Granular Base Failures in Low-Volume Roads in Ontario, Canada

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In 1990, 74 percent of the 196 773 km of Ontario roads was low-volume roads, and 68.5 percent of all roads was unpaved. For the same year in Canada, there was 825 743 km of roads, and 65 percent was unpaved and generally classed as low volume. This large percentage of low-volume roads is a reflection of the size of Canada and its sparse population. Gravel roads may fail due to a number of mechanisms. In this paper, only those cases are considered in which the road was constructed with aggregates that met or were predicted to meet the specifications but failed either because of unforeseen changes in the materials or because of unrecognized contaminants in the materials, which still met the specification. These cases can be divided into failures because of low permeability caused by the presence of plastic or micaceous fines, failures because of low permeability caused by the presence of bacteria and algae, and failures because of breakdown of the coarse aggregate to sand sizes. The Ministry of Transportation has recently changed the specifications for granular base course aggregate to introduce a Micro-Deval test in place of petrographic examination requirements and the Los Angeles abrasion and impact test. A maximum of 25 percent loss is permitted for granular base and 30 percent for granular subbase. In addition, there are strict grading requirements and a requirement that the materials be nonplastic and that there be less than 10 percent mica on the 75-μm sieve.

Road construction started in Ontario over 200 years ago in 1783 when the first United Empire Loyalists arrived to take up their land grants. The forerunner of the early road was the portage or "carrying place," which was a trail around rapids or between two waterways. These portage routes, developed by the aboriginal peoples of Canada, became the early road corridors. With the settlers came the wagon, creating an immediate need for passable roads. In 1783 the first 25 km of road, running west from Kingston to Bath in eastern Ontario, was built. This early road was a blazed trail through the dense forest. Because of the difficult terrain, the early roads were only usable during the dry midsummer months and during the winter when hard-packed snow provided a smooth traveling surface. By 1874 the length of major colonization roads was 1534 km. During the mid-1800s, a Canadian invention, the plank road, became popular. Plank roads were constructed by securing planks of wood with spikes to longitudinal timbers in the road bed and covering the planks with soil or sand and hot pitch to reduce friction and prolong the life of the road (*1*). The availability of good lumber meant that one kilometer of plank road could be constructed at one-fourth the cost of a kilometer of macadamized road. At the same time, the government focused on establishing the railroad system. This focus left Ontario's roads to deteriorate at an alarming rate. The Ontario Good Roads Association

TABLE 1 Ontario's Road System Length by Surface Type and Length in Kilometers and Low-Volume Road (AADT <2,000) Percentage, 1990

Road Type	Provincial Highways[a] (0 <AADT <15,000+)	Municipal Roads[a] (0 <AADT <4,000)	Access Roads[b] (0 <AADT <400)	System Total
Paved	17,379	44,610	0	61,989
Surface Treated	3,289	24,278	0	27,567
Gravel	998	62,797	39,646	103,441
Earth	0	3,752	0	3,752
Other	0	24	0	24
Length	21,666	135,461	39,646	196,773
AADT <2000	11,047 (51%)	94,823 (70%)	39,646 (100%)	145,516 (74%)

Source: [a] Ministry of Transportation, 1993
[b] Ministry of Natural Resources, 1993

was formed in 1894 by municipal officials and citizens in a concerted effort to protect their interests and make others aware of the benefits of well-made roads. Organized road building in Ontario began in 1896 when an engineer, A. W. Campbell, was appointed Provincial Instructor in Road Making.

In 1916 the Provincial Department of Public Highways was created under a Minister of Public Works and Highways. In 1916 there was approximately 88 000 km of country roads, mostly in southern Ontario. In the early 1920s, settlements in the northern regions of Ontario were still almost isolated, linked only by a few crude roads, most transport being by water or rail. Road construction in northern Ontario was instrumental in the development of the mining and lumbering potential of these vast areas. Strong highway links to the southern markets for northern resources and products ensured the prosperity of northern communities; many were still dependent on resource-based economies. After the Second World War, road construction and infrastructure development accelerated in both southern and northern Ontario. Ontario became Canada's most populated and affluent province with a population reaching 10 million in 1990 and enjoying one of the best road systems in the world. This road system includes 21 666 km of provincial highways, 135 461 km of municipal roads, and 39 646 km of access roads (Tables 1 and 2). There are about fifty people for each km of road in Ontario. Municipal roads and access roads provide the network among smaller communities and serve as connecting links for industry and commerce to main highways and export-import opportunities to Canada and the United States. The Ministry of Transportation (MTO) and the Ministries of Northern Development and Mines and Natural Resources are the provincial agencies responsible for new construction and maintenance of Ontario's road system. Approximate annual provincial government expenditures are $1 billion for provincial highways, $750 million for municipal roads, and $10 million for access roads.

