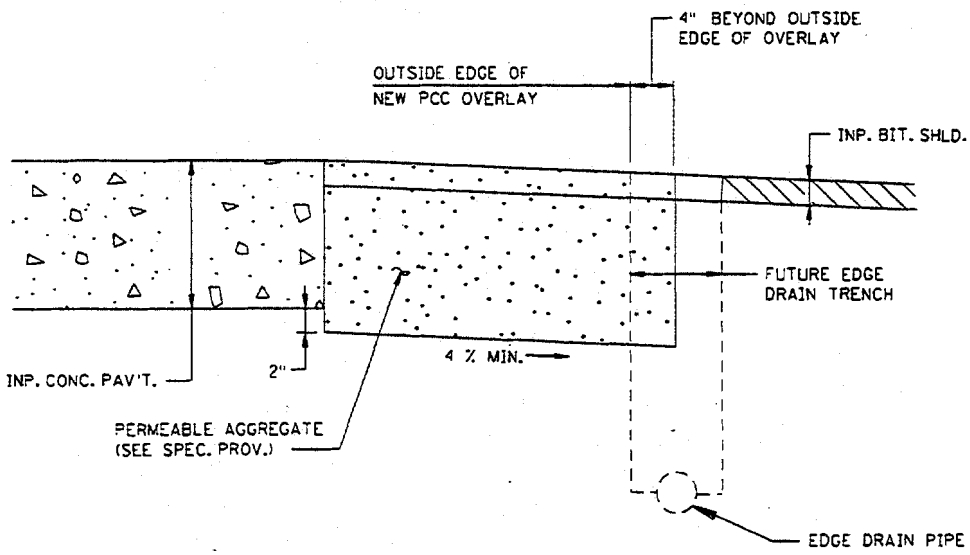


Figure 7. TRANSITION FROM UNBONDED OVERLAY TO PAVING ON GRADE (SUPPLEMENTAL STEEL DETAIL)

INTERCEPTOR DRAIN DETAIL ①



PLAN VIEW



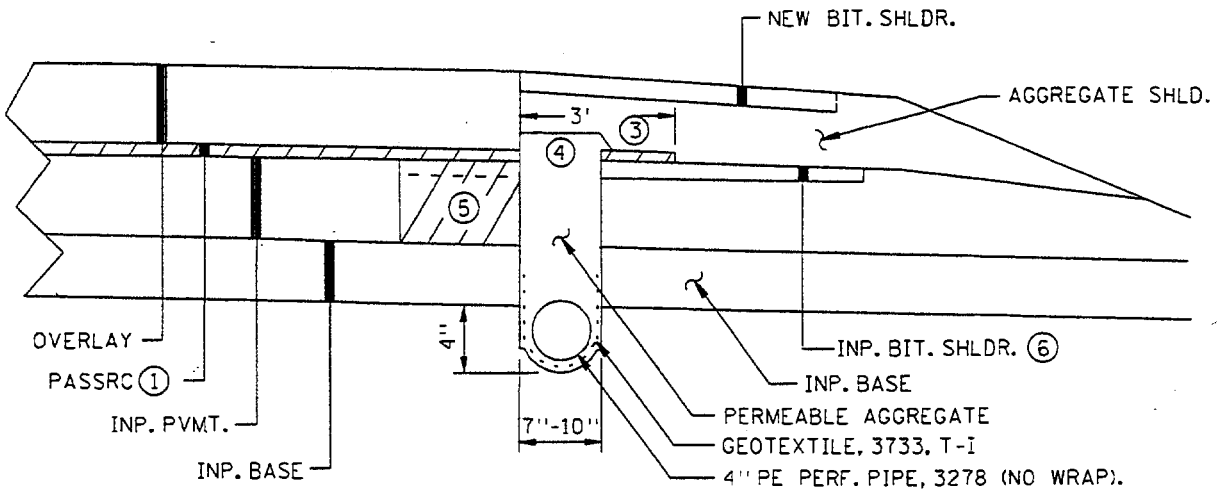
SECTION X-X

NOTE:

① SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS

Figure 8.

