PDHonline Course C803 (4 PDH)

An Introduction to Hot Mix Asphalt for Pavement

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2020

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J. Paul Guyer, P.E., R.A.

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(The figures, tables and formulas in this publication are in some cases a little difficult to read, but they are the best available. **DO NOT PURCHASE THIS PUBLICATION IF THIS LIMITATION IS NOT ACCEPTABLE TO YOU**.)

1. GENERAL. Hot-mix asphalt is often used for high-performance pavements. The degree of performance required should be selected based on traffic conditions and the availability of satisfactory materials. Hot-mix asphalt mixtures consist of mineral aggregate and asphalt cement. These hot-mix asphalt mixtures are particularly suitable for airfield pavements, roads and streets, and storage areas. In general, from 3 to 6 percent asphalt is required for asphalt base or intermediate courses, 4 to 7 percent asphalt cement for surface courses, and 5 to 7 percent for porous friction courses. However, the optimum asphalt content should be determined according to appropriate mix design procedures. The aggregate gradations specified for hot-mix asphalt pavements are shown in table 2-1.

1.1 ADVANTAGES AND DISADVANTAGES. The hot-mix method of preparing paving mixtures provides for thorough coating of the aggregates with a uniform film of asphalt cement and accurate control of aggregate sizes and quantity of asphalt cement. Hot-mix pavements require no curing period after being laid and can be used as soon as the pavement has cooled. The paving mixture must be rolled to compact the mix while sufficiently hot because rolling is relatively ineffective after the mixture has cooled and the required density will not be achieved. Hot-mix pavements can be constructed rapidly with a minimum probability of damage to unfinished pavements from unfavorable weather conditions. Immediately after adequate rolling and a cooling period, the pavement has a high degree of stability from the interlocking of the coarse and fine aggregate and adhesion of the asphalt cement, as well as a high resistance to moisture penetration and frost damage.

1.2 USES. Hot-mix asphalt paving mixtures can be designed that are satisfactory for an asphalt base course, intermediate course, surface course, or porous friction course. Wheel loads, wheel spacing, tire pressures, intensity of traffic, and subgrade strength (California bearing ratio (CBR)) dictate the thickness of pavement (TM 5-825-2/AFJMAN 32-1014). Normally, asphalt base courses of any desired total thickness may be constructed in layers up to 150 millimeters (6 inches) thick. For airfield pavement applications hot-mix asphalt will be used as the intermediate and surface courses on types A, B, C, and D traffic areas, blast areas, and any other areas (even non-traffic) where their use is economical. There are four types of airfield traffic areas (A, B, C, and D) Hot-mix asphalt can be used on any road or street classification A through F. Porous friction courses shall be used primarily to prevent hydroplaning on runways or other high-speed pavements. Areas subjected to fuel spills will require an application of a coal tar sealer to protect the hot-mix asphalt pavement. When possible, the use of a rigid pavement should be investigated. Stone Matrix Asphalt is used in applications requiring a rut and abrasion resistant surfacing.

2. EQUIPMENT.

2.1 PLANT EQUIPMENT. The purpose of an asphalt plant is to produce a mixture properly coated with asphalt cement that consistently meets the requirements specified in the job mix formula (JMF) for aggregate gradation, asphalt content, and temperature. Control of the asphalt mixture quality must be initiated at the aggregate stockpiles. Each aggregate stockpile should be stored to prevent segregation or mixing with adjacent stockpiles.

2.1.1 BATCH PLANT.

2.1.1.1 GENERAL. A batch plant is illustrated in figure 2-1. Cold feed hoppers have individual feeders for each of the aggregates to be used in the mixture. These feeders must be set so that the desired percentage of each aggregate is fed into the plant. The rate of feed may be controlled by the gate opening, belt speed, or other methods depending on the type of cold feed. If the aggregate feeders are improperly set, a combination of the following problems may occur:

- One of the aggregate hot bins will overflow with material while another hot bin runs low on material.
- The gradation of the aggregate in the mix being produced will not meet the design gradation.
- The amount of natural sand may vary from design proportion and may exceed the amount allowed in the specifications.

2.1.1.2 COLD FEED BIN CALIBRATION. Before the start of a project the cold feed bins should be calibrated so that each bin will feed the desired rate of material. The cold feed calibration involves feeding one aggregate at a time onto a belt that is common to all aggregates. The speed of this belt should be determined prior to calibration of the feeders. One way to do this is to divide the belt length by

Table 2-1
Aggregate Gradations for Hot-Mix Asphalt Concrete Pavements

Sieve Size, mm	Gradation 1 19 mm Nominal Percent Passing by Mass	Gradation 2 12.5 mm Nominal Percent Passing by Mass	Gradation 3 9.5 mm Nominal Percent Passing by Mass
25.0	100	–	–
19.0	76-96	100	–
12.5	68-88	76-96	100
9.5	60-82	69-89	76-96
4.75	45-67	53-73	58-78
2.36	32-54	38-60	40-60
1.18	22-44	26-48	28-48
0.60	15-35	18-38	18-38
0.30	9-25	11-27	11-27
0.15	6-18	6-18	6-18
0.075	3-6	3-6	3-6
Sieve Size, inch	**Gradation 1 3/4 inch Nominal Percent Passing by Mass**	**Gradation 2 1/2 inch Nominal Percent Passing by Mass**	**Gradation 3 3/8 inch Nominal Percent Passing by Mass**
1	100	–	–
3/4	76-96	100	–
1/2	68-88	76-96	100
3/8	60-82	69-89	76-96
No. 4	45-67	53-73	58-78
No. 8	32-54	38-60	40-60
No. 16	22-44	26-48	28-48
No. 30	15-35	18-38	18-38
No. 50	9-25	11-27	11-27
No. 100	6-18	6-18	6-18
No. 200	3-6	3-6	3-6

Table 2-1

Aggregate gradations for hot-mix asphalt concrete pavements

the time required for one revolution. After the material is fed onto the belt, the material over a given length (for example; 2 meters (6 feet)) should be completely removed and weighed. The following relationship can be used to convert the weight of the sample taken to kilograms per hour (pounds per hour) and later to metric tons (tons) per hour:

$$R = \frac{3600\ WS}{L}$$

where

R = rate of feed, kilograms per hour (pounds per hour)

W = weight of sample, kilograms (pounds)

S = speed of belt, meters per second (feet per second)

L = length of belt sampled, meters (feet)

Each aggregate should be fed at four to five different feeder settings and the rate of feed determined; a plot of this data showing the relationship between rate of feed (kilograms or metric tons (pounds or tons) per hour) and feeder setting (gate opening, feeder belt speed, or other method for setting aggregate feeder) should be used for each aggregate. These plots can be used to set each cold feed bin to feed at the desired rate.

2.1.1.3 DRYER. After the aggregate cold feed bins have been properly set, the aggregate is carried up the cold elevator and through the dryer. The dryer removes the moisture from the aggregate and heats the aggregate to the desired temperature.

2.1.1.4 DUST COLLECTOR. A dust collector collects the dust created in the dryer and other plant components and adds all or any portion of it back to the mix at the hot elevator. The plant should have the capability to remove any desired portion of the collected dust from the mixture.

2.1.1.5 SCREENING. The aggregate exits the dryer and is carried, along with the returned dust, up the hot elevator, over the screening deck, and into the hot bins. Screen sizes are selected such that the oversize material will be rejected and the remaining aggregates are separated into various sizes. Ideally, the screen sizes should be selected so that the amount of material going into each hot bin is proportional to the relative volume of that hot bin. For example, suppose that hot bin No. 1 has a volume of 3 cubic meters (4 cubic yards), hot bin No. 2 has a volume of 1.5 cubic meters (2 cubic yards), and hot bin No. 3 has a volume of 1.5 cubic meters (2 cubic yards). Screens should be selected so that 50 percent of the material will go into bin No. 1, 25 percent into bin No. 2, and 25 percent into bin No. 3.

2.1.1.6 PERCENTAGE OF EACH HOT BIN. The percentage of each hot bin to be used in the mixture should be determined. Samples of each hot bin should be taken and the gradation for each sample determined. The percentage of each bin should be selected so that the gradation of the combined materials from the hot bins is equal to the JMF.

2.1.1.7 MIXING AGGREGATE AND ASPHALT. After the cold feed and hot bins are properly set, the combined aggregate from the hot bins is mixed with the proper amount of asphalt. The mixing time, generally 5 seconds for dry mixing and 25 to 40 seconds for wet mixing, should be selected so that all aggregate particles are coated. The plant should now be set to produce a uniform asphalt concrete mixture having proper aggregate gradation, asphalt content, and temperature. The batch plant weighs the various nominal size aggregates and asphalt to produce a batch of material that is then mixed for a specified period of time.

2.1.2 DRUM MIXER.

2.1.2.1 GENERAL. The asphalt plant that has become popular throughout the paving industry is the drum mixer (figure 2-2). The drum mixer is less expensive than the batch plant and generally produces material at a higher production rate. When a drum mixer is used, the gradation must be closely controlled at the cold feed bins because no additional screening of the mixture occurs. The drum mixer is frequently used in the production of recycled hot-mix asphalt as well as conventional hot-mix asphalt.

2.1.2.2 COLD FEED BIN CALIBRATION. The cold feed bins are set up much the same way as for the batch plant, but the drum mixers should have a weight sensor on the aggregate feed belt that weighs a given length of the loaded belt. The asphalt pump adds binder based on the belt measured weight of aggregate. Thus, to calibrate the cold feeds, each aggregate can be fed onto the belt at various gate openings or individual belt speeds, weighed, and the feed rate computed. These steps should be followed for each of the aggregates to be added to the mixture, and a calibration curve should be developed.

2.1.2.3 DRYER. For the drum mixer the burner for the dryer is usually located on the high side of the drum. The aggregate enters the dryer just below the burner and helps to shield the asphalt binder from direct contact with the flame. The asphalt cement is added to the dryer at approximately the midpoint to two-thirds the length to prevent close contact with the flame, which could cause over-heating and damage the asphalt binder. A double barrel drum mixer has the burner on the low end of the drum and is more efficient than a conventional drum mixer. Information concerning the addition of a recyclable asphalt pavement to a drum mixer is available in the technical literature.

2.1.3 ASPHALT MIXTURE STORAGE SILO. Asphalt storage silos are used to store hot-mix asphalt mixture before loading onto trucks. Thus, plants can run continuously even when there is a temporary shortage of trucks. Material can be stored in silos for short periods of time, but if stored too long, the material may cool excessively or may oxidize excessively causing the bituminous binder to become hard and brittle. With some mixes the asphalt cement may tend to drain from the aggregate. As a general rule, hot-mix asphalt mixtures should not be stored more than 4 hours regardless of the type of storage silo used. If segregation of aggregate or draindown of asphalt cement occurs in the silo, use of the silo should be disallowed or changes should be made to prevent segregation and draindown.

2.2 PLACEMENT EQUIPMENT.

2.2.1 ASPHALT SPREADER (PAVER).

2.2.1.1 TYPES OF SPREADERS. An asphalt spreader is used to place most mixture types such as hot mix, cold mix, and base course material. Spreaders currently in use operate on either tracks or rubber tires and most have a vibrating screed to strike off and smooth the paving mixture. Some spreaders use a tamping bar in conjunction with the screed, or an oscillating screed with a vibrating compactor, and others use a vibrating screed for both strike-off and initial compaction. Conventional paving machines are capable of placing hot-mix paving mixtures satisfactorily, provided they are maintained in good mechanical condition, kept properly adjusted, and operated by experienced personnel. Poor pavement surfaces result if the screed plates are worn or rusty or if the tamping bars are worn or not properly adjusted.

2.2.1.2 AUTOMATIC GRADE CONTROL. Asphalt spreaders should have a means of automatically controlling the grade. If an automatic grade control device is used on the spreader for constructing pavements that consist of two paving lanes, it should include a sensing device for grade control of one end of the screed and a slope-control mechanism for control of the other end of the screed or a grade control sensing device on each end of the screed. Where the paver is used for constructing pavements with multiple paving lanes (more than two paving lanes), sensing devices will be used on each side of the spreader for control of the screed. The slope-control mechanism should not be used for grade control in multiple paving lane operation.

2.2.2 JOINT HEATERS. Joint-heating devices for attachment to asphalt spreaders have been used on construction projects. They are used to heat the edge of an adjacent pavement lane during placement so that a hot joint is obtained. Experience with joint heaters has shown that there is a danger of overheating the existing asphalt mixture. Accordingly, it is the policy of some Owners that pavement joint heaters will not be used without the written authorization of the Owner. If a contractor should desire to use a pavement joint heater, a request will be submitted to the Owner. To assure that the asphalt mixture will not be detrimentally affected, the request will include a description of the controls for the proposed joint heater.

2.2.3 ASPHALT DISTRIBUTOR. Asphalt distributors are used to apply asphalt material evenly over a surface. All nozzles should be free and open, and should be the same size and at the same angle with reference to the spray bar to produce a uniform fan of bituminous material. The height of the spray bar above the surface is important for uniform application. When the bar is too high or too low, a difference in application rate in the middle of the spray fan and at the ends will occur, causing streaking. The height of the spray bar should be adjusted so that a double or triple overlap of the spray fan is obtained. The Asphalt Institute's Manual Series No. 13 offers guidance for calibrating and checking application equipment.

2.2.4 ROLLERS.

2.2.4.1 ROLLER TYPES. A number of roller types are being used for paving operations. Rollers used to compact asphalt mixtures are static steel-wheel, vibratory steel-wheel, and rubber-tired rollers.

2.2.4.1.1 STATIC STEEL-WHEEL ROLLERS. The static steel-wheel rollers consist of two-wheel (tandem), and three-wheel (tricycle) versions. These rollers are generally used for breakdown and finish rolling. Static steel-wheel rollers leave a smooth finish on the pavement surface, but excessive rolling may result in lateral movement of the mixture causing surface cracking and a general loss in density. These rollers should be equipped with a system for watering the drums and should have scrapers to remove any material that sticks to the drums.

2.2.4.1.2 VIBRATORY STEEL-WHEEL ROLLERS. The vibratory steel-wheel rollers are commonly used for compacting hot-mix asphalt mixtures. They may consist of dual-drum vibration, single-drum vibration and single-drum static, or single-drum vibration and rubber tires on the rear axle. These rollers can be used for breakdown, intermediate, and finish rolling. Breakdown rolling can be performed in either static or vibratory mode. Intermediate rolling is performed in the vibratory mode while finish rolling is performed in the static mode. The Owner may require a maximum of 2 passes in the vibratory mode. Although the vibratory roller is used for intermediate rolling, it does not replace a rubber-tired roller. The vibratory roller should have a watering system on steel drums and rubber tires (if applicable) along with scrapers on the steel drums and scrapers and pads on the rubber tires.

2.2.4.1.3 RUBBER-TIRE ROLLER. Rubber-tired rollers are used for intermediate rolling of hot-mix asphalt mixtures. These rollers provide for an increase in compaction after breakdown rolling and produce a watertight surface. A large rubber-tired roller (capable of being loaded to a minimum of 2,043 kilograms, 4,500 pounds per tire and capable of minimum tire inflation pressure of 620 kPa, 90 psi) should be available for construction of heavy-duty pavements on roads or airfields. The rubbertired roller should have a watering system for the tires and should have scrapers and pads to prevent accumulation of materials on tires. A large rubber-tired roller should be used for compaction of all heavy-duty hot-mix asphalt pavements.

2.2.4.2 OPERATION OF ROLLERS. Rollers should generally be operated at or below a rate of 4.8 to 8 kilometers per hour (3 to 5 miles per hour) (fast walking speed). Starts and stops should be gradual to avoid damaging the freshly laid mixture. Quick turns or any turns that cause cracking on freshly laid mixture should not be allowed.

3. MATERIALS.

3.1 ASPHALT MATERIALS. Asphalt materials used in hot-mix paving operations include the products conforming to the specifications listed in table 2-2. The grades of asphalt specified by AASHTO MP-1 are the performance grades of asphalt developed as part of the Strategic Highway Research Program (SHRP). This grading system is currently being implemented and, within the U.S., is the method most often used to specify asphalt cement. The PG system has advantages over conventional grading systems. The specific gravity of the asphalt cement shall be obtained using ASTM D 3142. This value is sometimes required to compute a theoretical maximum specific gravity and for mixture void calculations. The maximum theoretical specific gravity can also be determined using ASTM D 2041. Asphalt cements for use in pavement design and construction are graded or classified in one of three ways. They can be graded on the basis of penetration ASTM D 946, on the basis of viscosity, ASTM D 3381; or by the performance grading system AASHTO MP-1. Currently, in the continental United States (CONUS), performance grades of asphalt are common. However, outside the continental United States (OCONUS), penetration grades of asphalt may be more easily obtained. When PG binders are used, paragraphs (1), (2), and (3) on asphalt selection should be disregarded and the procedures from AASHTO MP-1 and information given in paragraph (4) should be used for selection. In general, the softest grade of asphalt cement consistent with traffic and climate should be used. Selecting a grade of asphalt cement should be based on several items. Among the most important are climate, traffic conditions, economics of asphalt availability, and previous regional experiences. Traffic conditions and economic considerations will vary from project to project, but environmental conditions and regional experiences should have some similarity. For example, in warm and hot regions one should ensure that the mix is stable during the summer months, and in cold regions one should ensure that the mix is not prone to cracking during winter months. There are additional requirements for asphalt cements that will perform satisfactorily in very cold climates such as Alaska, Greenland, and the northern continental United States.