Ontario provincial road design guidelines indicate that secondary highways with an annual average daily traffic (AADT) of less than 2,000 fall within the low-volume road design criteria. Secondary highways are further subdivided into three categories. Major secondary highways, with AADT greater than 1,000, serve

TABLE 2 Ontario's Road System Length by Program Type and Length in Kilometers, 1990

Program Type	Length (km)
Provincial Highways Program[a]	
King's	15,797
Secondary	5,663
Tertiary	206
Subtotal	21,666
Municipal Roads Program[a]	
Upper Tier (counties and regions)	20,917
Large Lower Tier (cities, towns, townships)	41,951
Small Lower Tier (towns, villages, townships, Indian reserves)	63,065
Unincorporated Areas	9,528
Subtotal	135,461
Access Road Program[b]	
Forest Access	28,135
Agreement Forest	2,763
Private Forest	1,196
Fuelwood	31
Ministry Service	1,169
Recreation Access	3,750
Cottage Access	549
Residential Subdivision	34
Public Transport	333
Winter	433
Mining Access	418
Other	835
Subtotal	39,646
Total	196,773

Source: [a] Ministry of Transportation, 1993
[b] Ministry of Natural Resources, 1993

major regional centers, communities, and resource areas. Intermediate secondary highways, with AADT of 400 to 1,000, link villages, recreational areas, and resource areas. Minor secondary highways, with AADT of less than 400, provide access to recreational and resource areas or parallel a higher-level provincial road. In 1990, 74 percent of the 196 773 km of Ontario roads was low-volume roads, 68.5 percent was unpaved. For the same year in Canada, there was 825 743 km of roads, and 65 percent was unpaved (2). This large proportion of low-volume roads is a reflection of the size of Canada and its sparse population.

A persistent challenge in any type of road construction in Ontario has been the need to find suitable construction aggregates that will consistently meet stringent performance requirements in a geographic area where severe climatic changes prevail, geological history is complex, and the age of soils and rocks ranges from less than 10,000 years to over 3.5 billion years. Aggregates for road base can be obtained from glacially derived sand and gravel deposits or from a wide variety of bedrock formations.

Case Histories of Granular Base Failures

Performance of Granular Base

The function of granular base in road construction is to carry the load of vehicles and to drain water from the pavement structure (Figure 1). In cold climates it also acts as an insulating layer, preventing or reducing the penetration of frost into the underlying, sometimes frost-susceptible, finer-grained soils. To accomplish this, suitable granular bases must be strong, free draining, and durable.

Some of these properties can be measured by performance tests. Strength of compacted granular base is measured by the California bearing ratio (CBR). Typical values for granular base are over 100 percent. Typical values for loose, sandy granular base are between 20 and 60 percent. A very loose sandy granular, such as dune sand, in which a wheeled vehicle might get stuck, would have values less than 15 percent. Strength of the particles making up the coarse portion can also be measured in the Los Angeles abrasion and impact test. Stability of the base, measured by CBR, is related to the grading of the material and to the degree of interparticle friction. For practical purposes, interparticle friction is related to the amount of freshly fractured faces on the coarse aggregate particles.

The drainage capability can be measured by permeability tests (ASTM D 2434). Satisfactory granular bases will typically have permeabilities between 10^{-4} and 10^{-3}cm/sec. If permeability of a granular base is less than 10^{-4}cm/sec (typically in the range of 10^{-5} to 10^{-7} cm/sec or less), failure will probably occur. Permeability is largely determined by the amount of material finer than 75 μm in granular base. As fines increase above about 8 percent by mass, permeability is rapidly reduced. In Ontario a maximum amount of 8 percent passing the 75 μm sieve is permitted in base crushed from gravel. In base crushed from bedrock, up to 10 percent fines is permitted because the fines from bedrock are usually not as claylike as those from gravel. Occasionally, base materials that meet and exceed the grading requirement will fail in service. In wet weather, failure may occur during placement of the pavement or shortly after having placed the overlying asphalt pavement. Failure is usually seen as severe rutting of the granular base or alligator cracking of the overlying asphalt within a few hours or days of placement. The reason for the rapid failure of the asphalt is the very poor compaction achieved against the underlying saturated and spongy base. If the base is placed under relatively dry conditions, failure may not show up for some months or years. Paved roads will show failure by severe to moderate depressions in the wheel paths and alligator cracking of the overlying asphalt pavement.

Gravel roads may fail because of a number of mechanisms. In this paper (Figure 2), only those cases are considered in which the road was constructed with aggregates that met or were predicted to meet the specifications but failed either because of unforeseen changes in the materials or because of unrecognized contaminants in the materials, which still met the specification (Table 3). These cases can be divided into failures because of low permeability caused by the presence of plastic or micaceous fines, failures because of low permeability caused by the presence of bacteria and algae, and failures because of breakdown of the coarse aggregate to sand sizes.