SUBSURFACE DRAIN, PERMEABLE BASE & DRAIN USED WITH PASSRC ① ②



NOTES:

① PASSRC - PERMEABLE ASPHALT STABILIZED STRESS RELIEF COURSE

② SEE SPECIAL PROVISIONS FOR MATERIAL AND CONSTRUCTION DETAILS

③ WIDTH AS NEEDED TO SUPPORT PAVER TRACK

④ PERMEABLE AGGREGATE TO BE HEAPED 2" ABOVE TOP OF PASSRC AFTER COMPACTION

⑤ INTERCEPTOR DRAINS TYPICALLY USED AT THIS LOCATION. SEE DETAIL & SPEC. PROVISIONS IF APPLICABLE

⑥ IF THE BIT. SHLDR. REMAINS INPLACE, THE PASSRC AND SHLDR. CAN BE REMOVED BY MILLING, TRENCHING, OR OTHER METHOD, PROVIDED THE REMAINING BIT. SHLDR. IS NOT DISTURBED/DISPLACED

Figure 9.

APPENDIX E

PAVEMENT REHABILITATION

SELECTION PROCESS

January 4, 1993

Mn/DOT
REHABILITATION SELECTION PROCESS

Mn/DOT personnel continually design rehabilitations for pavements that are in need of repair. The rehabilitation selection is based on any number of factors (traffic, past performance, cost, pavement quality, etc.). The designer certainly did some type of economic analysis, however informally.

Federal-Aid Policy Guide (FAPG) 23 CFR 626 mandates that each state have an acceptable Rehabilitation Selection Process by January, 1993.

A Mn/DOT task force was formed in 1992 to look at a procedure for evaluating the applicability of unbonded concrete overlays. A STATUS REPORT dated July 20, 1992 was prepared to document their recommendations. This Task Force then expanded their scope to include bituminous and bituminous over concrete rehabilitation alternatives. This paper documents the process that the Task Force recommends for rehabilitation selection.

It must be stressed that although this report focuses on the procedure to be used for economic analysis, that there are other factors which will influence the final selection.

The two principle components needed for an economic analysis of different rehabilitation strategies are the cost and the anticipated life of the rehabilitation. The cost is quite easy to estimate with a fairly high degree of accuracy. The anticipated life is another matter altogether.

This process also does not specify the method to be used in computing maintenance costs and user delay costs. If these costs are significantly different for various alternatives, they should be entered into the economic analysis.

PAST PERFORMANCE LIMITATIONS

There are many people who feel that a quick examination of the past projects will easily lead to the service lives of the various rehabilitations. Those who work with the data closely, however, realize that this is a very complicated question, for some of the following reasons:

1) Life. When is a pavement "dead?" When it has been reconstructed or overlayed? If so, one must look at WHY it was rehabilitated - was it even pavement related? Should it have been rehabilitated earlier, but no money was available? How do you average in a section that has still not been rehabilitated? Every segment must have a detailed forensic study in an attempt to quantify WHY rehabilitate.

2) Pavement Conditions. the Surface Rating (SR) is a subjective measure. The Present Serviceability Rating (PSR) is certainly less subjective, but does not always capture the true "ride," particularly for severe transverse cracks in bituminous and faulting in concrete. There are many examples of pavement that have had no known rehabilitations that exhibit improved ratings over time.

3) Variability. It is very difficult to classify pavements as `similar.' Two 3" overlays may exhibit entirely different performance because of different:

A. **Traffic.** Not only how many loads, but when they are applied.
B. **Environment.** Rainfall, temperature, freeze-thaw cycles.
C. **Subgrade.** Cut or fill, uniformity.
D. **Drainage.** Water table, permeability.
E. **Construction.** Compaction, weather during, quality.
F. **Materials.** Aggregate, bituminous, cement.
G. **Existing pavement.** Condition of the old pavement.

4) Implementation of findings. Even after a detailed forensic study is done, and classification done, a lot of variability is inevitable, and perhaps very limited sample sizes. Even more, many people will wonder how you can predict the performance of a 1993 overlay by looking at pavements built 10 or more years ago -- "We don't build them like that anymore."

With the above items in mind, the Task Force felt that it would be best, at least int he interim, to develop estimated lives through a survey of pavement experts. These estimates were to be based on their past experience, modified by their feelings with how recent changes (QM, spec. changes, etc.) would influence performance.

SERVICE LIFE TABLES

Tables A,B and C are the result of the survey of the District materials Engineers and Central Office pavement engineers. The results were tabulated, and averages and standard deviations were calculated. Outliers were identified as those that were more than 2 standard deviations away from the average. The outliers were eliminated, and averages and standard deviations recalculated. The tables basically consist of these averages and standard deviations, with very minor modifications for uniformity.