Table 2-2
Specification References for Asphalt Materials

Bitumen Type	Specification
Asphalt cement (Performance graded asphalt binder)	ASTM D 946, D 3381 (AASHTO MP-1)
Cutback asphalt (slow-curing type)	ASTM D 2026
Cutback asphalt (medium-curing type)	ASTM D 2027
Cutback asphalt (rapid-curing type)	ASTM D 2028
Asphalt, emulsified	ASTM D 977
Asphalt, cationic emulsified	ASTM D 2397
Rubberized tar cement	ASTM D 2993
Tar	ASTM D 490 and D 2993

Table 2-2

Specification references for asphalt materials

3.1.1 ASPHALT CEMENT SELECTION BY TEMPERATURE REGION.

3.1.1.1 DETERMINING TEMPERATURE REGION. Table 2-3 gives guidance for selecting an asphalt cement by temperature region. Climatological data are required to provide input into the selection method. First, average monthly maximum temperature data are required to compute a pavement temperature index (PTI), when project locations have average monthly maximum temperatures above 23.9 ^{0}C (75 ^{0}F), the PTI is defined as the sum of the monthly increments exceeding 23.9 ^{0}C (75 ^{0}F). Conversely, when no average monthly temperature exceeds 23.9 ^{0}C (75 ^{0}F), the PTI is defined as the difference between the highest average maximum temperature for the warmest month and 23.9 ^{0}C (75 ^{0}F).

3.1.1.2 EXAMPLE OF CALCULATIONS FOR PAVEMENT TEMPERATURE INDEX. The method for calculating the pavement temperature index for two construction sites is given in this example. The average monthly maximum temperature and the difference above 23.9 ^{0}C (75 ^{0}F) for Site A and Site B are given below.

Table 2-3
Asphalt Cement Selection Criteria Based on Pavement Temperature Index

Pavement Temperature Index, Cumulative °C (°F)	Temperature Region	Asphalt Cement Selection Criteria
Less than 16.7 (30)	Cold	Penetration-viscosity method for cold regions (table 5)
16.7 to 44.4 (30 to 80)	Warm	85 to 100 penetration (original asphalt)
Greater than 44.4 (80)	Hot	60 to 70 penetration (original asphalt)

Table 2-3

Asphalt cement selection criteria based on pavement temperature index

The temperature index at these sites is the sum of the increments of average monthly maximums above 23.9 ^{O}C (75 ^{O}F); therefore, the pavement temperature index for each site is as follows:

Site A = 54.2, cumulative ^{O}C (98.0, cumulative ^{O}F)

Site B = 1.1, cumulative ^{O}C (2.0, cumulative ^{O}F)

Based on the criteria shown in table 2-3, Site A is a hot region, and Site B is a cold region.

	Site A		Site B	
Month	Average Maximum Temperature °C	Difference Above 23.9°C	Average Maximum Temperature °C	Difference Above 23.9°C
January	15.8	–	-1.2	–
February	20.3	–	-2.3	–
March	23.2	–	6.1	–
April	26.6	2.7	14.6	–
May	31.4	7.5	19.6	–
June	34.7	10.8	21.3	–
July	36.4	12.5	25.0	1.1
August	33.3	9.4	23.4	–
September	32.3	8.4	19.4	–
October	26.8	2.9	14.2	–
November	23.3	–	6.3	–
December	15.7	–	2.7	–
Cumulative total		54.2		1.1

3.1.1.3 COLD REGION REQUIREMENTS.

3.1.1.3.1 DETERMINING THE DESIGN AIR FREEZING INDEX. When it is determined that a project will exist in a cold region, as defined in table 2-3, additional climate data are required. For the project area under consideration, a design air freezing index (DFI) is also required to further satisfy cold region requirements. DFI's are used to differentiate between climates in cold temperature regions. A DFI of 1,667 degree-Celsius-days or 3,000 degree-Fahrenheit-days (degree-days) is used as the delineation between moderately cold and severely cold (extremely cold) climates. Moderately cold climates have DFI's up to 1,667 degree-days, and severely cold climates have DFI's greater than 1,667 degree-days.

3.1.1.3.2 PENETRATION-VISCOSITY NUMBER. Cold regions are areas where the penetration viscosity number (PVN) method is used to aid in selecting an asphalt cement. Site B in the previous example would require the use of the PVN method to select an asphalt cement. Asphalt cement factors considered in the original correlation were penetrations at 25 ^{O} C (77EF), viscosity at 135 ^{O} C (275 ^{O} F), and penetration index. The PVN method is used to quantify temperature susceptibility of an asphalt cement and estimate its ability to resist low-temperature cracking. Required input data are penetration at 25 ^{O} C (77 ^{O} F) and kinematic viscosity at 135 ^{O} C (275 ^{O} F). Figure 2-3 allows estimation of PVN for asphalt cements in cold regions. Table 2-4 provides minimum PVN selection criteria for asphalts in cold regions. Table 2-4 and figure 2-3 should always be used when selecting asphalts for use in cold regions unless performance graded asphalt cements are used. Table 2-4 also shows requirements for roads and other pavements. A design index is required for roads and other pavements; it is an index of the severity of traffic estimate. Temperature at a 5 centimeter (2-inch) depth of pavement can be estimated from a DFI for a given project location or site as shown in figure 2-4. This "minimum anticipated pavement temperature" and minimum PVN criteria of table 2-4 can be used with figure 2-3 to select an asphalt cement. An asphalt with given penetration and viscosity can be checked for satisfying PVN criteria of table 2-4 by plotting in figure 2-3. If this penetration and viscosity point falls on or above the minimum PVN value and to the right of the minimum anticipated pavement temperature, it is estimated that low temperature contraction cracking of the asphalt concrete layer will be prevented. If it plots to the left of the anticipated pavement temperature, the pavement will likely crack at low temperature. PVN values should be calculated for more accurate results.

Table 2-4
Minimum PVN Selection Criteria for Asphalt Cements in Cold Region Use

		Road and Other Pavement by Design Index	
Cold Region	Airfields	4	> 4
Moderate cold (DFI 1,667 degree-days[1])	- 0.5	- 0.5	- 0.5
Severe cold (DFI > 1,667 degree-days[1])	- 0.2	- 0.5	- 0.2

[1] Degree-Celsius-days (3,000 degree-Fahrenheit-days).

Table 2-4

Minimum PVN selection criteria for asphalt in cold region use

3.1.2 EXAMPLES OF ASPHALT CEMENT SELECTION IN THE THREE REGIONS.

3.1.2.1 ASPHALT CEMENT SELECTION IN A HOT REGION. A parking lot is to be built in a region that has a pavement temperature index of 54.4, cumulative ^{O}C (98, cumulative ^{O}F). An asphalt supplier can provide asphalt cements that meet the requirements in table 2 from ASTM D 3381. Viscosity and penetration data for the asphalt cements are given below.

	AC-10	AC-20	AC-40
Viscosity, 60°C (140°F), P	872	2,200	4,104
135°C (275°F), cSt	298	435	605
Penetration, 25°C (77°F), 0.1 mm	123	70	46

From table 2-3, an asphalt cement that has a penetration of approximately 60 to 70 should be selected. The AC-20 asphalt cement should be selected for this pavement.

3.1.2.2 ASPHALT CEMENT SELECTION IN A WARM REGION.

3.1.2.2.1 ASPHALT CEMENT (D 3381) ORIGINAL ASPHALT CEMENT. A street is to be constructed in a region that has a pavement temperature index of 23.3, cumulative ^{O}C (42, cumulative ^{O}F). An asphalt supplier can provide asphalt cements that meet the requirements in table 2 from ASTM D 3381. Viscosity and penetration data for the asphalt cements are given below.

	AC-5	AC-10	AC-20
Viscosity, 60°C (140°F), P	560	1,120	2,170
135°C (275°F), cSt	180	335	450
Penetration, 25°C (77°F), 0.1 mm	145	96	70

Based on table 2-3, an asphalt cement that has a penetration of approximately 85 to 100 should be selected. The AC-10 asphalt cement is selected.

3.1.2.2.2 ASPHALT CEMENT (D 3381 - RESIDUAL ASPHALT CEMENT). A parking lot is to be constructed in a region that has a pavement temperature index of 23.2, cumulative ^{O}C (42, cumulative ^{O}F). An asphalt supplier can provide asphalt cements that meet the requirements in table 3 from ASTM D 3381. Viscosity and penetration data for the asphalt cements are given below.

	AR-1000	AR-2000	AR-4000
Viscosity, 60°C (140°F), P	851	1,962	3,544
135°C (275°F), cSt	162	247	334
Penetration, 25°C (77°F), 0.1 mm			
Original	145	87	53
Residue	99	55	39

Based on table 2-3, an asphalt cement that has a penetration of approximately 85 to 100 should be selected. The AR-2000 asphalt cement is selected based on the original penetration of the material.

3.1.2.2.3 ASPHALT CEMENT SELECTION IN A COLD REGION. At a location in Watertown, NY, a heavy duty open storage area (design index of 0) for use by 22,680 kilogram (50,000 pound) forklift trucks has to be constructed in a region with a pavement temperature index of 1.1, cumulative ^{O}C (2, cumulative ^{O}F) and a DFI of 1,278 degree-Celsius-days (2,300 degree-Fahrenheit-days) calculated. An asphalt supplier can provide two asphalt cements that meet the requirements in table 2 from ASTM D 3381. Viscosity and penetration data for the asphalt cements are given below.

	AC-2.5	AC-5
Viscosity, 60°C (140°F), P	280	466
135°C (275°F), cSt	180	220
Penetration, 25°C (77°F), 0.1 mm	296	240

3.1.3 ANALYSIS AND ASPHALT SELECTION. The climatological data allow classification of the site by temperature region and allow an estimate of pavement temperature. According to table 2-3, the pavement temperature index classifies the site as a cold region where the PVN method should be used to select the grade of asphalt cement. The DFI allows the use of figure 2-4 to estimate a minimum pavement temperature at a 5 centimeter (2-inch) depth. From figure 2-4, a minimum anticipated pavement temperature is about -30 ^{O}C (-22 ^{O}F). Table 2-4 shows that this cold region can be further classified as a moderately cold region since its DFI is less than 1,667 degree-Celsius-days. Table 2-4 also indicates that the required PVN of the asphalt selected must be greater than !0.5 for a design index of 10. This will minimize low temperature pavement cracking. Now, PVN values must be determined for the available asphalt cements. This can be done by either plotting penetration and viscosity at 135 ^{O}C (275 ^{O}F) in figure 2-3 or by using PVN equations. If the details of figure 2-3 are not sufficient to accurately determine PVN values, equations should be used. The general PVN equation is as follows:

$$PVN = \frac{(L - X)(-1.5)}{(L - M)}$$

where

L = logarithm of viscosity in centistokes at 135°C (275°F) for a PVN of 0.0 at the given penetration

X = logarithm of viscosity in centistokes at 135°C (275°F) of a given asphalt

M = logarithm of viscosity in centistokes at 135°C (275°F) for a PVN of -1.5 at the given penetration

Values of X can be determined directly from asphalt cement viscosity data as provided in this example, but values of L and M are a function of the penetration values of each asphalt. Equations for the values of L and M are:

$$L = 4.25800 - 0.79674\ LOG\ (PEN)$$

and

$$M = 3.46289 - 0.61094\ LOG\ (PEN)$$

where PEN = penetration at 25°C (77°F) of a given asphalt cement.

Calculated PVN values of the two available asphalt cements are:

PVN = -0.638 for AC-2.5

PVN = -0.081 for AC-5

Based on table 2-4, an asphalt cement that has a PVN greater than -0.5 and lies on or to the right of the minimum temperature diagonal line should be selected. The AC-5 asphalt cement is selected because it has a PVN of -0.081 and lies to the right of the -30 ^{0}C (-22 ^{0}F) temperature diagonal line. This asphalt cement satisfies the requirements of table 2-4 and should prevent low-temperature pavement cracking.

3.1.4 SHRP PERFORMANCE GRADING (PG) OF ASPHALT CEMENTS.

3.1.4.1 GENERAL. The SHRP PG system (AASHTO MP1-93) classifies asphalt binders according to the temperatures at which certain performance-related properties are met (AASHTO PP6-93). These specifications will have replaced penetration and viscosity graded asphalts. The specifications can be applied to unmodified and modified asphalt cements. The SHRP performance grading procedures must be applied with caution to certain polymer modified asphalts binders (PMAB). The SHRP PG specifications are built around viscoelastic properties such as complex modulus, G*, phase angle, ä, low temperature stiffness, S, and creep rate, m. The criteria for G*/sin ä (the SHRP rutting parameter) are designed to insure a minimum stiffness of the binder immediately after placement to avoid "tender" mixtures and those mixtures with rutting potential early in the pavement life based on an estimated high pavement temperature. The high pavement temperature is determined from the mean 7-day high air temperatures to obtain an estimate for the pavement temperature. The maximum for G*@sin ä (the SHRP fatigue parameter) helps to identify binders that may be susceptible to fatigue damage as well as those exhibiting excessive embrittlement with age. Temperature data for specification of a binder for a particular region can be obtained from local weather stations or from an extensive database compiled by the FHWA (Federal Highway Administration). The database contains information from thousands of locations in the U.S. and Canada and is available in the SHRPBIND software. The SHRPBIND program is available at no cost from the FHWA. The software calculates the 7-day mean maximum pavement temperatures and the lowest yearly one-day pavement temperatures to determine the SHRP PG for the area selected. The calculations are based on air temperatures, average sunlight, and location. The software does not contain the low temperature modification equation above. SHRP PG's are calculated by the SHRPBIND program along with the reliability for the given area. For example, a PG64-22 binder refers to a material with the following properties: (1) a minimum flash point of 230 ^{0}C, (2) a maximum rotational viscosity of 3 Pa@sec at 135 ^{0}C, (3) a minimum of 1000 and 2200 Pa for G*/sin ä for the original (tank) and RTFOT-conditioned (Rolling Thin Film Oven Test) materials, respectively, at a 10 radian/sec oscillatory shear and 64EC, (4) a maximum of 5 MPa for G*@sin ä for the PAV-aged (Pressure Aging Vessel) material at 10 radian/sec oscillatory shear and 25EC, and (5) a maximum stiffness of 300 MPa and a minimum creep slope of 0.3 at -12EC for the PAV-aged material. This binder would be suitable in areas with a maximum pavement temperature of 64EC and a minimum pavement temperature of -22 ^{0} C.

3.1.4.2 USE OF PERFORMANCE GRADED BINDERS ON AIRFIELDS.

Performance graded binders can be used on airfields with some restrictions. A typical airfield pavement experiences much higher loads than a highway due to heavily-loaded cargo aircraft and high-performance fighter aircraft with high tire pressures. To avoid rutting, higher loads require a more stable asphalt binder, especially at higher pavement temperatures. In addition, most airfields suffer more from environmental distresses such as low temperature cracking and oxidative hardening of the asphalt (which leads to cracking). Thus, for airfields, use of SHRP performance graded asphalts above and below the recommendation for the given region is warranted. For example, if an airfield that is subjected to heavy cargo aircraft traffic is located in a region that requires a PG64-22, use of a PG 70-28 or 76-28 will provide additional insurance against rutting and cracking. The added cost of producing an enhanced PG asphalt must be balanced against expected benefits and life cycle costs.

3.4.1.3 POLYMER MODIFICATION OF ASPHALTS. The addition of polymers to asphalts is a burgeoning industry. Many polymers greatly improve the stiffness and flow characteristics of asphalts at high temperatures and are being demonstrated in pavement applications to significantly reduce rutting where this has been a problem in the past. The higher temperatures often required by the modification can cause difficulty with mixing, placement, and compaction. New classes of polymers have become available that are designed to improve the low temperature characteristics of an asphalt and should improve the ability of the pavement to resist low temperature-induced cracking. The selection of a modifier may be based on a range of factors that include availability, cost, properties, and familiarity with the product. Due to differences in chemistry of asphalts, a particular polymer used with one asphalt source will likely not yield the same physical properties as the same polymer with a different asphalt source. Of primary importance in the selection of an appropriate polymer is the phase separation characteristics of the asphalt/polymer combination during storage to ensure homogeneity of the binder prior to mixing with aggregate and during testing of the material. Phase separation can occur in the storage tank if the PMAB is not properly dispersed, leading to a heterogenous binder in the binder/ aggregate mix which may affect performance. The SHRP PG specification is currently the best utility available for judging the expected performance of a PMAB. Although, not perfect, performance grading offers much more pertinent information about the properties of a PMAB than conventional grading. Many state highway agencies have produced specifications for local use of PMABs that are modifications of current conventional tests. However, these specifications are often built around specific asphalts and specific polymers, are not of general use, and can cause problems when asphalt or polymer sources change. The SHRP PG binder specification was originally intended to be a “blind” specification where all binder materials (modified and unmodified) would be evaluated on the basis of properties that relate directly to performance. However, many commercial asphalt modifiers cannot be properly evaluated using some of the SHRP aging practices due to phase separation and some of the performance-related properties may not be applicable to some PMABs. The direct tension test must be employed according to current use practice (as of March, 1998, this is the horizontal test arrangement using deicing fluid as the bath medium and metal binder molds) on all PMABs to verify the low temperature grade determined by bending beam rheometry. The modified asphalts must be shown to not be prone to phase separation or gross morphology changes during the SHRP performance grading procedures. This may be addressed by applying phase separation testing (ASTM D-5892) to determine the ring and ball softening point difference between upper and lower sections of heated tubes of binder. A guideline is that this difference should be no more than 4 ^{0}C. In general, the maximum range of temperatures that a typical unmodified asphalt (non air-blown) demonstrates in SHRP grading is between 80 and 90 ^{0}C. For instance, a typical AC-20 or AC-30 viscosity graded asphalt may yield a PG58-22 or PG64-22 with a temperature use range of 80 and 86 ^{0}C, respectively. To extend this range past 90 ^{0}C, modifiers (primarily polymers) are added to asphalt binder. A typical polymer modified AC-20 or AC-30 will yield a PG70-22 or PG76-22 depending on the amount of polymer added. However, some modifiers may extend both the high and low temperature grades. Softer asphalt grades (such as an AC-5 or AC-10) combined with a polymer modifier can be used to yield a binder with better low temperature properties for a given region while maintaining the necessary high temperature properties.