Failures Due to Low Permeability Caused by Plastic Fines

Moosonee Area

Moosonee, located in the Hudson Bay Lowlands of northeastern Ontario, is a flat-lying, featureless, swampy plain that slopes gradually toward James Bay. Access is by air, rail, or boat. This community serves as a commercial and institutional center. Gravel-surface, low-volume roads provide local access.

Aggregate materials used in the building of these gravel roads originate from two main sources. First, some of the river bars in the Moose river have been traditional sources of fine to coarse sand and fine gravel.

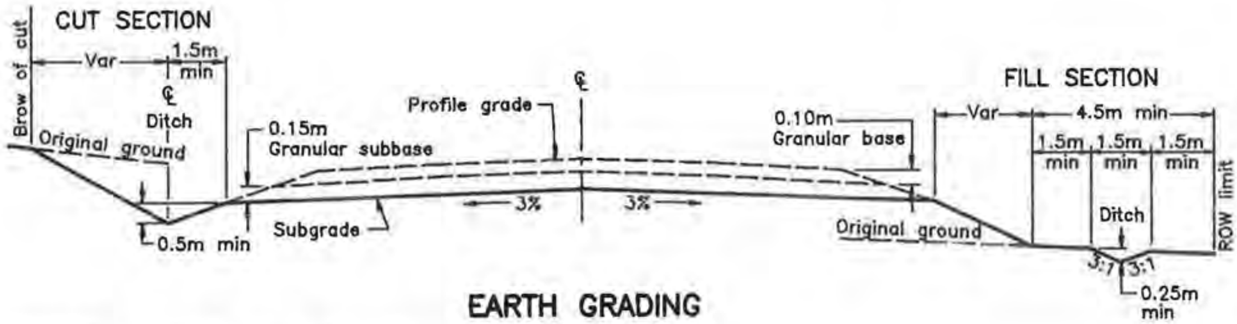

FIGURE 1 Typical Ontario low-volume road.

Second, flat-lying, medium-bedded Middle Devonian limestones are quarried for road gravel. Because of the high water table, all overburden stripping, drilling, blasting, and crushing operations are performed on a demand basis during the winter months with equipment brought in by rail. The quarried rock is an excellent aggregate exceeding all the requirements of granular base and being suitable for use in portland cement concrete (Table 3). The climate in the summer is frequently wet, and the poor condition of the gravel roads has been a constant source of distress to the people. The roads, when wet, become soupy and have ruts up to 150 mm deep. Walking along the road quickly covers the walker's boots with loose gravel of a muddy consistency, not unlike high-slump concrete. The granular base made from the quarried rock exhibits all the characteristics of failure because of the presence of plastic fines. Testing of the quarried rock showed that plastic fines in the pavements were not present. The source of the plastic fines was an unresolved problem.

FIGURE 2 Location map.

TABLE 3 Granular Base Test Results (Ranges Provided Where Applicable)

Case Study Location	Petrographic Number: Granular	Petrographic Number: Concrete	Los Angeles Abrasion, %	MgSO4: C.A., %	MgSO4: F.A., %	Pass 75 µm, %	Plasticity of Failed Material	Absorption, %
Moosonee Area, River Bar	134-208	196-257	-	-	-	5.0	plastic	1.0-1.7
Moosonee Area, Limestone	100-118	112-147	29	2.9	-	-	plastic	2.1-2.8
Windover Quarry	102	106	24	4	15	-	plastic	-
Moore Quarry	107-133	148-234	34-42	11	22	7.8-8.0	plastic	1.4-1.5
Thunder Bay Area[a]	120-140	160-180	17	-	-	5.0-7.0	plastic	-
Jackson Pit	101-103	118	19	2	-	4.8-6.2	non-plastic	0.57
Stormy Lake Pit	102-140	124-190	23-26	2-5	5-20	2.9-15.9	non-plastic	-
Spurline Pit	100-103	100-112	14-18	0.3-1.3	1.2-5.1	0.5-3.4	plastic	0.40-0.50
Oliver Quarry[b]	202	415	89	17	-	-	non-plastic	1.64
Parry Sound Quarry[c]	121	212	54 (39-60)	5	-	7.6	non-plastic	0.44
Granular A Specification Limits	200	-	60	-	-	2.0-8.0 pit 2.0-10.0 quarry	non-plastic	-

[a]Thunder Bay area coarse and fine aggregate Micro-Deval Abrasion Test results are 15 and 26%, respectively.
[b]Oliver Quarry Coarse Aggregate Micro-Deval Abrasion and Freeze-Thaw Test results are 13 and 3.2%, respectively.
[c]Parry Sound Quarry Coarse and Fine Aggregate Micro-Deval Abrasion Test results are 33 and 11%, respectively.