APPLICATION

All projects will be required to undergo a rehabilitation selection process. For new construction and reconstruction, this process will be as in the past - the formal Pavement Selection process. For resurfacing and reconditioning, the economic analysis should, as a minimum, be documented in the Design Recommendations Report. This analysis may actually be performed earlier, in the programming or scoping phase of project development. The economic analysis will be reviewed by the Pavement Design Engineer.

Table D illustrated two different cases:

Case 1 is a concrete pavement with high soil strength and low traffic. Five different rehabilitation scenarios are analyzed. All scenarios must of course have the same analysis period - in this case 30 years, which is the anticipated life of the unbonded overlay.

Case 2 is a concrete pavement with low soil strength and high traffic. Six scenarios are examined, again with a 30-year analysis period.

The anticipated lives came from Tables A, B and C with some very small adjustments to accommodate the 30 year analysis period. The LIFE should be used, unless there is reason to expect higher or lower lives. Only in very rare cases should a life be used that is outside of the range of the LIFE plus or minus the RANGE. In other words, if the LIFE is 9 and the RANGE is 3, the life of 9 should be used less there are reasons to expect a shorter or longer life. a shorter life than 6 years (9-3) or a longer life than 12 years (9+3) will only very rarely be allowed, with very good reason for exceeding the recommended range.

Note that all scenarios must have an equal analysis period, and that the real interest rate used should be 4.5%. Note also that a series of very short (less than 8 year life) fixes should not be used. In higher traffic areas, perhaps the minimum life should be extended, so as to minimize disruption to traffic. This minimal life requirement may be lifted when the last rehabilitation selected is simply to carry the scenario to the end of the analysis period.

FUTURE

Tables A, B and C must be monitored and updated as additional information becomes available. For example, there is a Task Force looking in great detail at full-depth bituminous pavements. Any findings relating to service lives should be integrated into the Rehabilitation Selection Process. In addition, an effort must be made to do the detailed `forensic' studies of past pavement performance to further refine the estimated service lives. The tables and updating of them will be the responsibility of the Pavement Management Engineer.

Mn/DOT must also continue working on developing accurate maintenance and user costs which can then be included in the economic analysis.

TABLE A

BITUMINOUS PAVEMENT REHABILITATION RECOMMENDED SERVICE LIVES

January, 1993

THIN (<=2") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	7	9	11
		RANGE	2	3	3
	FAIR	LIFE	6	8	9
		RANGE	2	2	3
	POOR	LIFE	4	6	8
		RANGE	2	2	2

MEDIUM (2-4") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	10	12	14
		RANGE	2	2	3
	FAIR	LIFE	8	10	12
		RANGE	2	2	2
	POOR	LIFE	7	8	10
		RANGE	2	2	2

THICK (>=4") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	13	15	17
		RANGE	2	2	3
	FAIR	LIFE	11	13	15
		RANGE	2	2	3
	POOR	LIFE	9	11	13
		RANGE	2	2	2

MILL & THIN OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	8	10	12
		RANGE	2	3	3
	FAIR	LIFE	7	9	10
		RANGE	2	3	3
	POOR	LIFE	5	7	9
		RANGE	2	2	

MILL & MEDIUM OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	11	13	15
		RANGE	3	3	3
	FAIR	LIFE	9	11	13
		RANGE	3	3	3
	POOR	LIFE	8	10	12
		RANGE	3	3	3

MILL & THICK OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	14	16	19
		RANGE	2	2	3
	FAIR	LIFE	12	14	17
		RANGE	2	2	3
	POOR	LIFE	11	13	16
		RANGE	2	2	3

CRITERIA FOR ABOVE TABLES:

SURF COND: CONDITION OF EXISTING PAVEMENT
<u>GOOD</u> - MINIMAL STRIPPING & NO RUTTING & 10 TON <u>FAIR</u> - SEVERE TRANSVERSE CRACKING OR MINIMAL RUTTING OR SOME STRIPPING OR 7-9 TON <u>POOR</u> - SEVERE RUTTING OR 5-7 TON SEVERE STRIPPING OR SEVERE MULTIPLE CRACKING

TRAFFIC	DESIGN ESALS
HIGH	>5M
MED	1-5M
LOW	<1M

TABLE B BITUMINOUS OVER CONCRETE PAVEMENT REHABILITATION RECOMMENDED SERVICE LINES

January, 1993

THIN (<=2") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	6	7	9
		RANGE	2	2	3
	FAIR	LIFE	4	6	7
		RANGE	2	2	2
	POOR	LIFE	3	4	6
		RANGE	2	2	2