3.4.2 AGGREGATES. Aggregates for use in hot mix asphalt should be clean, hard, and durable. Angular aggregates provide more stable hot mix asphalt mixtures than do rounded aggregates.

3.4.2.1 SIEVE ANALYSIS. Aggregates to be used in a paving mix, as listed in table 2-1, should be subjected to a sieve analysis. An experienced engineer can obtain information from an aggregate's grading curve concerning the suitability of the aggregate for a paving mix, the quantity of asphalt cement required, and whether mineral filler should be added. Sieve analyses of fine and coarse aggregates shall be conducted according to ASTM C 136.

3.4.2.2 SPECIFIC GRAVITY. Specific gravity values for aggregates used in paving mixture are sometimes required in the computation of percent voids total mix and percent voids filled with asphalt in the compacted specimens. Criteria have been established to specify limiting values for these void properties. Therefore, specific gravity values must be carefully determined following specified procedures to insure that the criteria are properly applied. Two different methods can be used for determination of the theoretical maximum specific gravity of a mixture. The selection of the appropriate test procedure depends in part on the water absorption of each aggregate blend.

3.4.2.2.1 APPARENT SPECIFIC GRAVITY OF AGGREGATE. The apparent specific gravity of the fine and coarse aggregate can be used to compute the theoretical maximum specific gravity with aggregate blends showing water absorption of less than 2.5 percent. The apparent specific gravity shall be determined as described in ASTM C 127 for coarse aggregate, ASTM C 128 for fine aggregate, and ASTM C 188 or D 854 (whichever is applicable) for mineral filler. Properly weighted values, based on the amount of each type of material in a given blend, should be used in computations subsequently discussed.

3.4.2.2.2 THEORETICAL MAXIMUM SPECIFIC GRAVITY OF MIXTURE. The theoretical maximum specific gravity can be determined by the test method described in ASTM D 2041. This test is conducted on the asphalt mixture and does not require a specific gravity test on the individual aggregates. The theoretical maximum specific gravity can be used to back calculate the effective specific gravity of the aggregate. This method can be used for aggregate blends having any amount of water absorption.

3.4.2.2.3 ABRASION AND IMPACT RESISTANCE OF COARSE AGGREGATE. The determination of percent loss for coarse aggregates may not be necessary if the aggregate has been found satisfactory by previous tests and/or performance. However, coarse aggregates obtained from new or doubtful deposits shall betested for resistance to degradation by evaluating the conformance to specification requirements for percent loss as measured using the Los Angeles Machine (ASTM C 131).

3.4.2.2.4 SOUNDNESS TEST. The soundness test is used where damage from freezing is expected to be a problem. The soundness test should not be performed on aggregate that has been found satisfactory by previous tests or performance data. However, aggregate obtained from new or doubtful deposits will be tested for conformance to specification requirements using the sodium sulfate or magnesium sulfate solution tests (ASTM C 88).

3.4.2.2.5 PERCENT CRUSHED PIECES. The percentage of crushed pieces in both the coarse aggregate and fine aggregate fractions must be sufficiently high to promote stability in hot-mix asphalt mixture. A description of a proper crushed face and the required percentage of crushed aggregate particles shall be specified in the contract specifications.

3.4.2.2.6 PARTICLE SHAPE. The particle shape of crushed aggregates is required to be essentially cubical. Flat and elongated aggregate particles are susceptible to breakage under compaction and subsequent traffic. The quantity of flat and elongated particles shall be tested for conformance to specification requirements using ASTM D 4791.

3.4.2.2.7 NATURAL SAND CONTENT. Natural sand is defined as any fine aggregate material other than that produced by mechanically crushing larger rocks and aggregates. Natural sands tend to be rounded particles which, when used in excess, can cause instability in the hot-mix asphalt mixture. For airfield or high-pressure mixtures the percentage of natural sand shall not exceed 15 percent of the combined weight of the coarse aggregate, fine aggregate, and the material passing the 75 Fm (No. 200) sieve. This percentage can increase to 25 percent for roadway (low-pressure) mixtures. The limitation on the percentage of natural (uncrushed) sand in the mixture assures a strong and stable pavement under traffic.

3.4.2.2.8 FINE AGGREGATE EFFECT. The uncompacted void content (C 1252, Method A) will help define the angularity of the fine aggregate. The lower limit should be 45, unless local experience indicates that aggregates with a lower value can provide good performance. Generally, a value of 43 should be the lowest value accepted.

3.4.2.2.9 VOIDS IN THE MINERAL AGGREGATE (VMA). The volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and volume of the asphalt not absorbed into the aggregates.

3.4.2.2.10 COMBINING AGGREGATES. When asphalt mixtures are produced, aggregates from two or more sources must be combined. Methods and procedures described in this manual will permit determination of the most suitable aggregate blend available and will prescribe the proper asphalt content for the particular aggregate blend determined to be the most suitable. Whenever an asphalt mixture does not meet established criteria, either the gradation of the aggregate must be improved, another aggregate must be used, or the asphalt content must be modified. The choice as to improvement of gradation or the use of another aggregate is a matter of engineering judgment involving an analysis of the available aggregate supplies and cost considerations.

3.4.3 MINERAL FILLERS.

3.4.3.1 GENERAL. Some mineral fillers are more desirable in asphalt paving mixtures than others. For example, fine sands and clays are less suitable fillers than limestone filler or portland cement, and well-graded materials are more suitable than poorly graded materials. Satisfactory pavements may be designed using commercial fillers that conform to ASTM specifications. The apparent specific gravity of the mineral filler is required to perform a void computation except when ASTM D2041 is used. The specific gravity will be determined following ASTM D 854 or ASTM C 188 procedures (as appropriate), except when ASTM D 2041 is used, in which case the mineral filler shall be included in the blended aggregate.

3.4.3.2 ADDITION OF MINERAL FILLER. The filler requirements of each aggregate blend must be estimated after the blends to be tested in the laboratory have been selected. The quantity of mineral filler to be added generally depends on the amount of filler naturally present in the aggregate. The amount of filler that exists naturally in most aggregates is sufficient to produce satisfactory hot-mix asphalt. Research has indicated that under normal circumstances, the addition of mineral filler reduces the quantity of asphalt cement required for the paving mixture. The addition of a satisfactory mineral filler within practical limits also increases the stability of a paving mixture. Excessive amounts of filler, however, may decrease the durability of the paving mixture because of the decrease in asphalt cement film thickness. Practical considerations and optimum performance usually will dictate quantities of about 5 percent filler for hot-mix asphalt and 10 percent for sand-asphalt mixtures.

3.4.4 ANTISTRIP AGENTS.

3.4.4.1 GENERAL. Several antistrip agents have been successfully used to reduce the probability of the asphalt stripping from the aggregate. Some antistrip agents are added to the asphalt binder before it leaves the refinery, while others are added directly into the mixer as mineral filler. The immersion compression test described in CRD-C 652-95 is used to evaluate the stripping property of a dense-graded bituminous hot mix.

3.4.4.2 RECOMMENDED PROCEDURE. The recommended procedure for improving the resistance of an aggregate to stripping is to add approximately 1 percent by weight hydrated lime to the mixture. This 1 percent lime must be included in the determination of the aggregate gradation.

3.4.5 ANTIFOAM AGENTS. Silicone additives or modifiers can reduce the effects of moisture or other conditions in asphalt mixtures. Silicone additives have been successfully used to suppress foaming of asphalt in asphalt plants. The silicone that has been used for this purpose is mixed at a rate of 1 milliliter per 640 liters (1 ounce per 5,000 gallons) of asphalt. The recommended range is also given as 1 to 2 parts per million. Silicones have been used to reduce the hardening of hot-mix asphalt while in storage silos. Silicone additives have successfully prevented slumping of mixes in trucks, which sometimes occurs when the hot-mix gradation is such that the mix traps escaping steam. In addition, silicones have provided better finishing qualities to pavement mixtures. These qualities include improved workability, reduced tearing during placement, and a reduction in the amount of effort required for compaction. Testing by several agencies has revealed no detrimental effects on the properties of asphalts when silicone is used in the recommended concentrations. Silicones are very persistent materials and their effects may carry over from one tank of asphalt to another. Proper mixing and control is best achieved by addition of the silicone at the refinery.

4. DENSE-GRADED HOT-MIX ASPHALT.

4.1 GENERAL. Dense graded hot-mix asphalt concrete consists of a mixture of well graded aggregate and asphalt cement. There are several other possible non-asphalt binders including tar and rubberized tar. However, asphalt cement is the binder used in a wide majority of paving mixtures. The hot-mix asphalt is produced at a central plant, laid to the desired grade with an asphalt spreader, and compacted. Hot-mix asphalt provides a high-strength, water resistant, smooth riding surface.

4.2 LABORATORY TESTING FOR MIX DESIGN.

4.2.1 CONTRACTOR PROVIDED JOB-MIX-FORMULA. Current practice is for the contractor to do the mixture design and develop the job-mix-formula (JMF) for the aggregates and asphalt used in the paving project. The contractor should supply a sufficient amount of aggregate and asphalt to the contract officer or his representative for possible verification tests. If verification tests are not performed, these material samples should be kept until the project is completed and accepted. The JMF supplied by the contractor should contain, as a minimum, the following information:

- Percent passing each sieve size of individual aggregate and combined gradations.
- Percent of optimal asphalt cement
- Percent of each aggregate and mineral filler to be used.
- Asphalt viscosity grade, penetration grade, or performance grade.
- Number of blows of hammer per side of molded specimen.
- Laboratory mixing temperature.
- Lab compaction temperature.
- Temperature-viscosity relationship of the asphalt cement.
- Plot of the combined gradation on the 0.45 power gradation chart, stating the nominal
- maximum size.

- Graphical plots of stability, flow, air voids, voids in the mineral aggregate, and unit
- weight versus asphalt content. (example MS-2).
- Specific gravity and absorption of each aggregate.
- Percent natural sand.
- Percent fractured faces (in coarse aggregate).
- Fine aggregate angularity.
- Percent flat or elongated particles (in coarse aggregate).
- Tensile strength ratio (TSR).
- Antistrip agent (if required) and amount.
- List of all modifiers and amount.

The JMF may be adjusted when field conditions warrant a change. The JMF should only be adjusted when changes in materials or procedures occur. The JMF should only be adjusted with the approval of the Owner.

4.2.2 GENERAL PROCEDURE. Laboratory tests are conducted on laboratory-compacted samples with densities equal to densities anticipated in the in-place hot-mix asphalt after being subjected to traffic. A final selection of aggregate blend and asphalt content will be based on these data with due consideration to relative costs of the various mixes. The procedures set forth in the following paragraph are directly applicable to all mixes containing not more than 10 percent by weight of total aggregate retained on the 25 millimeter (1-inch) sieve.

4.2.3 PREPARATION OF TEST SPECIMENS. The selection of materials for use in designing the paving mix has been discussed earlier. As an example, suppose that an aggregate gradation for a hot-mix design shall be the median of the 19 millimeter (3/4 inch) maximum (high pressure) aggregate gradation given in table 2-1. Design data are required on this blend. The initial mix design tests will usually be conducted in a central testing laboratory on samples of stockpile materials submitted by the contractor. The procedure for proportioning stockpile samples to produce a blend of materials to meet a specified gradation is outlined below. The final mix design will be based on samples taken from the asphalt plant and will usually be conducted in a field laboratory near the plant.

4.2.3.1 PROPORTIONING OF STOCKPILE SAMPLES. As a preliminary step in mixture design and manufacture, it is necessary to determine the approximate proportions of the different available stockpiled materials required to produce the desired gradation of aggregate. This step is necessary to determine whether a suitable blend can be produced and, if so, the approximate proportion of each aggregate to be fed from the cold feeder bins into the dryer. Sieve analyses are conducted on material from each of the stockpiles, and the data are shown graphically in figure 2-5. Another method of plotting or graphically illustrating the data is through the use of the 0.45 power curve. This was developed in the early 1960's by the Federal Highway Administration using formula developed in a study by Fuller and Thompson. The equation developed by Fuller for maximum density was:

$$P = 100\,(d/D)^n$$

where d is the diameter of the sieve size in question, P is the total percent passing or finer than the sieve, D is the maximum size of the aggregate, and n is equal to 0.45 for maximum density. The FHWA recommends this chart be used as part of the hot-mix design process. The four aggregate fractions must be combined to produce the desired blend. The estimated percentage of each fraction needed to produce this blend is determined by trial-and-error calculations. Two or three trials are normally required
to obtain the desired blended gradation.

4.2.3.2 PROPORTIONING OF BIN SAMPLES FROM BATCH PLANTS. Once it is demonstrated that a suitable blend can be prepared from the available materials, samples of these materials can then be processed through the asphalt plant for verification of mix design. Sieve analyses must be conducted for each batch of processed aggregate. The data are shown graphically in figure 2-6. The hot-bin aggregates should be blended to produce the same gradation as that produced at the cold feeders. The percentage of each bin is estimated and calculations are made to determine the gradation produced from these estimated percentages. The gradation of this recombined blend is then checked against the desired gradation. Two or three trials are usually sufficient to produce a combined mixture having a gradation within the allowable tolerances.

4.2.4 ASPHALT CONTENTS FOR SPECIMENS. The quantity of asphalt required for a particular aggregate is very important to assure satisfactory performance. The procedures to follow are described in 4.2.5 selection of design compaction method below. An estimate for the optimum amount of asphalt based on total weight of mix is normally made in order to start the laboratory tests. Laboratory tests usually are conducted for a minimum of five asphalt contents: two above, two below, and one at the estimated optimum content. Incremental changes of 1 percent of asphalt may be used for preliminary work, but increments of 0.5 percent are generally used when the optimum asphalt content can be estimated and for final design.

4.2.5 SELECTION OF DESIGN COMPACTION METHOD. Some Owners allow two methods of compacting asphalt paving mixtures in the laboratory—the Marshall and the Gyratory Testing Machine methods. The procedures for conducting the Marshall mix design tests are described in CRD-C 649. The procedure for gyratory compaction is given in CRD-C 651 or the standard testing method of ASTM D 3387 except as follows:

- Use 101.6 millimeter (4-inch) diameter molds in lieu of 152.4 millimeter (6-inch) molds when Marshall stabilities and flows are to be determined.
- Use mixing and compaction temperature requirement as given in CRD-C 649.
- The Gyratory Testing Machine setting and equivalent compaction requirements shall be as listed in table 2-5.

Type of Traffic	Gyratory Compaction Requirement
Tire pressures less than 690 kPa (100 psi)	Gyratory compaction at 690 kPa (100 psi), 1-degree, 30 revolutions (50 blows Marshall equivalent)
Tire pressures 690 kPa (100 psi) and greater but less than 1,586 kPa (230 psi)	Gyratory compaction at 1,380 kPa (200 psi,) 1-degree, 30 revolutions (75 blows Marshall equivalent)
Tire pressures 1,586 kPa (230 psi) and greater	Gyratory compaction at 1,655 kPa (240 psi), 1-degree, 60 revolutions

Table 2-5

Traffic Type and Gyratory Compaction Requirements

In the event an airfield pavement is to be subjected to aircraft with tire pressures of 1,586 kPa (230 psi) or more, the Gyratory Testing Machine (GTM) method may be for some Owners' airfield pavements and is preferred but is not mandatory for other Owners' airfield pavements. If the GTM method is used for design, it should also be used for control testing. If the GTM cannot be used for control testing, the Marshall apparatus can be used by developing a correlation between the GTM and Marshall specimens for the job mix. Care should be taken to insure that excess breakage of the aggregate particles does not occur during Marshall compaction.