After an exhaustive search, it was found that the source of the plastic fines was the sands dredged from the Moose River. In order to meet the gradation specified for gravel roads more easily and to reduce cost, sands dredged from the river were blended in small quantities into the crushed quarried rock. These sands were of low fines content and apparently nonplastic. Inspection of stockpiles of the sand showed the presence of armored clay balls up to 100 mm in diameter. The clay was of a highly plastic nature and from the marine clays found over much of the region. Their presence in the sand had not been noticed because the balls were completely enveloped with coarse sand. When the balls were broken open, the highly plastic clay could be seen enclosed by a thin rind of sand. The obvious solution to the problem of the poor roads in Moosonee was to stop blending river sand into the quarried stone.

Windover Quarry

The Windover Quarry is located 15 km north of Buckhorn in Petersborough County. The site is a butte about 30 m high above the surrounding country. The geology of the site consists of about 19 m of horizontally bedded Middle Ordovician limestone interbedded with small amounts of slightly sandy and clayey dolomitic limestone. The limestone butte is an outlier unconformably lying on a peneplane of high-grade metamorphic gneisses of Late Precambrian age. Before the quarry was developed, the butte was drilled with a diamond drill in the center and the core was tested. The results showed that the rock met all the physical requirements for concrete and asphalt use and would certainly be an excellent source of granular base for an adjacent low-volume road (Table 3). There was no evidence of weathering of the rock in the drill core.

During reconstruction of the adjacent road, a quarry face about 18 m high was opened by drilling and blasting at the side of the butte. The contractor crushed the rock with a portable crushing plant. Extensive testing was done for gradation, and the crushed materials met the specification. No testing was done for other properties since previous testing showed the rock to be of excellent quality. During wet weather and before the road was paved with asphalt, some sections of the gravel surface failed and had to be replaced. The failure was similar to that experienced in Moosonee. The roads, when wet, become soupy and have ruts up to 150 mm deep. Walking along the road quickly covers the boots with mud. The granular base had become contaminated by plastic fines.

The source of the plastic fines was found to be two weathered beds, 1 and 2 m thick, of clayey dolomitic limestone in the quarry face. At the site of the drill core, these beds were unweathered, but at the side of the butte, these beds were deeply weathered for a distance of about 5 to 10 m into the face. These weathered beds had altered to a plastic, low-strength rock. The inspection staff and the contractor had ignored, or not considered significant, the deeply weathered nature of the rock since it only made up a small proportion of the face, which was composed of unweathered limestones. These beds of easily weathered dolomitic limestones are well known in this area of Ontario and are called "green markers" by local quarry operators. The rocks get their name from their greenish-grey color caused by the presence of reduced iron associated with the mineral dolomite. After slight weathering, these beds assume a characteristic light-brown color. When encountered in a quarry, they are normally unweathered but are susceptible to frost action, which causes popouts in concrete and asphalt, so they are not used for road bases (*3*). Crushed rock containing these unweathered green markers is usually satisfactory for use as granular base. The lessons learned from this experience were the necessity always to test the properties of material during production, whatever the previous exploratory testing has indicated, and also to remember that preliminary samples taken with a diamond drill may not reveal potential problems elsewhere on the site.

Moore Quarry

The Moore Quarry is located on the south shore of Lake Nipissing. The quarry was tested and shown in contract documents as meeting the specifications for use as a source of granular base aggregates for the reconstruction of a local low-volume road (Table 3).

The geology of the quarry is complex. The quarry face is in the side of an upthrust block of a vertical fault. The rock is highly fractured and consists of metamorphosed migmatite and gneissic biotite granite of Late Precambrian age. On the downthrust block of the fault a veneer of thin-bedded Middle Ordovician shaley dolostone overlies the gneiss.

The quarry was highly unusual because the operator did not have to drill and blast the rock before crushing. The contractor was able to extract the rock from the fault zone by digging it out with a large mechanical shovel. This was no doubt possible because of the sheared and broken nature of the rocks adjacent to the fault. The rock was crushed in a portable crushing plant. Extensive gradation testing was done, and the crushed materials met the specification. Before the road was paved with asphalt, some sections of the surface failed and had to be replaced. The failure was due to the presence of plastic fines, which made the granular material impermeable to water. The source of the plastic fines was highly plastic red clay found in the fault zone. In addition, several highly weathered, very soft biotite mica seams are associated with the fault. The red clay and mica were responsible for reducing the permeability of the compacted crushed rock, leading to rutting and soft spots on the grade. The contractor had found that as his operation moved away from the fault, it became more difficult to pull down rock with his excavator. As a result, he moved his operation along the strike of the fault in a search for more easily extracted rock. It was here that he encountered the plastic clay and mica seams. An interesting observation was the formation of miniature slope failures on the sides of the stockpile of contaminated granular base. These miniature debris slides could be initiated by pouring small amounts of water on the material. The lessons to be learned from this experience were to distrust any quarry operation that does not require drilling and blasting and to conduct an extensive investigation before approval of new sources.