MEDIUM (2-4") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	8	10	12
		RANGE	2	2	2
	FAIR	LIFE	6	8	10
		RANGE	2	2	2
	POOR	LIFE	5	6	8
		RANGE	2	2	2

THICK (>=4") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	11	13	15
		RANGE	2	2	2
	FAIR	LIFE	10	12	13
		RANGE	2	2	2
	POOR	LIFE	8	10	12
		RANGE	2	2	2

MILL & THIN OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	6	8	10
		RANGE	2	3	4
	FAIR	LIFE	5	7	8
		RANGE	2	2	3
	POOR	LIFE	3	5	7
		RANGE	2	2	2

MILL & MEDIUM OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	9	11	13
		RANGE	2	2	3
	FAIR	LIFE	7	9	11
		RANGE	2	2	3
	POOR	LIFE	6	8	9
		RANGE	2	2	3

MILL & THICK OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
SURF COND	GOOD	LIFE	12	14	17
		RANGE	2	3	4
	FAIR	LIFE	11	13	15
		RANGE	2	3	4
	POOR	LIFE	9	11	13
		RANGE	2	3	4

CRITERIA FOR ABOVE TABLES:

SURF COND: CONDITION OF EXISTING PAVEMENT

GOOD - MINIMAL STRIPPING & NO RUTTING
FAIR - SEVERE TRANSVERSE CRACKING OR MINIMAL RUTTING OR SOME STRIPPING
POOR - SEVERE RUTTING OR SEVERE STRIPPING OR SEVERE MULTIPLE CRACKING

TRAFFIC	DESIGN ESALS
HIGH	>5M
MED	1-5M
LOW	<1M

TABLE C CONCRETE PAVEMENT REHABILITATION RECOMMENDED SERVICE LINES

January, 1993

THIN (<=3") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
R-VALUE	HIGH	LIFE	9	10	13
		RANGE	3	3	3
	MED	LIFE	7	9	11
		RANGE	3	3	3
	LOW	LIFE	5	8	10
		RANGE	3	3	3

THICK (>3") BIT OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
R-VALUE	HIGH	LIFE	12	14	17
		RANGE	2	2	2
	MED	LIFE	10	13	15
		RANGE	2	2	2
	LOW	LIFE	9	11	13
		RANGE	2	2	2

CRACK & SEAT + THICK OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
R - VALUE	HIGH	LIFE	13	15	18
		RANGE	2	2	2
	MED	LIFE	11	13	15
		RANGE	2	2	2
	LOW	LIFE	9	11	14
		RANGE	2	2	2

SHATTER + THICK OVERLAY			DESIGN ESALS		
			HIGH	MED	LOW
R - VALUE	HIGH	LIFE	14	17	19
		RANGE	3	2	2
	MED	LIFE	13	15	17
		RANGE	3	2	2
	LOW	LIFE	11	13	15
		RANGE	3	3	2

CONC PAVEMENT RESTORATION (CPR)			DESIGN ESALS		
			HIGH	MED	LOW
R - VALUE	HIGH	LIFE	10	11	12
		RANGE	2	2	3
	MED	LIFE	9	10	11
		RANGE	2	2	3
	LOW	LIFE	7	9	10
		RANGE	2	2	3

PLANE & EDGE DRAINS			DESIGN ESALS		
			HIGH	MED	LOW
R - VALUE	HIGH	LIFE	9	11	13
		RANGE	1	2	2
	MED	LIFE	7	9	11
		RANGE	1	2	2
	LOW	LIFE	6	8	10
		RANGE	1	2	2

CRITERIA FOR ABOVE TABLES:

SOIL STRENGTH	R - VALUE
HIGH	>50
MED	12-50
LOW	<12

TRAFFIC	DESIGN ESALS
HIGH	>5M
MED	1-5m
LOW	<1M

RECONSTRUCTION AND JOINT SEAL LIVES

	LIFE	RANGE
NEW BITUMINOUS	20	3
NEW CONCRETE	35	3
UNBONDED OVERLAY	30	5
JOINT SEAL: NEW PAVEMENT CPR	 12 9	 2 2

TABLE D

REHAB SELECTION TASK FORCE
EXAMPLE ECONOMIC ANALYSIS

CASE 1 - CONCRETE ROAD, HIGH SOIL STRENGTH, LOW TRAFFIC					
	STRATEGY	YEAR	FUTURE COST	(P/F)	PRESENT COST
A)	UNBONDED OVERLAY	0	$325,000	1	$325,000
	CPR	18	$50,000	0.4528	$22,640
				TOTAL	$347,640
B)	THIN OVERLAY	0	$90,000	1	$90,000
	THICK OVERLAY	13	$160,000	0.5642	$90,283
				TOTAL	$180,283
C)	THICK OVERLAY	0	$160,000	1	$160,000
	THIN OVERLAY	17	$90,000	0.4731	$42,586
				TOTAL	$202,586
D)	CPR	0	$50,000	1	$50,000
	THICK OVERLAY	12	$160,000	0.5896	$94,346
				TOTAL	$144,346
E)	PLANE & EDGE DRAIN	0	$65,000	1	$65,000
	THICK OVERLAY	13	$160,000	0.5642	$90,000
				TOTAL	$155,283
CASE 2 - CONCRETE ROAD, LOW SOIL STRENGTH, HIGH TRAFFIC					
	STRATEGY	YEAR	FUTURE COST	(P/F)	PRESENT COST
A)	UNBONDED OVERLAY	0	$325,000	1	$325,000
	CPR	18	$50,000	0.4528	$22,640
				TOTAL	$347,640
B)	THIN OVERLAY	0	$90,000	1	$90,000
	MILL + OVERLAY	5	$90,000	0.8024	$72,221
	MILL + OVERLAY	10	$90,000	0.6439	$57,953
	MILL + OVERLAY	15	$90,000	0.5167	$46,505
	MILL + OVERLAY	20	$90,000	0.4146	$37,318
	MILL + OVERLAY	25	$90,000	0.3327	$29,946
				TOTAL	$333,943
C)	THICK OVERLAY	0	$160,000	1	$160,000
	MILL + OVERLAY	9	$160,000	0.6729	$107,665
	MILL + OVERLAY	18	$160,000	0.4528	$72,448
	THIN OVERLAY	27	$90,000	0.3046	$27,422
				TOTAL	$367,535
D)	SHATTER & OVERLAY	0	$185,000	1	$185,000
	MILL + OVERLAY	11	$160,000	0.6161	$98,592
	MILL + OVERLAY	20	$160,000	0.4146	$66,343
				TOTAL	$349,935
E)	CPR	0	$50,000	1	$50,000
	CPR	7	$80,000	0.7348	$58,786
	THICK OVERLAY	14	$160,000	0.5399	$86,396
	MILL & OVERLAY	23	$160,000	0.3633	$58,136
				TOTAL	$253,318
F)	PLANE & EDGE DRAIN	0	$65,000	1	$65,000
	PLANE + CPR	6	$95,000	0.7678	$72,950
	THICK OVERLAY	12	$160,000	0.5896	$94,346
	MILL + OVERLAY	21	$160,000	0.3967	$63,486
				TOTAL	$295,782
NOTE: COSTS SHOWN ARE FOR ILLUSTRATION ONLY.					

APPENDIX F

ENVIRONMENTAL BORE HOLES

MINNESOTA DEPARTMENT OF TRANSPORTATION
Engineering Services Division
Technical Memorandum No. 93-18-MRE-10
May 7, 1993

TO: 57, 612, 618, 650

FROM: J. E. Sandahl, Director/Assistant Chief Engineer
Engineering Services Division

SUBJECT: Soils Exploration Borings, New Regulations

This Technical Memorandum will continue in force until May 1, 1995 or until superseded or suspended.

The 1992 legislature imposed requirements for sealing certain soil borings to prevent potential contamination of aquifers and vertical connection of different aquifers. The Minnesota Department of Health has gone through the state rule making process and has issued Rules (Chapter 4725) effective May 10, 1993, which will affect some operations of district soils and Aggregate Unit drill crews.

Previous rules had already restricted construction of water wells, monitoring wells, piezometers, and dewatering systems. The new regulations now include borings for subsurface exploration. Borings to install footings, piles, posts etc. are not affected.