4.2.6 TABULATION OF DATA. After the laboratory design method has been selected and test specimens have been prepared, data should be tabulated on forms similar to those shown in CRD-C 649 and CRD-C 650 if the Marshall procedure is used. These forms, along with the forms shown in CRD-C 651, are normally used for the gyratory procedure. Arranging data as shown in table 2-6 will facilitate tabulation of specimen test property data and is preferable to similar but less complete methods except that the peak of the unit-weight curve is normally at a slightly higher asphalt content than the peak of the stability curve used in CRD-C 649 and CRD-C650.

Specimen No.	Asphalt Cement %	Weight - Grams: In Air	Weight - Grams: In Water	Volume cc	Specific Gravity: Actual	Specific Gravity: Theor.	AC by Volume - %	Voids - Per Cent: Total Mix	Voids - Per Cent: Filled	Unit Weight Total Mix Kg/cu m (lb/cu ft)	Stability - N (lb): Measured	Stability - N (lb): Converted	Flow of Units of 0.25 mm or 0.01 in.
A	B	C	D	E	F	G	H	I	J	K	L	M	N
				(C-D)	$\left(\frac{C}{E}\right)$		$\frac{(B \times F)}{(\text{Sp. Gr. of AC})}$	$\left(100 - 100\frac{F}{G}\right)$	$\frac{H}{H+I}$	F×1000 (F×62.4)		*	
A-3.5 1	3.5	1228.3	716.3	512.0	2.399						8985 (2020)	8985 (2020)	11
2		1219.5	712.2	507.3	2.404						8283 (1862)	8612 (1936)	10
3		1205.5	705.3	500.2	2.410						8100 (1821)	8425 (1894)	8
4		1206.2	708.4	497.8	2.423						8416 (1892)	8754 (1968)	8
Avg					2.409					2,404 (150.3)		8696 (1955)	9
Curve					2.409	2.579	8.3	6.6	55.7	2,404 (150.3)			
A-4.0 1	4.0	1276.9	747.3	529.6	2.411						9386 (2110)	9012 (2026)	10
2		1252.6	733.3	519.3	2.412						9008 (2025)	9008 (2025)	9
3		1243.5	730.7	512.8	2.425						8874 (1995)	8874 (1995)	9
4		1230.4	722.8	507.6	2.424						8985 (2020)	9348 (2101)	9
Avg					2.418					2,418 (150.9)		9061 (2037)	9
Curve					2.421	2.559	9.5	5.4	63.8	2,421 (151.1)			
A-4.5 1	4.5	1254.4	738.2	516.2	2.430						9119 (2050)	9119 (2050)	12
2		1238.3	726.8	511.5	2.421						9319 (2095)	9319 (2095)	9
3		1239.0	724.9	514.1	2.410						9386 (2110)	9386 (2110)	10
4		1273.5	752.0	521.5	2.442						9097 (2045)	9097 (2045)	10
Avg					2.426					2,426 (151.4)		9230 (2075)	10
Curve					2.426	2.539	10.7	4.5	70.4	2,426 (151.4)			

Sp. Gr. of Bit. = 1.020

(Continued)

Table 2-6

Computation of properties of asphalt mixtures

Specimen No.		Asphalt Cement %	Weight - Grams In Air	Weight - Grams In Water	Volume cc	Specific Gravity Actual	Specific Gravity Theor.	AC by Volume - %	Voids - Per Cent Total Mix	Voids - Per Cent Filled	Unit Weight Total Mix Kg/cu m (lb/cu ft)	Stability - N (lb) Measured	Stability - N (lb) Converted	Flow of Units of 0.25 mm or 0.01 in.
A		B	C	D	E	F	G	H	I	J	K	L	M	N
					(C-D)	$\left(\frac{C}{E}\right)$		$\frac{(B \times F)}{(\text{Sp. Gr. of AC})}$	$\left(100 - 100\frac{F}{G}\right)$	$\frac{H}{H+I}$	F×1000 (F×62.4)		*	
A-5.0	1	5.0	1237.9	727.0	510.9	2.423						8340 (1875)	8340 (1875)	14
	2		1300.0	763.7	536.3	2.424						9475 (2130)	8812 (1981)	10
	3		1273.6	746.9	526.7	2.418						8452 (1900)	8114 (1824)	12
	4		1247.9	731.8	516.1	2.418						8251 (1855)	8251 (1855)	12
	Avg					2.421					2,421 (151.1)		8380 (1884)	12
	Curve					2.421	2.519	11.9	3.9	76.3	2,421 (151.1)			
A-5.5	1	5.5	1237.3	724.1	513.2	2.411						6450 (1450)	6450 (1450)	12
	2		1264.0	740.6	523.4	2.415						6806 (1530)	6534 (1469)	14
	3		1286.4	752.4	534.0	2.409						7184 (1615)	6895 (1550)	13
	4		1253.5	733.8	519.7	2.412						6695 (1505)	6695 (1505)	16
	Avg					2.412					2,411 (150.5)		6646 (1494)	14
	Curve					2.409	2.500	13.0	3.6	78.3	2,404 (150.3)			

Table 2-6 (continued)

Computation of properties of asphalt mixtures

Plots of data from table 2-6 for stability, flow, unit weight, percent voids total mix, and percent voids filled with asphalt should be made, as shown in table 2-6. The average actual specific gravity is obtained for each set of test specimens, as shown in column G of table 2-6. Each average value is the core density in grams per cubic meter (g/m3) at 10 ^{0}C, 50 ^{0}F. At this temperature the average values are multiplied by 62.4 to obtain density in pounds per cubic foot (pcf). These data are entered in column K. The density values thus obtained are plotted as shown in figure 2-7, and the best-fit smooth curve is then drawn. The data from columns I and J are used to plot curves for percent voids total mix and voids filled with asphalt, respectively, in figure 2-7. The corrected stability values in column M and the flow values in column N of table 2-6 are plotted on figure 2-7 to evaluate stability and flow properties of the mixture.

4.2.7 RELATIONSHIP OF TEST PROPERTIES TO ASPHALT CEMENT CONTENT. Test property curves, plotted as described above, have been found to follow a reasonably consistent pattern for mixes made with penetration and viscosity grades of asphalt cement. Trends generally noted are outlined as follows:

4.2.7.1 FLOW. The flow value increases with increasing asphalt content at a progressive rate except at asphalt contents significantly below optimum.

4.2.7.2 STABILITY. the Marshall stability increases with increasing asphalt content up to a point, after which it decreases.

4.2.7.3 UNIT WEIGHT. The curve for unit weight of total mix is similar to the curve for stability,

4.2.7.4 VOIDS TOTAL MIX. Voids total mix decreases with increasing asphalt content. The void content of the compacted mix approaches a minimum void content as the asphalt content of the mix is increased.

4.2.7.5 VOIDS FILLED WITH ASPHALT. Percent voids filled with asphalt increases with increasing asphalt content and approaches a maximum value in much the same manner as the voids total mix discussed above approaches a minimum value.

4.2.8 REQUIREMENT FOR ADDITIONAL TEST SPECIMENS. The curves in figure 2-7 are typical of those normally obtained when penetration or viscosity grades of asphalt cement are used with aggregate mixes. Aggregate blends may be encountered that will furnish erratic data such that plotting of the typical curves is difficult. In most of these cases, an increase in the number of specimens tested at each asphalt content will normally result in data that will plot as typical curves.

4.3 OPTIMUM ASPHALT AND DESIGN TEST PROPERTIES.

4.3.1 SELECTION OF ASPHALT CONTENT. Previous testing has indicated that the optimum asphalt content is one of the most important factors in the proper design of an asphalt paving mixture. Extensive research and pavement behavior studies have resulted in establishment of certain criteria for determining the proper or optimum asphalt content for a given blend of aggregates. Criteria have also been established to determine whether the aggregate will furnish a satisfactory paving mix at the selected optimum asphalt content.

4.3.2 DETERMINATION OF OPTIMUM ASPHALT CONTENT AND ACCEPTABILITY OF MIX BY MARSHALL METHOD. Data plotted in graphical form in figure 2-7 are used to determine optimum asphalt content. In addition, optimum asphalt content and acceptability of the mix are determined based on table 2-7. Separate criteria are shown for use where specimens were prepared with 50- and 75-blow compactive efforts. As shown in table 2-8, the optimum asphalt content (average) for the example provided is computed as 4.6 percent. Table 2-9 shows the criteria for acceptability of the mix for a 75-blow compactive effort at the optimum bitumen content of 4.6 percent.

Section 1. Procedure for Determining Optimum Asphalt Content

Property	Type of Mix	Point of Curve 50 Blows	Point of Curve 75 Blows
Marshall stability	Hot-Mix Asphalt surface course	Peak of curve	Peak of curve
	Hot-Mix Asphalt intermediate course	Peak of curve	Peak of curve
	Sand asphalt	Peak of curve	______[a]
Unit weight	Hot-Mix Asphalt surface course	Peak of curve	Peak of curve
	Hot-Mix Asphalt intermediate course	Not used	Not used
	Sand asphalt	Peak of curve	______[a]
Flow	–	Not used	Not used
Percent[b] voids total mix	Hot-Mix Asphalt surface course	4	4
	Hot-Mix Asphalt intermediate course	4	4
	Sand asphalt	6	______[a]
Percent[b] voids filled with asphalt	Hot-Mix Asphalt surface course	80[c]	75[c]
	Hot-Mix Asphalt intermediate course	80[c]	75[c]
	Sand asphalt	70	______[a]

(Continued)

[a] Sand asphalt will not be used in designing pavements for traffic with tire pressures in excess of 690 kPa (100 psi).
[b] The theoretical maximum specific gravity can be determined either with the apparent specific gravity as determined in ASTM C-127 and C-128 or by the use of ASTM D 2041. ASTM D 2041 will be used for absorptive aggregate.
[c] If inclusion of asphalt contents at these points in the average causes the voids total mix to fall outside the limits, then the optimum asphalt content should be adjusted so that the voids total mix is within the limits.

Table 2-7

Design criteria

Section 2. Procedure for Determining Acceptability of Mix

Test Property	Criteria: Type of Mix	50 Blows	75 Blows
Marshall stability	Hot-Mix Asphalt surface course	500 pounds or higher	1,800 pounds or higher
	Hot-Mix Asphalt intermediate course	500 pounds or higher	1,800 pounds or higher
	Sand asphalt	500 pounds or higher	______ᵃ
Unit weight	–	Not used	Not used
Flow	Hot-Mix Asphalt surface course	20 (0.01 inch) or less	16 (0.01 inch) or less
	Hot-Mix Asphalt intermediate course	20 (0.01 inch) or less	16 (0.01 inch or less
	Sand asphalt	20 (0.01 inch) or less	______ᵃ
Percentᵇ voids total mix	Hot-Mix Asphalt surface course	3-5 percent	3-5 percent
	Hot-Mix Asphalt intermediate course	3-5 percent	3-5 percent
	Sand asphalt	5-7 percent	______ᵃ
Percentᵇ voids filled with asphalt	Hot-Mix Asphalt surface course	75-85 percent	70-80 percent
	Hot-Mix Asphalt intermediate course	75-85 percent	70-80 percent
	Sand asphalt	65-75 percent	______ᵃ

[a] Sand asphalt will not be used in designing pavements for traffic with tire pressures in excess of 690 kPa (100 psi).
[b] The theoretical maximum specific gravity can be determined either with the apparent specific gravity as determined in ASTM C-127 and C-128 or by the use of ASTM D 2041. ASTM D 2041 will be used for absorptive aggregate.

Table 2-7 (continued)

Design criteria

4.3.3 DETERMINATION OF OPTIMUM BITUMEN CONTENT WHEN USING THE GYRATORY TESTING MACHINE METHOD.

4.3.3.1 THE CRITERIA FOR selecting the optimum bitumen content when using the GTM method of compaction are the same as used for the Marshall method and are as follows:

4.3.3.1.1 GYRATORY COMPACTION at 690 kPa (100 psi), 1-degree, 30 revolutions shall use the mix design criteria contained in Sections 1 and 2 of table 2-7 for the 50-blow mix. Additionally, the mix shall have a gyratory stability index (GSI) equal to or less than 1.

Criteria	Percent
Peak of stability curve	4.3
Peak of unit-weight curve	4.5
4 percent voids in total mix (hot-mix asphalt)	4.8
75 percent total voids filled with asphalt (hot-mix asphalt)	4.9
Average	4.6

[1] Based on data in figure 2-7.

Table 2-8

Computation of optimum asphalt content

Test Property	4.6 Percent Asphalt	Criteria for Acceptability
Flow 0.025 centimeters (0.01 inch)	11	Less than 16
Stability, kN (psi)	9.1 (2,050)	More than 8.0 (1,800)
Percent voids in total mix	4.3	3-5 percent (hot-mix asphalt)
Percent total voids filled with asphalt	72	70-80 percent (hot-mix asphalt)

Table 2-9

Evaluation of acceptability of design mix

4.3.3.1.2 GYRATORY COMPACTION at 1,380 kPa (200 psi), 1-degree, 30 revolutions shall use the mix design criteria contained in Sections 1 and 2 of table 2-7 for the 75-blow mix. Additionally, the mix shall have a gyratory stability index (GSI) equal to or less than 1.

4.3.3.1.3 GYRATORY COMPACTION at 1,655 kPa (240 psi), 1-degree, 60 revolutions shall use the mix design criteria combined in Sections 1 and 2 of table 2-7 or 2-8 for the 75-blow mix. Additionally, the mix shall have a GSI equal to or less than 1.

4.3.3.2 IF THE OPTIMUM BITUMEN content selected by using the design parameters in Sections 1 and 2 of table 2-7 or table 2-8 does not produce a GSI equal to or less than 1, the asphalt content shall be reduced slightly to meet the GSI requirement.

4.3.3.3 LABORATORY DENSITIES, for field control, shall be determined in the field by GTM compaction or by Marshall compaction correlated to GTM compaction. The correlation will be made as part of the mix design effort and will result in establishing the required number of Marshall blows to achieve the same density as the GTM compactor provided there is no significant additional aggregate breakage with the hand hammer than with the GTM. The correlation may also be established by determining the difference in unit weight between the GTM compaction curve and the 75-blow Marshall compaction curve at the desired asphalt content. This difference can be added to the 75-blow field density for comparison with the GTM density. Laboratory densities for field control can then be determined using the Marshall hammer.

4.3.3.4 IF THE 1,655 kPa (240 psi), 1-degree, 60 revolutions compaction effort is used for the mix design, greater field compaction effort will be required by the contractor to achieve the specified density. Also, the compaction effort to determine the field control density is likely to be greater than the standard 75-blow compaction effort.

4.3.3.5 WHEN TWO OR more paving mixes have been investigated, the one used for field construction should be the most economical mix that satisfies all of the established criteria.

4.3.4 THE TENSILE STRENGTH ratio (TSR) of the mixture at the selected optimum will be performed according to ASTM D 4867. A TSR value of less than 75 percent will require the use of an antistrip additive in the mixture.

4.4 MIXTURE CONTROL.

4.4.1 THE AGGREGATES and asphalt must be fed through the plant at a constant rate to obtain efficient plant operation and to produce a mixture conforming to requirements. The approximate proportion of aggregates and asphalt to be fed into the plant is determined from the laboratory mix design. However, some adjustment in these proportions is usually required because gradations of the stockpile aggregates generally will not entirely duplicate the gradation of the aggregate samples obtained for laboratory design use; fines may be lost or manufactured while passing through the dryer; aggregate may degrade in the dryer; and material mixed at an asphalt plant is more uniformly coated with asphalt than materials mixed in the laboratory.

4.4.2 TO EVALUATE the quality of the material produced and to insure the best possible paving mixture, a reasonably complete plant laboratory is necessary. The laboratory should be located at the plant site and should contain about the same equipment listed in CRD C 649 and CRD C 650. Because of the capacity of most asphalt plants, at least two technicians should be assigned to conduct control tests; otherwise, all necessary testing cannot be completed in a timely manner.

4.4.3 THE HEAVIEST DEMANDS on plant laboratory facilities occur at the initiation of plant production. For batch plants, preliminary computations may be made to determine the weight of material from each bin that will provide the gradation on which the mixture design is based. However, the gradation of the aggregate supplied by the plant may not precisely reproduce the desired gradation. The gradation of the plant-produced aggregate generally approximates the gradation used in design, within reasonable tolerances, if initial sampling for design purposes has been accomplished properly and if the plant is operated efficiently. Certain steps should be taken, however, to insure that satisfactory mixtures are produced from the beginning and throughout the period of plant production. Procedures subsequently outlined will insure that satisfactory paving mixes are produced.

4.4.4 THE AGGREGATES obtained from the hot bins of batch plants sometimes cannot be proportioned to satisfactorily reproduce the gradation of the aggregate used in the laboratory design. It is then necessary to redesign the mix using plant-produced aggregates. Specimens are prepared and tested for the new design in the same manner as for the original design tests. Optimum asphalt content and acceptability of the mix produced by the plant are determined. Occasions may arise where the gradation of the plant-produced aggregate will differ from that on which the laboratory design was based to the extent that specified criteria cannot be met. Necessary steps should be taken to produce a asphalt mixture meeting the specification requirements.

Sufficient additional tests should be performed to establish optimum asphalt requirements and to insure that the mix will meet applicable criteria.