Thunder Bay Area

Thunder Bay is on the northwest shore of Lake Superior near Jackson Pit (Figure 2). In this area, there has been extensive deposition of gravels by glacio-lacustrine action in the past 10,000 years. The area is noted for extensive raised beach deposits about 30 m above the present lake level. These beaches were formed when the level of Lake Superior was raised by glacial ice blocking the present outlet. The beach deposits are noted for their coarse gravels and cobbles with the relative absence of finer gravels and sand.

Following beach formation, the level of the glacial lake rose above the beach level. There was a period of deposition of highly plastic clay (Plasticity Index, 16 to 20) on the upper surface of the beaches. Following subsequent lowering of lake levels, the plastic clays were washed by precipitation into the openwork beach gravels so that today the upper layers of the raised beaches are clean and apparently suitable sources of aggregate. At depth (>1 m), however, the upper surfaces of cobbles and gravels are covered with a coating of the highly plastic clay. During exploration, testing of the gravels did not reveal the presence of significant amounts of the clay.

The current test for plasticity of gravels calls for testing the fraction finer than 425 μm. The fraction between 425 and 75 μm is made up of good-quality sand. The gritty nature of the sand can mask the plastic nature of the fines. Consequently, gravels from two sources were produced that met the grading (<8 percent passing the 75-μm sieve) and plasticity requirements. When used in highway construction, the overlying asphalt pavement failed within a few hours of placement. The failures appeared as severe alligator cracking and distortion in the wheel paths. The underlying granular base was found to be saturated and impermeable ($<10^{-6}$ cm/sec). The cause of the impermeability was clay derived from the clay-coated gravel particles.

These failures were extremely costly, in one case costing more than $1.5 million. As a result, MTO is considering changing the test method for plasticity of granular base. The testing should be done on the fraction passing 75 μm, rather than the fraction passing 425 μm.

Failures Due to Low Permeability Caused by Mica

Jackson Pit

Jackson Pit is located at the southeast corner of the Kaministikwia River Bridge on Highway 102 west of Thunder Bay. The crushed gravel and sand from this source was tested and shown in contract documents as meeting the specification requirements for granular base and for asphalt paving. The source was opened and used for road construction.

Bedrock underlying the Jackson Pit and its vicinity is over 2.5 billion years old. The bedrock and gravels of this area are composed of extensively altered and metamorphosed mafic metavolcanic metasedimentary and felsic intrusive rocks. Glaciofluvial outwash gravel is found in Jackson Pit. This material was deposited along the course of the ancient Kaministikwia River meltwater channel that emptied into the Lake Superior basin. The outwash had its origins at the Dog Lake Moraine 17 km north of the pit. Outwash gravel deposits near Jackson Pit are veneered by glacial lake red clay. In places the clay exceeds depths of 10 m and may also sometimes be found as boulders associated with the outwash gravels.

It was found during asphalt paving that the granular base was unstable; the asphalt pavement moved under a person's weight and was judged to have a deflection of about 5 mm. After the first day of paving, the westbound lane showed alligator cracking and broke up for a length of 260 m. The base was found to be saturated and to have low stability. The asphalt pavement was removed from this area and the grade allowed to dry before repaving. Several other shorter sections broke up in the same manner. A longer section on the eastbound lane also broke up and dished very badly. The asphalt pavement was removed, the base excavated to a depth of 150 mm, and a 16-mm crushed stone was used to replace the base before repaving. After the first winter, two further areas of pavement break-up developed, each about 5 to 7 m long. There were also several random longitudinal cracks and general distortion throughout the job, probably because of frost heaving.

The failed material had a fines content of 5.4 to 7.0 percent, which is well within the maximum allowed -75 μm content of 8 percent for crushed gravel. The stockpiled material when tested had 4.0 percent -75-μm content, with a standard deviation of 0.31 percent ($n = 31$). It was initially thought that clay boulders associated with the outwash deposit were responsible for the granular base failure. The granular base fines, however, proved to be nonplastic. The aggregate met and exceeded all other physical requirements for granular base. Petrographic examination and X-ray diffraction showed the 150-μm to -75-μm sieve fractions to consist of about 27 percent chlorite and weathered chlorite schist. This mica was thought to be responsible for the failure. The CBR was 122 percent, but the permeability was so low that it was judged "relatively impermeable" (probably less than 10^{-5} cm/sec). The major source of the chlorite schist was found to be bedrock outcropping just to the north of the source. The schist was found abundantly as weathered cobbles and boulders in the gravel source. Crushing of the gravel concentrated the weak and friable chlorite schist in the fine aggregate.