The new regulations require that any boring meeting the legal definition of an "Environmental Bore Hole" (EBH) be drilled only by licensed operators and equipment, be sealed in a prescribed manner, and be reported to the Department of Health within 30 days. Effective May 10, 1993 EBH's will be bored and sealed only by Mn/DOT personnel licensed and equipped to so do legally. Presently the Foundations Unit has the only crews licensed and equipped to drill and seal EBH's. Mn/DOT's solid-stem auger equipment does not presently have the capability to properly seal EBH's, and crews and equipment are not licensed to do so. With some additional advanced planning by District Soils Engineers (and others who may be affected) and some increased care, department field crews will be able to operate within the Department of Health regulations. Right now it appears that the majority of borings customarily made by solid stem auger crews can continue to be drilled. However each employee involved in subsurface exploration must take appropriate care to comply with these regulations.

Attached is a guide for soils field crew chiefs to enable them to respond positively to the new regulations. This guide was developed cooperatively by a task force, chaired by Tim Andersen, comprising members of the District Soils Engineer's group and members of the Geotechnical Engineering staff in the Office of Materials Research and Engineering. It has been reviewed by the Department of Health, which generally concurs with it's intent.

In brief, the guide provides crew chiefs the information they need to avoid drilling "Environmental Bore Hole's" as defined by the Department of Health regulations.

Mn/DOT policy to avoid drilling contaminated sites with conventional solid stem augers is also stated in these instructions.

If obtaining subsurface information at a site requires one or more EBH the Foundations Unit is prepared and equipped to properly acquire the information. If the Foundations Unit cannot do the work a licensed boring contractor will be needed. The attached Request for Drilling form should be used for all work requested of the Foundations Unit. District Soils Engineers should notify the Foundations Unit as early as possible so the work can be scheduled.

Chuck Howe, Hydrogeologist, of the Geology Unit of the Geotechnical Engineering Section, is Mn/DOT's official registered Monitoring Well Contractor under the regulations. Any questions regarding the regulations or application of this technical memorandum and guidelines should be directed to him. His phone number is (612) 779-5602. He can also help with abandonment procedures for EBH's drilled accidently.

The Geotechnical Section will assist the Districts in developing a contingency plan to deal with an accidental EBH. The Department of Health has encouraged Mn/DOT to procure simple grouting equipment that can be used to seal EBH drilled by solid stem auger. The Geotechnical Section will work with the Districts and develop a process and equipment to be used.

Violation of other similar environmental rules has been both costly and embarrassing to the department. Please use your best efforts to comply with the guidelines provided to prevent aquifer contamination.

SPECIAL NOTE: THESE GUIDELINES APPLY TO MN/DOT EMPLOYEES ONLY.

These guidelines represent an attempt by Mn/DOT to coordinate and implement a reasonable internal administrative response to the official Department of Health Rules. They should not be construed, in any way, to officially interpret those rules for any parties outside of the Department of Transportation or to apply to any other entities, public or private.

Attachment

Notice: TM-93-18-MRE-10 is hereby made Appendix F of the Geotechnical and Pavement Manual. Termination dates are no longer in force.

- END -

Attachment to Tech. Mem. 93-18-MRE-10 **4/23/93**

Mn/DOT SOLID-STEM AUGER BORINGS BACKFILL POLICY (GUIDELINES FOR DISTRICT SOILS AND AGGREGATE UNIT AUGER CREWS)

These guidelines are for use by Mn/DOT employees only should not be construed to interpret State Rules for any other entities, public or private.

The Minnesota Department of Health has new rules regulating wells and borings that will take effect May 10, 1993. It will be Mn/DOT policy that borings which meet the criteria for Environmental Bore Holes (EBH) will not be drilled by solid-stem auger rigs, since they do not have the capability of properly grouting bore holes.

A boring is considered an Environmental Bore Hole when all three of the following criteria are met:

1) The boring encounters a *water bearing layer.*
2) The boring is greater than 25 feet in depth or goes through a *confining layer.*
3) The boring is used for measuring or testing physical parameters (e.g., field identification of soils, checking water levels, rock depths, etc.).

A *water bearing layer* is defined as a saturated layer of unconsolidated material or bedrock with sufficient permeability, thickness and areal extent to produce enough water to supply a well. Unconsolidated granular materials such as sand, loamy sand, or sand and gravel, and bedrock units such as sandstone or carbonates will generally have sufficient permeability to produce the amounts of water required by the definition. Seams, lenses or strata of saturated granular soil, up to 5 feet thick, should not be considered as having sufficient thickness or areal extent to sustain a well. Nongranular seams, which are often found within a granular deposit, should not be considered when determining the thickness of a water bearing layer unless such seams are sufficiently thick and areally continuous. Organic deposits, such as peat or muck do not meet the definition of a water bearing layer. A *confining layer* is defined as a layer of clay (including sandy-clay and silty-clay) or shale at least ten feet thick. Soils described in these definitions are as defined by the Mn/DOT Soils Triangle.