4.4.5 AFTER THE AGGREGATE and asphalt binder qualities have been determined to be satisfactory and a proper mix design has been completed, the next step is to insure that the JMF is produced at the asphalt plant. Several items must be routinely controlled during the production and laydown operation to provide an acceptable pavement. The mixture items include aggregate gradation, asphalt content, voids, and voids filled. To a great extent these items are interdependent on each other and they should be analyzed as a group. The laydown items include density, smoothness, and final grade. Some Owners require that five of these items be measured and analyzed statistically. These items are air voids, asphalt content, density, smoothness, and final grade. When these items do not meet the specified requirements, the contract unit price is reduced or the mixture is rejected. Small projects of less than 1,000 metric tons of hot-mix can be constructed without the pay reduction clause for economic reasons.

4.4.6 IN ORDER TO EVALUATE the quality of a job, the work is divided into lots. Each lot is considered as a separate job and as such is evaluated solely on the test results for that lot. A lot should generally not exceed 2,000 metric tons (2,000 tons) of production or one normal day's production. The lot should be subdivided into four equal sublots, and a random sample should be taken from each sublot for evaluation of air voids, asphalt content, and density. The random sublot sample for these properties will include one sample of uncompacted asphalt mixture, one field core from a pavement joint area and one field core from the compacted hot-mix asphalt at least 0.3 meter (1-foot) away from the joint.

4.4.7 THE ASPHALT CONTENT and aggregate gradation will be determined from samples of the asphalt mix taken somewhere between the production and the laydown operation. The exact location of the sample is not important, but the sample should be taken from the same location each time (for example, truck at asphalt plant, truck, at laydown site, bituminous storage bin, or other locations). The same sample of asphalt mixture should be used for determining asphalt content and aggregate gradation.

4.4.8 IF A LOT SIZE equal to 1,000 metric tons (1,000 tons) is selected, a sample of asphalt mix will have to be taken for each 250 metric tons (250 tons) produced. Any approved method for locating random samples can be used. As an example, suppose that a random number is selected between 1 and 250 and is determined to be 200. This selection means that the 200th ton batched will be sampled.

4.4.9 AFTER THE FOUR aggregate gradations and asphalt contents are determined for a lot, these results are compared with the JMF and the absolute difference is determined. Suppose the design asphalt content is 5.5 percent and the four extracted asphalt contents are determined to be 5.2, 5.4, 5.5, and 5.8. The mean absolute deviation from the JMF is determined to be:

$$\textit{Mean absolute deviation} = \frac{0.3 + 0.1 + 0.0 + 0.3}{4} = 0.175$$

The same procedure is used to determine the mean absolute deviation for each sieve size for the aggregate gradation. After the mean absolute deviation is determined for the asphalt content and aggregate gradation of a lot, the maximum percent payment for that lot can be determined from the tables provided in the specification requirements.

4.4.10 DENSITY MUST BE DETERMINED within the mat and at the joints between mats. One sample should be obtained in the mat and one in the joint for each sublot. The total linear length of joint constructed for a given lot will be divided into quarters and one random sample taken for each sublot. These sample locations can be determined in a similar way as that for aggregate gradation and asphalt content. All mat samples should be taken at least 0.3 meter (1 foot) from the edge of mat or joint. In order to determine sample locations in the mat, each sublot must be divided into grids. The number of possible sampling locations will be approximately equal to the length in meters times 2 minus 1 times the width in meters time 2 minus (length in feet - 1 foot) 1 (width in feet - 1 foot).

4.4.11 AS AN EXAMPLE, suppose that 1,000 metric tons (one lot) of hot-mix asphalt were placed in two adjacent lanes, one lane 2,000 meters long and the other 1,000 meters long. The joint length between the two lanes would be 1,000 meters; thus, one sample would be taken at random for each 250 meters of joint length to evaluate joint density. The total length of the two lanes would be 3,000 meters; therefore, one random sample should be taken from the mat for each 750 meters of hot mix asphalt. The first 750 meters would have [750 x 2 - 1] x [3 x 2 - 1] possible sampling locations if a 3-meter-wide paver was used. (Possible sample locations are at 0.5 meter or 2 every meter intervals longitudinally and transversely and no closer than 0.5 meter from the edge.) Hence, there are 7,495 (1,499 x 5) possible sampling locations for each of the 4 cores to be taken from the mat. Suppose that the random number selected was 3,108. Divide 3,108 by 5 to get 621 with a remainder of 3 [0.6 x 5]. Hence, the sample should be taken 311 meters (621 ÷ 2 + 0.5, possible horizontal sample locations + 0.5 meter) from the origin and 2.0 meters (3 ÷ 2 + 0.5, possible transverse sample locations + 0.5 meter) from the edge (since the start point is 0.5 meter from the edge and 0.5 meter from the beginning). The random samples do not have to be precisely located, but it is important that the surface appearance does not affect the selection of sample locations.

4.4.12 THE AVERAGE MAT DENSITY and average joint density will each be expressed as a percentage of the laboratory density. The laboratory density for each lot will be the average density determined from at least two sets of samples representing the in-place material compacted in the laboratory. Suppose that the average laboratory density is 2,404 kilograms per cubic meter (150 pounds per cubic foot), the four mat samples have individual densities of 2.324 grams/centimeter^3, 2.356 grams/centimeter^3, 2.348 grams/centimeter^3, and 2.373 grams/centimeter^3, and the

four joint samples have individual densities of 2,311 grams/centimeter^3, 2.324 grams/centimeter^3,
2.343 grams/centimeter^3, and 2.325 grams/centimeter^3. Based on these results the average mat density would be:

$$Mat\ density = \frac{2.324 + 2.356 + 2.348 + 2.373}{4(2.404)} = 97.8\ percent$$

and the average joint density would be

$$Joint\ density = \frac{2.311 + 2.324 + 2.343 + 2.325}{4(2.404)} = 96.8\ percent$$

The average density in the mat and the average density in the joint can be used along with the tables in the specifications to determine the maximum percent payment for the lot of material being evaluated.

4.4.13 THE SURFACE of the completed pavement will be evaluated on a systematic basis to determine the acceptability of grade and smoothness. The results will be compared with the specification requirements to determine percent payment for grade and smoothness.

4.4.14 IN ORDER TO PROPERLY EVALUATE quality control of a mixture and maintain up-to-date records of test results, control charts should be maintained. It is recommended that the control charts be plotted for each sieve size specified in the gradation requirements, asphalt content, laboratory density, stability, flow, voids in total mixture, voids filled with asphalt, mat density, and joint density. A plot should be made of individual values and for the running average of four samples.

4.4.15 AN EXAMPLE OF THE USE of control charts follows. Assume the density results shown in table 2-10 were obtained from the in-place mat.

4.4.16 FIGURE 2-8 SHOWS the control charts for mat density. The first test result obtained is plotted in figure 2-8a. Note that this measurement falls below the desired range. At this point, it should have been concluded that the process was out of control; thus, the operation should be stopped until the cause of the deficiency is identified and corrected.

Lot 1	Lot 2	Lot 3	Lot 4	Lot 5
95.2	98.4	99.5	98.1	100.0
96.4	99.1	98.0	96.8	99.1
97.8	97.3	97.5	99.8	98.3
96.6	98.1	98.3	98.6	98.9
Average				
96.5	98.2	98.3	98.3	99.1

Table 2-10

Lot density as a percent of laboratory density

4.4.17 THE SECOND, THIRD, AND FOURTH samples were obtained after corrections were made to the process and found to be higher but still below the desired range. At this point, the weight of the rubbertired roller used in compacting the mat was increased from 20 to 25 metric tons (20 to 25 tons), and the tire pressure was increased from 480 kPa (70 psi) to 620 kPa (90 psi). After these changes, the density results were generally within the desired range.

4.4.18 THE MOVING AVERAGE is determined for the last four samples tested (figure 2-8b). Plotting the moving average smooths out the plot of individual values and allows trends to be spotted earlier.

4.5. SIGNIFICANCE OF CHANGES IN MIXTURE PROPERTIES.

4.5.1 GENERAL. As a general rule, the flow and stability values are obtainable quickly and are reasonably reliable indicators of the consistency of the plant-produced mix. A measurable increase in flow value generally indicates that either the gradation of the mix has changed sufficiently to require a revision in the optimum asphalt content for the mix, or too much asphalt is being incorporated in the mix. A review of the control charts should indicate the problem. Substantial changes in stability or void content also may serve as an indication of these factors. Mix proportions shall be adjusted whenever any test property consistently falls outside of the specified tolerances. In the case of batch plants, the use of faulty scales and the failure of the operator to accurately weigh the required proportions of materials are common causes for paving-mixture deficiencies. Improper weighing or faulty scales may be detected readily and corrective measures taken by maintaining a close check of load weights. Figure 2-9 lists other probable causes of paving-mixture deficiencies.

4.5.2 EXTRACTION TESTS.

4.5.2.1 REPRESENTATIVE SAMPLES of paving mixture should be obtained for extraction tests to determine the percentage of bitumen in the mix and the gradation of the extracted aggregates. Extraction tests shall be made according to ASTM D 2172. Sieve analyses of recovered aggregates shall be determined according to D 5444.

4.5.2.2 NUCLEAR GAGES are currently being used to determine asphalt content in accordance with ASTM D 4125. After the nuclear gage is calibrated, it can be used to check the asphalt content of a mixture in a few minutes. Results indicate that this procedure is more accurate than the conventional extraction test, but the aggregate gradation is not determined by this test. Therefore, extraction tests must also be conducted to determine the aggregate gradation.

4.5.2.3 ASPHALT CONTENT can be determined with the Ignition Method in accordance with ASTM PS 090. The asphalt content obtained may be more accurate than that obtained by the conventional extraction method. The aggregates remaining after the asphalt binder is burned off may be used for gradation purposes; however, there is a correction factor that must be determined for each type of aggregate and gradation used.

4.5.3 HOT-BIN GRADATIONS. Hot-bin gradation tests should be made on the aggregate in the fine bin at least twice daily during operation. Hot-bin gradations shall be determined on all bins in conjunction with sampling of the pavement mixture. Washed sieve analyses shall be determined initially.

4.5.4 CONSTRUCTION CONTROL. Well-designed mixes can be compacted by adequate field rolling to about 98 percent or greater of the density obtained by compacting specimens with previously specified laboratory procedures. Asphalt intermediate, base course, or surface course mixes shall be rolled to the density specified in applicable Department of the Army and Air Force guide specifications.

4.5.5 PAVEMENT SAMPLING. Samples for determining pavement density and thickness may be taken either with a coring machine (at least 100 millimeters, (4 inches) nominal diameter) or by cutting out a section of pavement at least 100 millimeters (4 inches) square with a concrete saw. These samples should include the entire thickness of the pavement. Density samples of each day's production should be taken and delivered to the project laboratory by noon of the following day, and the density determinations made by the end of the day. Any changes in placing technique necessary to obtain the required density can be made before a large amount of pavement is placed.

4.5.6 TESTING PAVEMENT SAMPLES.

4.5.6.1 PAVEMENT SAMPLES shall be prepared for testing by carefully removing all particles of base material or other foreign matter. All broken or damaged edges of sawed samples for density tests shall be carefully trimmed from the sample. Thickness measurements shall be made before separating the sample into layers. A sample consisting of an intermediate course and surface course shall be split at the interface of these layers before testing. The density of the sawed samples shall be determined by weighing in air and in water as previously described. Samples from which density measurements are desired shall be discarded if damage is apparent. Additional samples will be taken from the same sublot.

4.5.6.2 NUCLEAR GAGES are currently being used to check density of hot-mix asphalt. This method is fast, but the results are often questionable. Some factors which affect the results of density measurements with the nuclear gage are thickness of asphalt mixture, density of material below asphalt mixture, and smoothness at test location. The nuclear gage is useful for developing roller patterns, but density tests for acceptance should be conducted by removing samples from the pavement andweighing in air and water.

4.5.7 DENSITY DATA. Density data obtained from specimens in the manner previously described will be compared with the average laboratory density determined for the same lot.

4.5.8 PAVEMENT IMPERFECTIONS AND PROBABLE CAUSES. Many types of pavement imperfections result from improper laying and rolling operations as well as from improper mixes or faulty plant operation. These imperfections can be controlled only by proper inspection. Figure 2-10 presents the pavement imperfections that may result from laying unsatisfactory mixes or using faulty construction procedures.

5. POROUS FRICTION COURSE.

5.1 GENERAL. A porous friction course (PFC) is an open-graded, free-draining asphalt paving mixture that can be placed on an existing pavement to minimize hydroplaning and to improve skid resistance in wet weather. This surface should not be used for low speed applications or in areas subjected to tank traffic (especially tank turning areas). The thickness of the finished course can vary from approximately 19 millimeters (3/4 inch) to 25 millimeters (1 inch). A PFC has a coarse surface texture and is sufficiently porous to permit drainage of water internally as well as along the surface. A combination of water pressure relief through the internal and surface voids and the rough surface texture promote tireto- aggregate contact. PFC paving mixtures are produced in hot-mix asphalt plants and placed with conventional asphalt paving machines. They should be placed on pavements which are in good condition. A leveling course may be required to achieve the desired conditions before construction of the PFC.

5.2 MATERIALS.

5.2.1 AGGREGATES. High quality aggregates are required for PFC's with a maximum LA abrasion loss (ASTM C 131) of 25 percent and 40 percent for high and low tire pressure loadings, respectively. A crushed aggregate is required and shall have a minimum of 90 percent by total weight of aggregate with one crushed face and 70 percent with two crushed faces. Antistrip agents shall be specified when required. The Air Force currently requires an antistripping agent in all PFC and the underlying hot-mix asphalt layer regardless of the results of the immersion compression test. Table 2-11 presents the aggregate gradation requirements for porous friction courses.

Sieve Designation (Square Openings)	Percent Passing by Weight of Total Aggregates Gradation "A" 19 mm (3/4-inch) Maximum (Compacted Nominal Thickness, 25 mm, 1 inch)	Gradation "B" 12.5 mm (1/2-inch) Maximum (Compacted Nominal Thickness, 19 mm, 3/4 inch)
19 mm (3/4 inch)	100	100
12.7 mm (1/2 inch)	70-100	100
9.5 mm (3/8 inch)	35-75	80-100
4.75 mm (No. 4)	25-40	25-40
2.36 mm (No. 8)	10-20	10-20
600 m (No. 30)	3-10	3-10
75 m (No. 200)	0-5	0-5

Table 2-11

Aggregate Gradation for PFC's

5.2.2 ASPHALT CEMENT. Test requirements for asphalt cements are outlined in the appropriate specification (ASTM D 946, D 3381, or AASHTO MP-1). The asphalt type should be selected as indicated in paragraph asphalt cement selection by temperature region of this chapter. Several PFC's with latex rubber added to the asphalt have been constructed. The addition of a latex rubber additive should improve the ability of the asphalt to hold the aggregate in place and reduce oxidation deterioration in the porous mat. When economically available, the use of a latex rubber modified binder should be specified.

5.3 MIXTURE DESIGN.

5.3.1 PROPORTIONING OF AGGREGATES. The proper aggregate gradation should be selected from table 2-11.

5.3.2 ASPHALT CONTENT. The asphalt content of PFC's is expressed as a percentage of the total mix by weight. A surface area constant, K , as described in the centrifuge kerosene equivalent (CKE) test (ASTM D 5148), is used to determine the optimum asphalt content. The K value is used in the c relation 2K + 4.0 to determine the Estimate of Asphalt (EOA). This asphalt content is valid for c aggregates with an apparent specific gravity in the range of 2.60 to 2.80 and with a water absorption less than 2.50 percent when tested by ASTM C 127 for coarse aggregate and ASTM C 128 for fine aggregate. A slight increase in asphalt content (up to 0.5 percent) is required when the absorption is greater than 2.50 percent. The EOA is inversely proportional to the specific gravity of the aggregate used and adjustments must be made when the specific gravity is outside of the 2.60 to 2.80 range.

5.3.2.1 K FACTOR. The K factor indicates the relative particle roughness and surface capacity based on porosity of the aggregate to be used for the PFC. The K factor is determined from the percent of SAE 10 oil retained, which represents the total effect of the coarse aggregate's absorptive properties and surface roughness. The K factor is determined from that portion of the aggregate sample which passes the 9.5 millimeter (3/8-inch) sieve and is retained on the 4.75 millimeter (No. 4) sieve using the procedure as described in ASTM D 5148. If the specific gravity for the aggregate is greater than 2.70 or less than 2.60, apply a correction to oil retained, using the formula given in ASTM D 5148. No correction need be applied for asphalt viscosity.

5.3.3 MIXING TEMPERATURE. The mixing temperature shall be chosen to provide an asphalt viscosity of 275 ± 25 centistokes. To obtain this temperature, the temperature-viscosity relationship must be evaluated for the type of asphalt selected at a minimum of three temperatures (ASTM D 2170 and ASTM D 2171). Plotting this information on a graph with temperature versus log viscosity will normally result in a straight-line relationship, and the temperature for the correct viscosity can be chosen from the graph.