The effect of mica on permeability results from its large surface area per unit mass, a result of its flat flakey particle shape. For instance, a flake of mica with a ratio of thickness to size of 1:10 has four times the surface area of a spherical quartz particle of the same mass. In addition to the effect of large surface area, mica is extremely efficient at blocking the flow of water through porous media because the flakes orient themselves perpendicular to the direction of flow. This ability is used in drill mud technology to stop or reduce "lost circulation" into permeable bedrock formations below the surface.

Mica in fine aggregate can reduce the permeability of granular base aggregates to the point where failure can occur. As a result of this experience, examination of the 75-μm sieve fraction of all crushed granular base aggregates was adopted in this part of Ontario. If the mica content is greater than about 10 percent, further testing and study are conducted before approval of the material.

Stormy Lake Pit

Stormy Lake Pit No. 3 is located in an unsurveyed area in Kenora District, about 30 km south of Highway 17 on the west side of Highway 622. It is in a small glaciofluvial outwash gravel deposit located between glacial moraines. This source was tested and shown in contract documents as being suitable for use as granular base and asphalt paving.

Bedrock in this area is part of the Wabigoon Subprovince of the Superior Structural Province of the Canadian Shield. The Wabigoon Subprovince is an east-trending granite-greenstone belt containing metavolcanic and subordinate metasedimentary rocks. These Early Precambrian rocks are surrounded and cut by granitoid batholiths.

Granular base produced from the pit met and exceeded all specification requirements and was judged to be well graded, strong, and sound. This aggregate would normally be considered suitable and acceptable for use as granular base. The one qualification of this observation was the presence of 23 percent mica in the 75-μm fraction. Further testing showed that the granular base had a low permeability of 3.7×10^{-5} cm/sec, which revealed lack of a potential permeability problem.

During the Proctor compaction test, there was very substantial breakdown of both stone- and sand-sized material. Stone contents were reduced by some 1.6 times, and the fines increased approximately threefold. Following the Proctor test, permeabilities were on the order of 3×10^{-7}. There was, at the same time, a marked reduction in the mica present in the sand portion, especially a threefold reduction in that retained on the 75-μm sieve. It is likely that the mica had broken down and that most of it was finer than 75 μm. X-ray diffraction carried out on the passing 75-μm material showed that these micas consisted of chlorite and biotite. The X-ray diffraction signature of the Stormy Lake fines was identical to that of the fines found in Jackson Pit. At this point, while the contractor was still crushing in the pit, it was decided to stop production of granular base; 220,000 tons of subbase had already been placed on this 43-km contract. This subbase was unstable, and it was decided to spread a thin layer of the granular base from the Stormy Lake Pit on the subbase to improve stability. At considerable extra cost, Granular A was imported from a pit some 30 km further from the job.

From this study, it was concluded that the total amount of mica is not as important as the amount retained on or passing the 75-μm sieve.

Failures Due to Low Permeability Caused by Algae and Bacteria

MTO encountered a very unusual failure with the granular base used on a contract on Highway 17. During this failure, the granular material became "liquiplastic," and transport trucks actually bogged down. The source of the granular material was a pit located near Algoma Mills on the north shore of Lake Huron. Crushed gravel from this source exceeded all physical requirements for use as granular base, and the material was also used successfully for asphalt paving (Table 3).

The material forming the gravel in the Spurline Pit and vicinity is made up of a complex array of rocks of Middle Precambrian age. Proterozoic rocks of the Huronian Supergroup and post-Huronian diabase intrusions form the bedrock underlying the immediate area. Overlying the Precambrian rocks are glacial and postglacial deposits related to the last major ice advance and melting of a continental ice sheet during the Great Ice Age. Low to moderate topography characterizes most of the area. Swamps and bogs are common in poorly drained areas.

Of particular importance with respect to the Spurline site are the glacial Lake Nipissing beach sands and gravels exposed within the pit. Carbon dating of wood found within the gravels showed an age of 6,500 years before the present. It was from these beach gravels that the granular materials were produced. Within the pit faces, well-developed accretionary beds dipped approximately 20 to 30 degrees toward Lake Huron. Faces within the pit showed well-developed imbricated zones, sand run, infill zones, and thin layers of open work, with fine granules considered as having been deposited at the swash-backwash zone during fair weather conditions. Fairly well sorted, medium-to-coarse sand was exposed in lensitic remnants. Lenses of fairly well sorted, fine-to-coarse subrounded gravel were common. Many of the gravel particles in the open-work zones were encrusted with iron-bearing brown coatings very similar to axle grease in feel, color, and texture.