Auger borings may be drilled to any depth provided a water bearing layer is not encountered. Solid-stem augers may be used to drill through any saturated mineral soil placed by prior construction as backfill of swamp or muck excavation without being considered an EBH.

Constructing a piezometer in a bore hole that does not meet the criteria for an EBH is permitted, since it does not change the status of the hole. However, auger borings or piezometers shall not be used to extract water samples, since such usage would place them under the category of monitoring wells.

Should the conditions which define an EBH be unexpectedly encountered during drilling, work should be stopped and the next level supervisor contacted immediately for further instructions.

The Foundations Unit is equipped and licensed to drill and seal EBH's and should be contacted when such borings are necessary or anticipated.

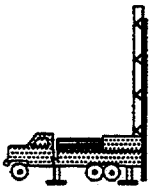
All auger borings shall be backfilled with the drill cuttings, on-site soils, or imported material, with a texture and permeability similar to materials encountered in the bore holes. Imported backfill materials should have a lower permeability than material encountered. The bore hole shall be completely filled from the bottom or cave-in depth to the original ground surface. Tamping or compacting the backfill material should be performed as necessary to minimize voids or backfill subsidence. Backfilling shall be performed in a timely manner after completion of the bore hole.

Auger borings shall not be permitted in known or suspected contaminated areas regardless of boring depth or groundwater elevation. If contamination of any type is noted while drilling, work should be stopped and the next level supervisor contacted immediately for further instructions.

Mn/DOT - GEOTECHNICAL SECTION - FOUNDATIONS UNIT

REQUEST FOR DRILLING

Note: This form shall be used for drilling needs not specifically identified in the Artemis Program.



TO: J.L. Dempsey, Field Operations Supervisor
Geotechnical Engineering Section
Materials Research & Engineering Lab
Phone: 779-5600 Fax: 779-5616

FROM:_____ District:_____
Phone:_____ Fax:_____

DATE:_____

DATE NEEDED BY:_____

TYPE OF BORING

- ☐ Solid Stem Auger Boring
- ☐ Foundation/Environmental Boring

USER NOTES

This form shall be used by District Personnel for requesting environmental and/or solid stem auger boring explorations by the Foundations Unit.

For solid stem auger borings, the District is responsible for clearing utilities, staking borings, providing access, obtaining access permission, supplying survey information and furnishing personnel for soil classification and record keeping.

For Foundation borings, the Foundations Unit will provide for soil classification, lab testing, finished boring logs and a report.

PROJECT INFORMATION

S.P._____ TH_____

Location_____

FROM STA._____ TO STA._____

CHARGE I.D._____

LAYOUT PLAN

- ☐ not available
- ☐ attached
- ☐ available on workstation
  - node name: _____
  - directory: _____
  - filename: _____

BOREHOLE INFORMATION

Desired start date for drilling_____

Est. No. of Borings_____ Est. Depth_____

Estimated drilling duration_____

STRUCTURE

- ☐ roadway
- ☐ culvert
- ☐ building
- ☐ ret. wall
- ☐ other_____

TERRAIN

- ☐ swamp
- ☐ steep hills
- ☐ heavily wooded
- ☐ roadway (lane closure)
- ☐ shoulder (no lane closure)
- ☐ surface water/ice
- ☐ other_____

SITE ACCESSIBILITY

- ☐ no anticipated access problems (rubber tire rig)
- ☐ difficult access anticipated (track rig required)

Date in:_____ Date out:_____ form tdn 498

TRANSMITTAL RECORD SHEET
FOR
ENGINEERING MANUALS

Mn/DOT 1962 (5-96)

TRANSMITTAL		INDEX NO.	SUBJECT
NO.	DATE		

TRANSMITTAL RECORD SHEET
FOR
ENGINEERING MANUALS

Mn/DOT 1962 (5-96)

TRANSMITTAL		INDEX NO.	SUBJECT
NO.	DATE		

UPDATED TO 97-1

Please record the date you got the book on the transmittal record sheet.

Record the updated to transmittal letter on the transmittal record sheet.