5.4 PLANT CONTROL.

5.4.1 PLANT LABORATORY. A plant laboratory is needed to insure that the aggregate is properly graded and that the mix contains the prescribed percentage of asphalt binder. The laboratory should be located at the plant to minimize the time between production and testing. If the laboratory is not located at the plant, testing could fall behind and cause considerable quantities of unsatisfactory mix to be produced.

5.4.2 SIEVE ANALYSIS. All sieve analyses should be conducted by the method described in ASTM C 136. Recommended sieve sizes for plant sieve analysis are: 19 millimeter (3/4 inch), 12.5 millimeter (1/2 inch), 9.5 millimeter (3/8 inch), 4.75 millimeter (No. 4), 2.36 millimeter (No. 8), 600 m (No. 30), and 75 m (No. 200). For batch-mix plants, sieve analyses shall be made on material from each plant hot bin. Samples for these sieve analyses shall be obtained after a few tons of aggregate have been processed through the dryer and screens in order that the sample will be representative. For drum mixers, the sieve analysis must be made directly from the cold feeds. Final mix proportions may be determined on the basis of these analyses.

5.4.3 EXTRACTION TESTS. Extraction tests shall be made in accordance with ASTM D 2172 using trichloroethylene as the extraction solvent. A nuclear gage can be used to determine the asphalt content, when tested in accordance with ASTM D 4125, provided it is calibrated. The asphalt content can also be determined by the ignition method in accordance with ASTM PS 090. Sieve analysis of recovered aggregates shall follow procedures specified in ASTM D 5444.

5.4.4 MIX PROPORTIONS. Mix proportions shall be adjusted whenever tests indicate that specified tolerances are not being met. In the case of batch plants, faulty scales, and failure of the operator to accurately weight the required proportions of materials are common causes for mixture deficiencies. Improper weighing or faulty scales may be detected readily and corrective measures taken by maintaining a close check of load weights. Figure 2-9 presents other probable causes of mixture deficiencies due to improper plant operations.

5.4.5 CONTROLLING PLANT PRODUCTION. The plant inspector should obtain a sample of the PFC mix after the plant has been in production about 30 minutes. The sample should be tested as rapidly as possible for compliance with gradation and asphalt content requirements.

5.5 CONSTRUCTION.

5.5.1 PAVEMENT CONTROL. A PFC pavement has no density requirements. A characteristic of this overlay is its rapid cooling. If minimum asphalt drainage is desired, the roller should closely follow the paver to initially set the PFC so that asphalt drainage is minimized. If more drainage is desired, the roller should wait longer before rolling the PFC. Rich spots will tend to drain if rolling is delayed. Two passes with a 10-metric ton (10 ton) steel-wheel roller should be satisfactory to properly seat the PFC mix.

5.5.2 PAVEMENT SAMPLING. Samples for determining thickness (ASTM D 979) may be taken either with a coring machine or by cutting out a sample of pavement at least 100 millimeters (4 inches) square with a concrete saw. The sample should include the entire thickness of the PFC.

5.5.3 STORAGE SILOS. Storage of PFC mix should be avoided whenever possible; the maximum allowable storage time under any circumstance should not exceed 15 minutes. Excessive storage time will allow the asphalt to drain, causing segregation of the mixture. Proper coordination between the plant and the laydown operations will eliminate the need for extended storage.

5.3.4 PAVEMENT OPERATIONS UNDER NORMAL CONDITIONS. Some Owners do not permit placement of PFC when the surface temperature of the existing pavement is below 60 ^{0}F. The most important consideration is whether the contractor can apply the necessary rolling before the mixture becomes too cool to be properly seated. Generally, all rolling should be performed before the PFC mixture cools to 80 ^{0}C (175 ^{0}F). A PFC will cool quickly because of the thin layer of material and high void content in the thin PFC layer. Thus, judgment should be used in the application of the temperature limitations in the guide specifications to avoid shutting down operations during periods when satisfactory final pavement properties could be obtained.

6. STONE MATRIX ASPHALT.

6.1 GENERAL. Stone matrix asphalt (SMA) is a mixture of aggregate, mineral filler, asphalt cement, and a stabilizer (cellulose or mineral fiber and/or modified asphalt). SMA is designed to prevent rutting and abrasion even under high loads and/or high tire pressures. SMA mixtures depend on aggregate to aggregate contact to support traffic loads thereby requiring a large percentage of coarse aggregate. Excess fine aggregate or too much mastic can prevent the coarse aggregate particles from obtaining full contact and therefore lower the mixture's resistance to rutting. The high void content of the mix is occupied by fine aggregate, mineral filler, asphalt cement, and a stabilizer (polymer, cellulose, or mineral fiber) which forms a mastic portion of the SMA mixture. This mastic stabilizes the coarse aggregate and reduces the final air voids in the SMA to about 3 to 4 percent. SMA is comparable to hot-mix asphalt in regards to structural design, mix design, and construction. SMA originated in Europe and has recently

been placed by state and federal agencies on projects throughout the country. The design and construction of SMA pavements are described in this appendix.

6.2 MATERIALS.

6.2.1 AGGREGATES. The gradation used for SMA is gap-graded for the coarse aggregate retained on the 4.75 millimeter (No. 4) sieve. This coarse aggregate will make up from 72 to 80 percent of the aggregate in the mix. The coarse aggregate should be 100 percent passing the 19 millimeter (3/4-inch) sieve. The amount of the fine material passing the 75 m (No. 200) sieve will be from 8 to 10 percent. Table 2-12 lists the gradation as recommended by the FHWA. This gradation is based on the recommendations of a technical working group that reviewed the performance of SMA mixtures In place. Table 2-13 lists the recommended coarse and fine aggregate properties for SMA.

Sieve Size	Percent Passing
19 mm (3/4 inch)	100
12.7 mm (1/2 inch)	85-95
9.5 mm (3/8 inch)	75 maximum
4.75 mm (No. 4)	20-28
2.36 mm (No. 8)	16-24
600 m (No. 30)	12-16
300 m (No. 50)	12-15
75 m (No. 200)	8-10

Table 2-12

SMA Gradation Guideline

6.2.2 FILLER. As presented in table D1 for SMA mixtures, the recommended amount of aggregate filler (dust) passing the 75 m (No. 200) sieve is 8 to 10 percent. This amount of filler in the SMA is higher than that usually found in dense graded hot mix asphalt (HMA). The amount of filler is important in terms of obtaining the desired mixture air voids and in affecting the optimum asphalt content. The SMA asphalt content is sensitive to the aggregate fines and filler content. In Europe, SMA mixtures commonly employ a filler-to-asphalt ratio of approximately 1.5. In contrast, conventional dense graded hot mix in the United States typically recommend a filler-to-asphalt ratio of less than 1.2. A well-graded filler with no more than 20 percent of the total filler smaller than 20 microns is required. Commercial fillers are added by mineral filler feeder systems. Fly ash, limestone dust, and other types of rock dust have been used successfully as fillers for SMA applications.

6.2.3 STABILIZER.

6.2.3.1 GENERAL. There is a tendency for the asphalt binder to drain from the aggregate during storage, transportation, or placement because of the high asphalt content in the mix, the thick asphalt coating on the coarse aggregate, and the high voids in the aggregate skeleton. In order to reduce this drainage potential, stabilizers are used to stiffen the mastic or to increase the asphalt binder viscosity. These stabilizers can be categorized into two groups: either (cellulose fibers or mineral fiber) or polymers. A large percentage of SMA has been placed with a combination of fibers and asphalt polymer.

Property	Specifications	Requirement
Coarse Aggregate:		
Los Angles abrasion, %	ASTM C 131, AASHTO T 96	30 max.
Flat and elongated, + No. 4	ASTM D 4791	
3 to 1 (length to thickness), %		20 max.
5 to 1 (length to thickness), %		5 max.
Fractured faces, + No. 4		
One fractured face, %		100 min.
Two fractured faces, %		90 min.
Absorption, %	ASTM C 127, AASHTO T 85	2 max.
Coarse and fine durability index	ASTM D 3744, AASHTO T 210	40 min.
Sulfate soundness loss, 5 cycles	AASHTO T 104, ASTM D 3744	
Sodium sulfate, %		15 max.
or Magnesium sulfate, %		20 max.
Fine Aggregate:		
Crushed manufactured fines, %		100 min.
Sulfate soundness loss, 5 cycles		
Sodium sulfate, %		15 max.
Liquid limit	AASHTO T 89, ASTM D 4318	25 max.

Table 2-13

Recommended SMA Coarse and Fine Aggregate Properties

6.2.3.2 FIBER STABILIZERS. Tables 2-14 and 2-15 provide the requirements and test procedures to be used with cellulose and mineral fibers, respectively. The dosage rates normally used for cellulose fibers is 0.3 percent by weight of the total mix and for mineral fibers is 0.4 percent by weight of the total mix. The recommended tolerance for the fibers is approximately 10 percent of the required fiber weight.

6.2.3.3 ASPHALT-POLYMER STABILIZERS. SMA mixtures have been placed using a polymer to modify the asphalt cement and stabilize the mixture so that fibers are not required. There have also been instances where a polymer and a fiber have been used in conjunction to stabilize SMA mixtures. Manufacturer's design and construction recommendations should be followed as the standard SMA guidelines may not be applicable for asphalt-polymer stabilizers.

6.2.3.4 ASPHALT. The asphalt cement shall comply with the requirements of ASTM D 946, ASTM D 3381, or AASHTO MP-1. The asphalt cement used shall be the grade normally used in the area. The temperature of the asphalt cement at the time of mixing shall be that required to achieve a viscosity of 170 ± 20 centistokes. Where a polymer modified asphalt cement is used, manufacturer's recommendations for mixing temperature shall be followed.

6.3 MIXTURE DESIGN. The optimum asphalt content shall be determined with procedures similar to those outlined in Hot-Mix Asphalt. Table 2-16 contains the recommended mix design requirements for SMA. These requirements are based on work done with the Federal Highway Administration (FHWA) and others and published by the National Asphalt Pavement Association (NAPA).

Properties	Requirement
Sieve Analysis	
Method A	
Alpine Sieve[1] Analysis	
Fiber length	6 mm (0.25 in.) (max.)
Passing 150 m (No. 100) sieve	70% (± 10%)
Method B	
Mesh Screen[2] Analysis	
Fiber length	6 mm (0.25 in.) (max.)
Passing 850 m (No. 20) sieve	85% (± 10%)
425 m (No. 40) sieve	65% (± 10%)
106 m (No. 140) sieve	30% (± 10%)
Ash Content[3]	18% (± 5%) non-volatiles
pH[4]	7.5 (± 1.0)
Oil Absorption[5]	5.0 (± 1.0) (times fiber weight)
Moisture Content[6]	< 5% (by weight)

[1] Method A, Alpine Sieve Analysis. This test is performed using an Alpine air jet sieve (Type 200 LS). A representative 5 gram sample of fiber is sieved for 14 minutes at a controlled vacuum of 75 kPa (11 psi). The portion remaining on the screen is weighed.

[2] Method B, Mesh Screen Analysis. This test is performed using standard 850, 425, 250, 180, 150, 106   m (No. 20, 40, 60, 80, 100, 140) sieves, nylon brushes, and a shaker. A representative 10 gram sample of fiber is sieved using a shaker and two nylon brushes on each screen. The amount retained on each sieve is weighed and the percentage passing calculated. Repeatability of this method is suspect and needs to be verified.

[3] Ash Content. A representative 2-3 gram sample of fiber is placed in a tared crucible and heated between 595° and 650°C (1100° and 1200°F) for not less than 2 hours. The crucible and ash are cooled in a desiccator and reweighed.

[4] pH Test. Five grams of fiber is added to 100 ml of distilled water, stirred, and let sit for 30 minutes. The pH is determined with a probe calibrated with pH 7.0 buffer.

[5] Oil Absorption Test. Five grams of fiber is accurately weighed and suspended in an excess of mineral spirits for not less than 5 minutes to ensure total saturation. It is then placed in a screen mesh strainer (approximately 0.5 square millimeter hole size) and shaken on a wrist action shaker for 10 minutes (approximately 1 1/4 inch motion at 240 shakes/minute). The shaken mass is then transferred without touching, to a tared container and weighed. Results are reported as the amount (number of times its own weight) the fibers are able to absorb.

[6] Moisture Content. Ten grams of fiber is weighed and placed in a 121°C (250°F) forced air oven for 2 hours. The sample is then reweighed immediately upon removal from the oven.

Table 2-14

Properties of Cellulose Fibers

Properties	Requirement
Sieve Analysis	
Fiber length[2]	6 mm (.25 in.) max. mean text value
Thickness[3]	5 m (0.0002 in.) max. mean test value
Shot content[4]	
250 m (No. 60) sieve	95 percent passing (min.)
63 m (No. 230)	65 percent passing (min.)

[1] The European experience and development of the above criteria are based on the use of basalt mineral fibers.
[2] The fiber length is determined according to the Bauer McNett fractionation.
[3] The fiber diameter is determined by measuring at least 200 fibers in a phase contract microscope.
[4] Shot content is a measure of non-fibrous material. The shot content is determined on vibrating sieves. Two sieves, No. 60 and No. 230, are typically utilized; for additional information see ASTM C612.

Table 2-15

Properties of Mineral Fibers[1]

Design Parameter	Value
Marshall[1]	
(1) VTM, percent[2]	3-4
(2) Asphalt content, percent[3]	6.0 min.
(3) VMA[4]	17 min.
(4) Stability, N (lbs)	6200 (1400) suggested minimum
(5) Flow, 0.25 mm (0.01 inch)	8-16
(6) Compaction, number of blows at each side of test specimen	50
(7) Draindown, percent[5]	0.3 max. (1 hour reading)

[1] Marshall procedures are in accordance with AASHTO T 245 (ASTM D 1559).
[2] VTM (voids in total mix or air voids) (see figure 2-12) is based on AASHTO T 166, T 209 (ASTM D 2041), and T 269 (ASTM D 3203). Maximum density will be based on AASHTO T 209 (ASTM D 2041).
[3] Based on weight of total mix.
[4] VMA (see Asphalt Institute Manual Series No. 2 (MS-2).
[5] NCAT SMA asphalt draindown test (see paragraph 2.7,c, (1))

Table 2-16

SMA Mix Design Requirements

6.3.1 SMA ASPHALT DRAINDOWN TEST. For the purpose of this test method, draindown is considered to be that portion of the asphalt cement which separates itself from the sample as a whole and is deposited outside the wire basket during the test. (Note, any noticeable aggregate particles that are deposited outside the basket should be added back into the mixture and not counted as draindown. Alternatively, the test should be rerun.) This test method can be used to determine whether the amount of draindown measured for a given SMA mixture is within acceptable levels. It also provides an evaluation of the draindown potential of an SMA mixture produced in the field.

6.3.1.1 SCOPE. This test method covers the determination of the amount of draindown in an uncompacted SMA mixture sample when the sample is held at elevated temperatures comparable to those encountered during the production, storage, transport, and placement of the mixture.

6.3.1.2 SUMMARY OF METHOD. A sample of the SMA mixture to be tested is prepared in the laboratory or obtained from field production. The sample is placed in a wire basket which is positioned on a pre-weighed paper plate. The sample, basket, and plate are placed in a forced air oven for one hour at a preselected temperature. At the end of one hour, the basket containing the sample is removed from the oven along with the paper plate and the paper plate is weighed to determine the amount of draindown that occurred.

6.3.1.3 EQUIPMENT.

6.3.1.3.1 OVEN, capable of maintaining the temperature in a range from 120^0-175^0 C (250^0 - 350^0 F). The oven should maintain the set temperature to within ± 2^0 C (± 3.6^0 F).

6.3.1.3.2 PAPER PLATES of appropriate size. The paper plates used should be of appropriate durability to withstand the oven temperatures.

6.3.1.3.3 STANDARD CYLINDRICAL SHAPED BASKET meeting the dimensions shown in figure 2-11. The basket shall be constructed using standard 6.3 millimeter (0.25 inch) sieve cloth as specified in ASTM E 11.

6.3.1.3.4 SPATULAS, trowels, mixer, and bowls as needed.

6.3.1.3.5 BALANCE accurate to 0.1 gram.

6.3.1.4 SAMPLE PREPARATION. For each mixture tested, the draindown characteristics should be determined at the anticipated plant production temperature. Duplicate samples should be tested.

63.1.4.1 LABORATORY SAMPLE PREPARATION. Dry the aggregate to constant mass and sieve it into appropriate size fractions as indicated in ASTM D 1559. Determine the anticipated plant production temperature or select a mixing temperature in accordance with ASTM D 1559. The asphalt cement supplier's recommendations should be sought when using modified asphalt cement. Weigh into separate pans for each test sample the amount of each size fraction required to produce completed SMA mixture samples having a mass of 1200 grams. The aggregate fractions should be combined such that the resulting aggregate blend has the same gradations as the job mix formula. Place the aggregate samples in an oven and heat to a temperature not to exceed the mixing temperature established above by more than approximately 28 ^{0}C (50 ^{0}F). Heat the asphalt cement to the established mixing temperature. Place the heated aggregate in the mixing bowl. When a stabilizer is used it should be added as directed by the supplier. Some types of stabilizers such as fibers or some polymers must be added directly to the aggregate prior to mixing with the asphalt cement. Other types must be added directly to the asphalt cement prior to blending with the aggregate. The aggregates and any other components should be thoroughly mixed together. Form a crater in the aggregate blend and add the required amount of asphalt. The amount of asphalt shall be such that the final sample has the same asphalt content as the job-mix-formula. At this point, the temperature of the aggregate and asphalt cement shall be within the limits of the mixing temperature. Using a spatula (if mixing by hand) or a mixer, mix the aggregate (and stabilizer) and asphalt cement quickly until the aggregate is thoroughly coated.