In conjunction with W. Fyfe, Department of Geology, University of Western Ontario, it was verified that the greasy matrix material that encrusted the beach gravels was a complex, fine-grained product of the activity of an array of iron oxide bacteria. The material was largely composed of organic molecules with dispersed iron oxide, manganese oxide, and possibly clay minerals with particle sizes on the order of 0.01 μm and less. The surface area of these fine matrix materials was astronomical, as was their water-adsorbing capacity. Therefore, drainage through the granulars would be severely restricted.

In this case, it was apparent that certain conditions existed that assisted in the formation of the iron oxide bacteria discovered encrusting the gravels. They include

1. The presence of iron-bearing beds in the Huronian Supergroup (quartzite, feldspathic quartzite, and arkose),
2. A beach environment that contained imbricated open work (high porosity) of fine to coarse gravel clasts, and
3. The presence of a water body (carbon-rich bog or swamp providing organic matter and iron-, aluminum-, and manganese-rich waters, combined with a relatively high water table in the pit).

Iron oxide and algae bacteria have been found in similar deposits throughout the province, but they are uncommon and have not been recognized as a problem in the past. Now, when iron oxide bacteria are recognized, the source is not used for granular base or subbase.

Failures Due to Coarse Aggregate Breakdown

Oliver Quarry

The Oliver Quarry is located 2 km south of Perth in Lanark County. It was opened for local low-volume road construction. The quarry face consists of about 3 m of horizontally bedded orthoquartzitic sandstone of Early Ordovician age. The rock consists of alternating thin beds of hard and friable sandstone. The friable beds are weakly cemented and can be broken to their constituent sand grains by being hit with a hammer.

It is not known if the quarry was tested before being used. Testing results of rock samples taken from the face are shown in Table 3. It can be seen that the Los Angeles abrasion and impact loss is extremely high (89 percent loss). At the time of construction in 1967, there was no requirement in Ontario for granular base aggregates to meet any Los Angeles abrasion and impact loss requirement. The crusted stone met the gradation requirements for granular base use that required a minimum of 50 percent coarser than the 4.75-mm sieve and less than 8 percent passing the 75-μm sieve. Following construction, the road was left open to traffic over the winter before paving the following spring.

In the spring it was found that many of the coarse aggregate particles had broken down to sand-sized particles. This breakdown was caused by the abrasion and impact of vehicle tires on the particles. The pavement was rutted, had low stability, and showed some unevenness. It was found that, despite the breakdown, the pavement drained well. The upper surface of the sandy granular base was graded off into the shoulders, and new granular base was then placed before asphalt paving. As a result of this experience, specifications for maximum Los Angeles abrasion and impact losses of 60 percent for granular base aggregates were introduced in Ontario in 1970.

Parry Sound Quarry

Parry Sound Quarry is located at the intersection of Highways 124 and 69 in the district of Parry Sound. The rock was not tested before it was listed in the contract documents as being satisfactory for use as granular base and hot-mix asphalt aggregate. During blasting of bedrock adjacent to the unopened quarry for associated road construction, the potentially poor quality of the rock was noted, and a drilling and testing investigation was conducted during the winter when construction had ceased.

The bedrock at this location consists of a highly metamorphosed and strongly foliated amphibolite gneiss of Late Precambrian age. The rock has a dark grey color because of the presence of about 50 percent dark-colored amphibole (hornblende). The remainder of the rock consists of quartz and feldspar with small amounts of biotite mica. The degree of interlocking of the minerals is generally low because rounded grain boundaries are frequent. The lack of interlocking texture accounts for the generally weak nature of these rocks, which are thought to have been volcanic in origin, but high-grade metamorphism has destroyed all original minerals and textures.

Petrographic examination showed that the crushed rock was reasonably satisfactory, but particles smaller than 10 mm were often weak and brittle. Examination of the sand-sized portion found that about 25 percent of the particles could be easily broken by impact or rubbing into their constituent minerals, each grain being between 0.5 to 1.5 mm in diameter. The generally brittle nature of the rock was confirmed by the Los Angeles abrasion and impact losses of up to 60 percent (Table 3). The stone was also found to break down in the Proctor compaction test. It was concluded that the rock would make a marginal aggregate for use as granular base. The contractor, however, had already indicated his intention to use the quarry site as his source of aggregate. Directing the contractor to move to another site would have resulted in considerable extra costs. It was decided to use the quarry despite its marginal nature. The aggregate from the quarry performed satisfactorily as granular base, but it was noted that the stone did break down, and this led to some loss of bearing capacity and more rutting and potholing of the base than might normally have been the case. The source was not used for hot-mix asphalt aggregate.

New Aggregate Test Methods

In the early 1980s it was recognized that some of the traditional aggregate quality tests suffered from a number of defects. For instance, the Los Angeles abrasion and impact test, although of satisfactory precision, is done on oven-dried aggregate even though it is recognized that aggregates are always wet in the pavement and water significantly weakens some rock types. This test is useful for measuring the behavior of particles under a steel drum roller and for measuring breakdown during repeated handling and stockpiling, but it is not useful for identifying the majority of aggregates that cause granular base failure.