6.3.1.4.2 PLANT PRODUCED SAMPLES. For plant produced samples, duplicate samples should be tested at the plant production temperature. Samples may be obtained during plant production by sampling the mixture at the trucks prior to the mixture leaving the plant. Samples obtained during actual production should be reduced to the proper test sample size by the quartering method.

6.3.1.5 PROCEDURE. The following procedure can be used for both laboratory and plant produced SMA mixtures.

6.3.1.5.1 TRANSFER the uncompacted SMA mixture sample to a tared wire basket described in figure 2-17. Place the entire sample in the wire basket. Do not consolidate or otherwise disturb the sample after transfer to the basket. Determine the mass of the sample to the nearest 0.1 gram.

6.3.1.5.2 DETERMINE the record the mass of a paper plate to the nearest 0.1 gram. Place the basket on the paper plate the place the assembly into the oven at the temperature as determined for 1 hour ± 1 minute.

6.3.1.5.3 AFTER THE SAMPLE has been in the oven for 1 hour, remove the basket and paper plate. Determine and record the mass of the paper plate to the nearest 0.1 gram.

6.3.1.5.6 CALCULATIONS. Calculate the percent of mixture which drained by subtracting the initial paper plate mass from the final paper plate mass and divide this by the initial total sample mass. Multiply the result by 100 to obtain a percentage.

6.3.1.5.7 REPORT. Report the average percent drainage at the test temperature.

6.3.2 JOB-MIX FORMULA REQUIREMENTS. It is the contractor's responsibility to ensure that, in addition to the aggregate gradation requirements, the produced material will provide an asphalt mixture that conforms to the applicable design parameters listed in table 2-16. The contractor shall submit in writing the proposed job-mix formula (JMF) for approval including the following:

6.3.2.1 THE PERCENTAGE (in units of 1 percent) of aggregate passing each specified sieve (except the 75 m (No. 200) sieve), based on the total dry weight of aggregate as determined by ASTM C-117 and C-136.

6.3.2.2 THE PERCENTAGE (in units of 1/10th of 1 percent) of aggregate passing the 75 m (No. 200) sieve, based on the dry weight of aggregate as determined by ASTM C-117.

6.3.2.3 THE PERCENTAGE (in units of 1/10th of 1 percent) of aggregate finer than 0.020 millimeter in size, based on the dry weight of aggregate as determined by ASTM D 422.

6.3.2.4 THE PERCENTAGE (in units of 1/10th of 1 percent) of asphalt material to be added, based upon the total weight of mixture.

6.3.2.5 THE PROPOSED PERCENTAGE of each stockpile to be used, the average gradation of each stockpile, and the proposed target value for each sieve size. The target values and the combined average gradation of all the stockpiles when combined in accordance with the contractor's recommended stockpile combinations shall be within the gradation ranges for the designated grading in table D1.

6.3.2.6 THE TYPE and amount by weight of mix of stabilizer additive to be used.

6.3.2.7 ADDITIONAL INFORMATION required as part of the JMF shall include the following:

6.3.2.7.1 THE MATERIAL sources for all ingredients.

6.3.2.7.2 THE MATERIAL PROPERTIES, as listed, for all ingredients:

6.3.2.7.2.1 The Specific Gravities of the individual aggregates and asphalt.

6.3.2.7.2.2 THE L.A. ABRASION of the aggregates.

6.3.2.7.2.3 THE SAND EQUIVALENT value of the combined aggregate.

6.3.2.7.2.4 THE FLAT AND ELONGATED PERCENT of the coarse aggregate (3 to 1 and 5 to 1 ratios), retained above the 4.75 mm (No. 4) sieve.

6.3.2.7.2.5 THE PLASTIC INDEX of the aggregate.

6.3.2.7.2.6 THE ABSORPTION of the aggregates.

6.3.2.7.2.7 THE ASPHALT temperature/viscosity curves.

6.3.2.7.3 THE MIXING temperature.

6.3.2.7.4 THE MIX DESIGN test property values and curves used to develop the job mix in accordance with those provided for hot-mix asphalt in this chapter and also in the Asphalt Institute's Manual Series No. 2 (MS-2).

6.3.2.7.5 THE PLOT of the gradation on the FHWA 0.45 power gradation chart.

6.4 MIXING PLANTS. SMA has been mixed in both batch and drum mix plants. These plants can be utilized with none or minor adaptations required to mix the SMA components.

6.4.1 BATCH PLANTS. The mineral filler will be added directly into the weigh hopper. Most batch plants have an existing mechanism for accomplishing this. However, special attention is required to assure accurate proportioning of the relatively large amounts of filler required for SMA. The fiber is also added directly into the weigh hopper and should occur when the hot aggregate is also being placed into the hopper. An alternative method of adding the fibers directly to the pugmill as the hot aggregates are added has also been used successfully.

6.4.2 DRUM-MIX PLANTS. The mineral filler will be added directly into the drum mixer. Special attention is required to assure accurate proportioning of the relatively large amounts of filler added. The fiber is also added directly into the drum mixer. A separate feeding system is usually employed especially in the case of loose fibers. These fibers are added to the aggregates far enough down the drum to avoid direct contact with the burner flame.

6.4.3 MIXING TIME. The time required to mix SMA is usually greater than dense-graded hot-mix asphalt. For batch plants the dry-mixing time will be increased from 5 to 15 seconds and wet-mixing will be increased at least 5 seconds for cellulose fibers and up to 5 seconds for mineral fibers.

6.4.4 STORAGE. Temporary (less than 1 hour) storage of SMA in surge bins will be used only for balanced production capacity. Storage in heated and insulated storage bins should be limited to 4 hours unless laboratory testing indicates additional time is acceptable. Acceptability must be based on no adverse changes in binder properties and excessive draindown not occurring. No mixture shall be stored overnight.

6.4.5 TEST SECTION. The construction of a test section is important to allow examination of the contractor's mixing and placement procedures. This is especially true if the contractor has not had experience in mixing or placing SMA.

6.4.6 PLACEMENT.

6.4.6.1 EQUIPMENT. The trucks, pavers, distributors, and other general equipment are the same as those used for any asphalt concrete construction. Only steel-wheel rollers are used for SMA. Rubber tire rollers are not used for SMA. Vibratory rollers can be used but care must be taken to prevent breakdown (fracture) of the aggregate.

6.4.6.2 SURFACE PREPARATION. The surface shall be cleaned of all loose or deleterious material. A tack coat shall be applied as described in the technical literature. The atmospheric temperature shall be a minimum of 7EC (45EF) and rising at the time of placement.

6.4.6.3 PAVING. The SMA mixture, when delivered to the paver, shall be a minimum temperature equal to the laboratory compaction temperature as determined in ASTM D 1559.

6.4.6.4 COMPACTION. The SMA shall be compacted to a minimum of 94 percent of maximum theoretical density. Rolling shall be accomplished with steel-wheel rollers. Vibratory rollers can be used provided the aggregate is not crushed. Rubber-tire rollers will not perform well with SMA due to the high amounts of asphalt cement in the mixture causing asphalt build up on the wheels. Rolling should continue until the required density is obtained. This is usually controlled through the use of a nuclear density gage. Due to the large amount of binder coating the aggregate particles it is important that the roller drums be properly moistened with water containing small amounts of detergent to prevent adhesion. Traffic should remain off the SMA surface until it has cooled below a minimum of 60 ^{0}C (140 ^{0}F). Flooding with water from a truck after the completion of all rolling has been used to increase the rate of cooling to allow for earlier trafficking.