In 1985 MTO started looking for alternative wet abrasion tests to replace the Los Angeles abrasion and impact test. The test showing most promise was the Micro-Deval abrasion test (*4*), which was a development of the Deval abrasion test first used in the 1870s. The Deval abrasion test is still described in the AASHTO Manual (AASHTO T 4-35) but has been largely forgotten because of a very poor correlation with field performance. The Micro-Deval test is a wet abrasion and grinding test done in a 5-L steel jar containing 5000 g of 9.5-mm steel balls, 2 L of water, and a 1500-g test sample of 19- to 9.5-mm aggregate. The jar is turned at 100 rpm for 2 hr, and the amount of breakdown of the aggregate is measured by sieving the sample over a 1.18-mm sieve. Weak, water-susceptible aggregates break down rather easily and give relatively high losses. The precision of the test is good. For instance, for an average 12 percent loss in the test, the multilaboratory coefficient of variation is 6.4 percent of the test value, or, in other words, different laboratories will report values between 10.9 and 13.1 percent 19 times in 20. In France the test has been used for a number of years for evaluating granular base aggregates. More recently, it has been adopted in Quebec for selecting base course aggregates from limestone quarries of the St. Lawrence Lowlands.

The research methodology was to select about 100 coarse aggregates of known field performance from across Ontario. These aggregates were then tested in a variety of test procedures and the results evaluated. The aggregates were of good, fair, or poor performance. A good aggregate was one that had a long prior history of satisfactory use in a particular application. A fair aggregate was one that had been used, but at the time of use or subsequently, there had been a suspicion or report of performance problems without a complete failure. A poor aggregate was one that had been used and resulted in an early failure of the pavement. Alternatively, a poor rating was given to an aggregate of such unsatisfactory composition (shale, very shaley limestone) that, if it had been used, failure would have certainly occurred. Senior and Rogers (5) describe this research in more detail.

Figure 3 shows Los Angeles abrasion and impact loss against Micro-Deval abrasion loss. It can be seen that the Micro-Deval test is good at separating the majority

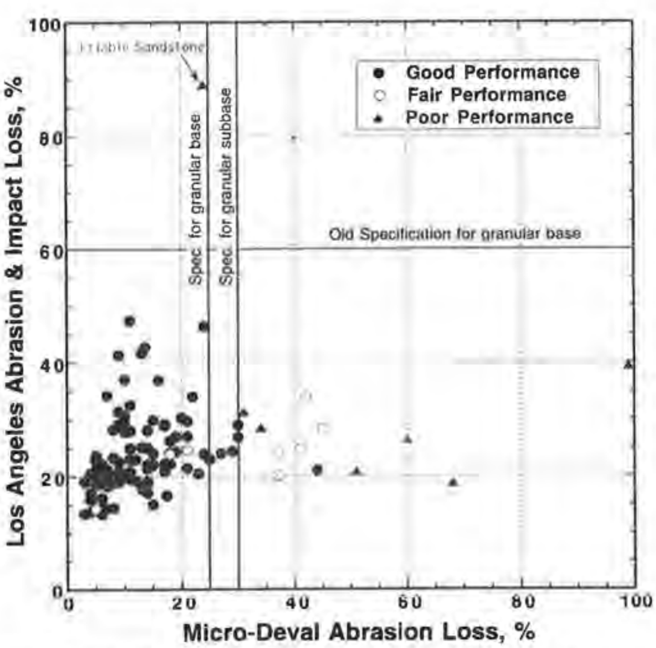

FIGURE 3 Los Angeles abrasion and impact loss value plotted against Micro-Deval abrasion loss value.

of poor aggregates from the good aggregates. Samples that are poor performers and fall at the boundary between the good and fair groups are shaley limestone and very shaley limestone. The one sample not detected by the Micro-Deval test was the friable sandstone from the Oliver Quarry. In 1994 MTO changed the specifications for granular base aggregates. The Los Angeles abrasion and impact test and petrographic examination requirement was abandoned, and a maximum Micro-Deval test loss of 25 percent for granular base and 30 percent for granular subbase was instituted. In addition, there are requirements that the material be nonplastic and that mica not constitute more than 10 percent of the material on the 75-μm sieve. Aggregates that contain more than 10 percent mica may be used if the field performance or results of permeability testing are satisfactory.

Conclusions

Further improvements of aggregate test methods that will provide more precise correlation with field performance and will be able to identify marginal or problem aggregates are being developed and implemented by the MTO to alleviate granular base failure problems. Placing good-quality aggregate materials in Ontario's low-volume roads is a good investment since many of these roads will become high-volume roads.

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