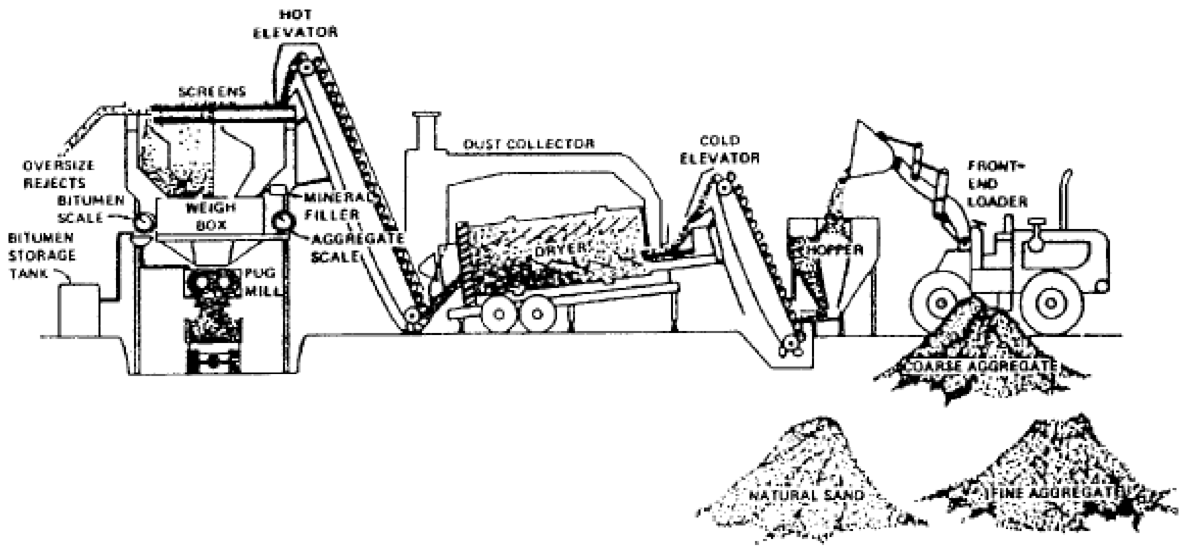

Figure 2-1

Batch plant

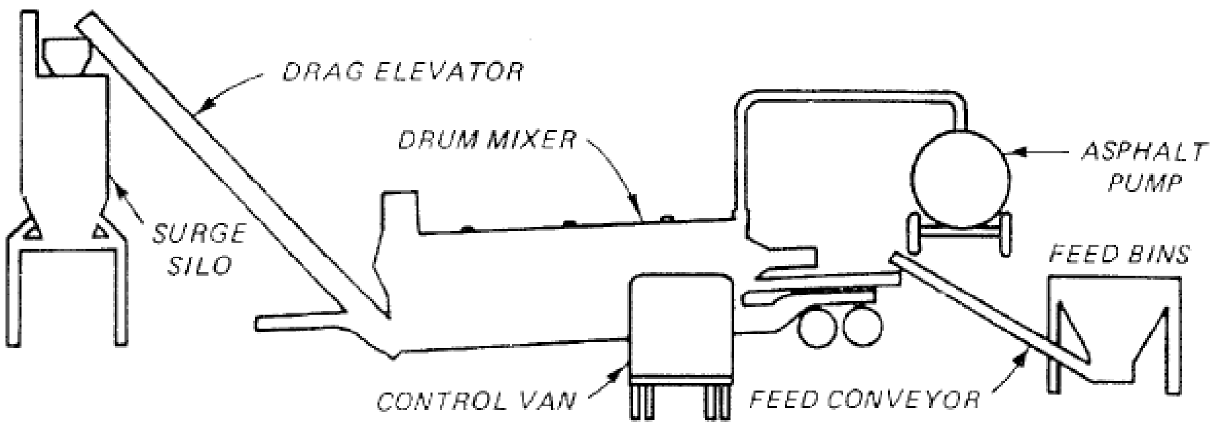

Figure 2-2

Drum mixer

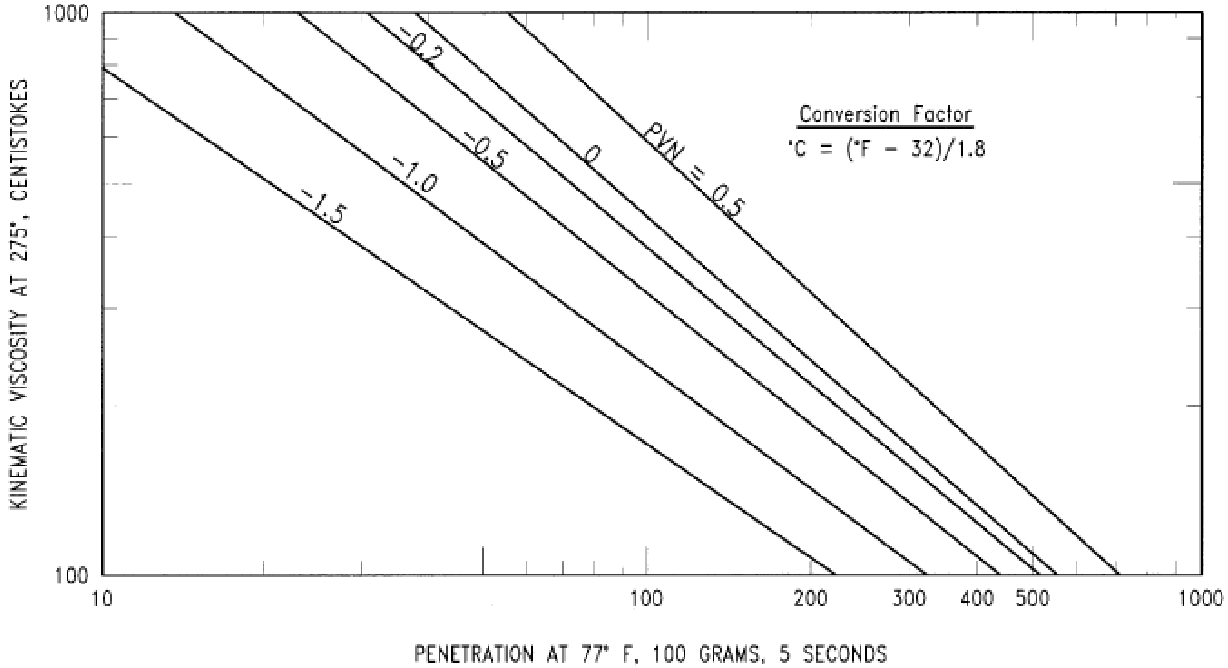

Figure 2-3

Pen-Vis numbers of asphalt cement

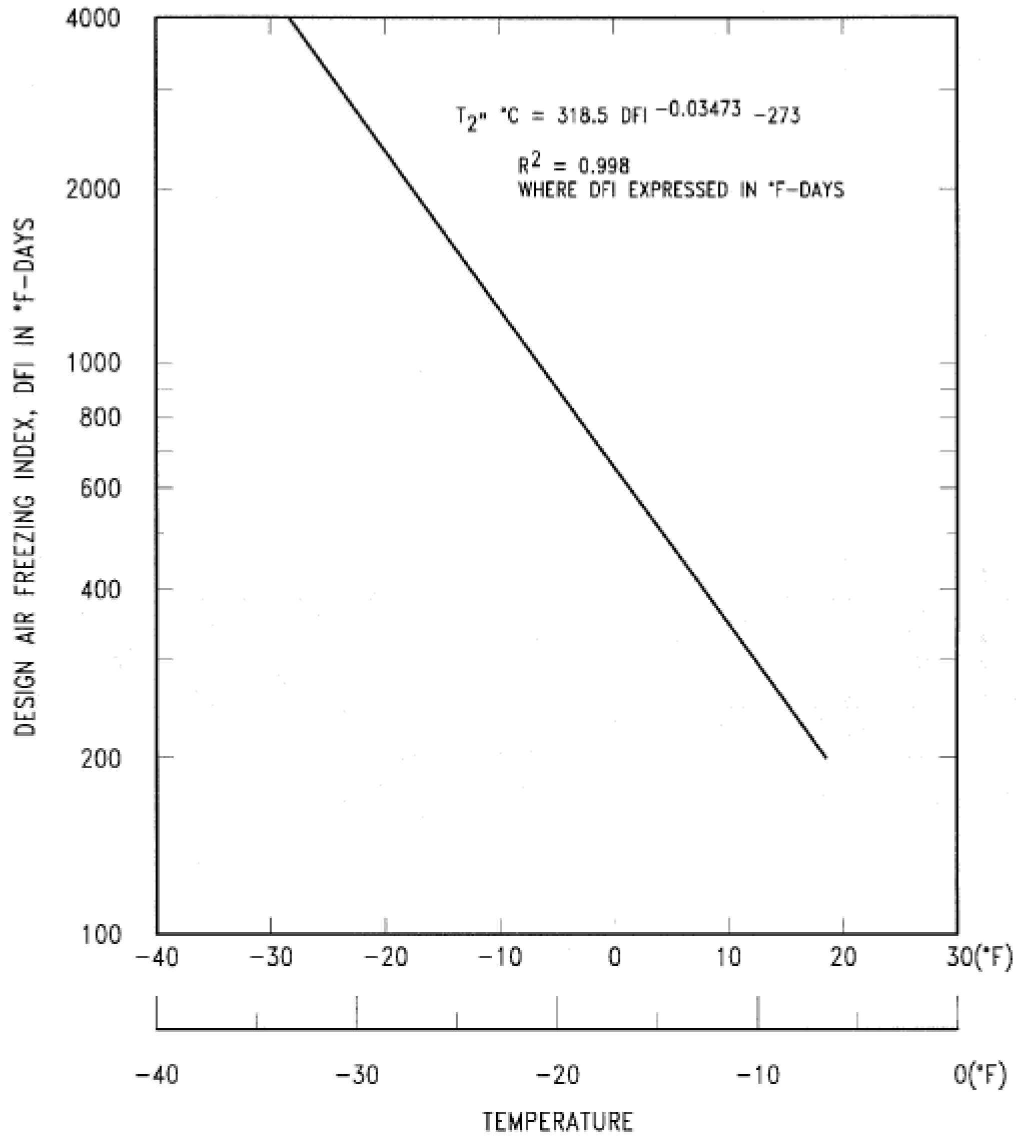

Figure 2-4

Pavement temperature as a function of design air freezing index

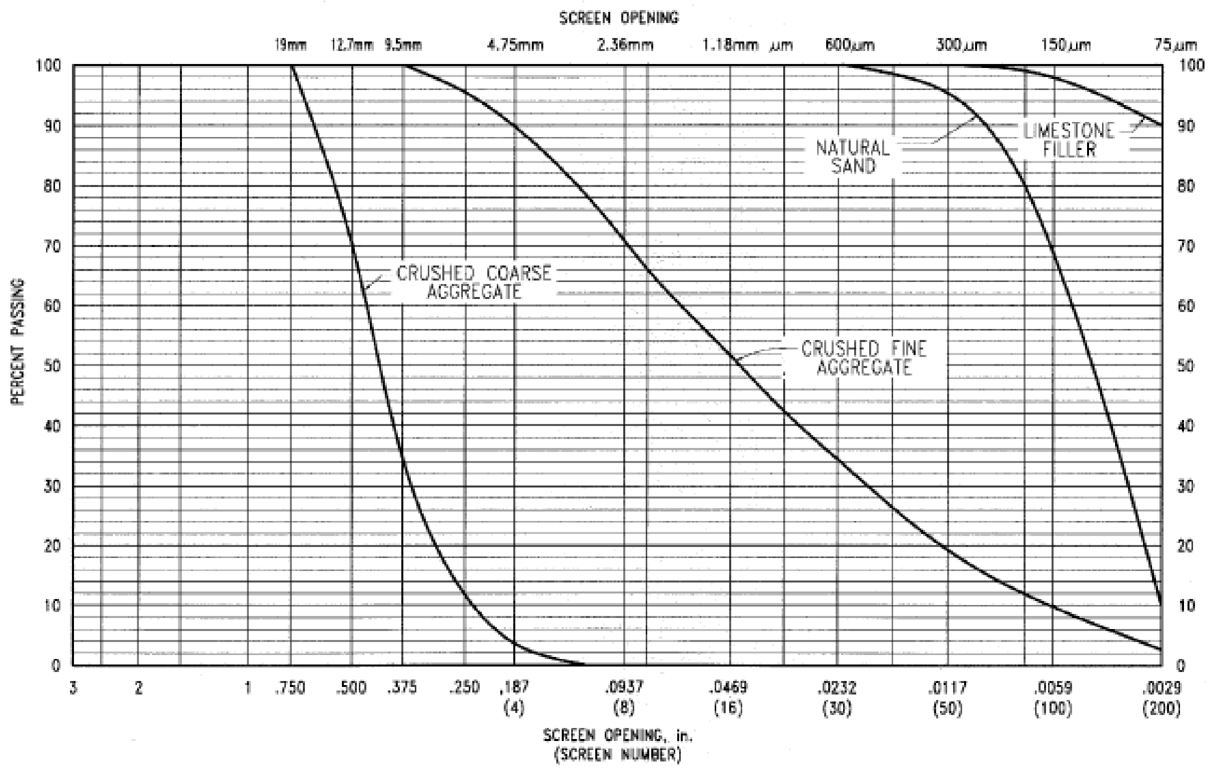

Figure 2-5

Gradation curves for stockpile samples

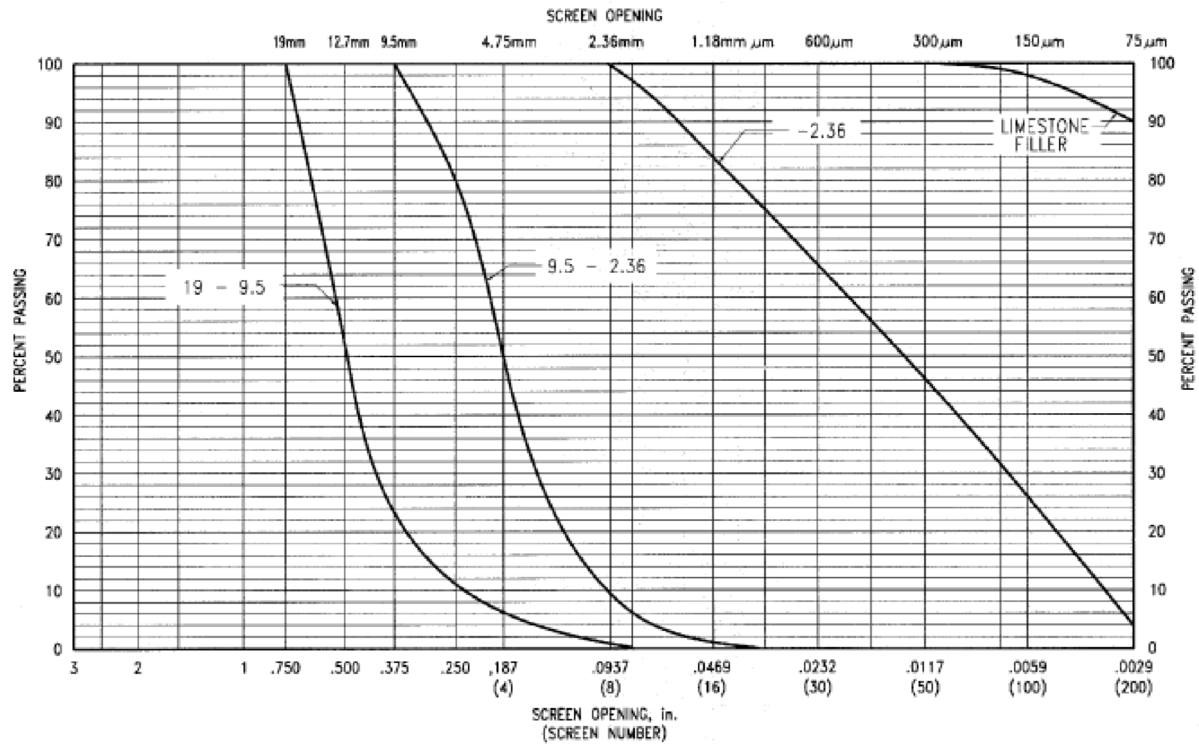

Figure 2-6

Gradation curves for bin samples

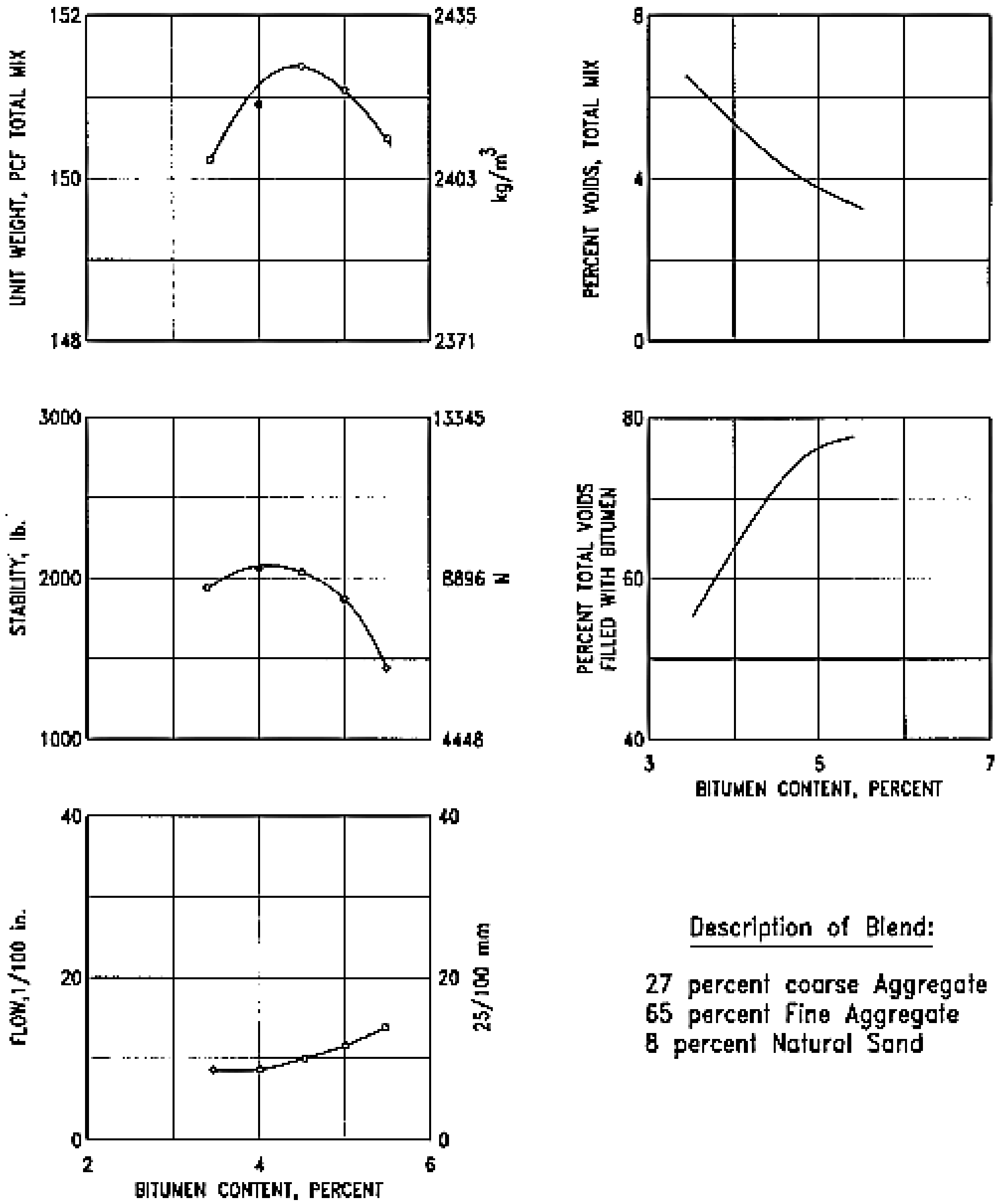

Figure 2-7

Asphalt paving mix design for typical mix

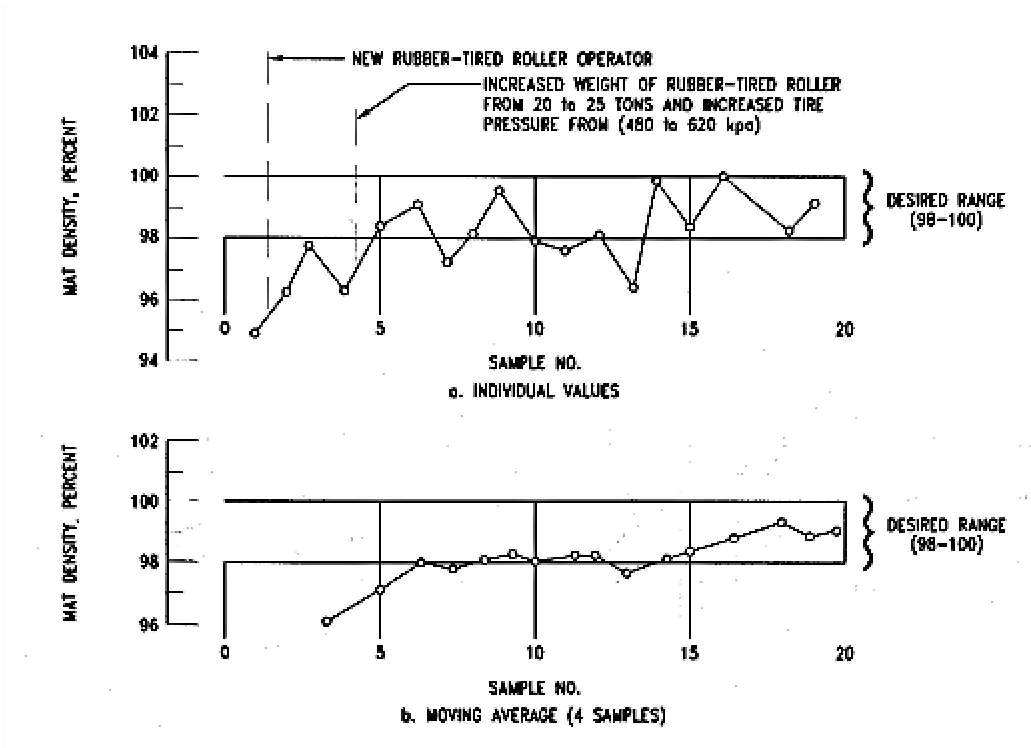

Figure 2-8

Mat density control chart

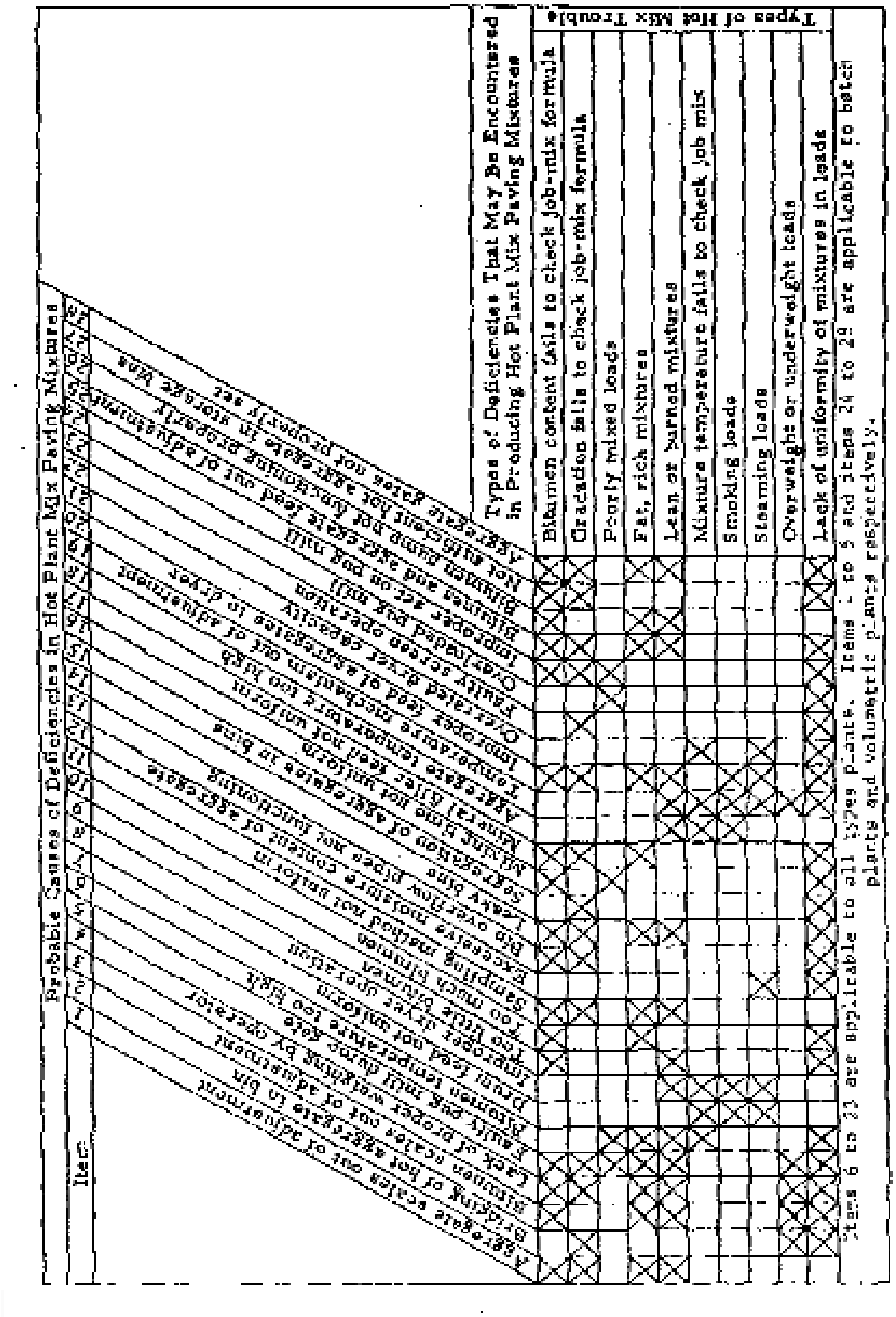

Figure 2-9

Types of hot-mix asphalt deficiencies and probable causes

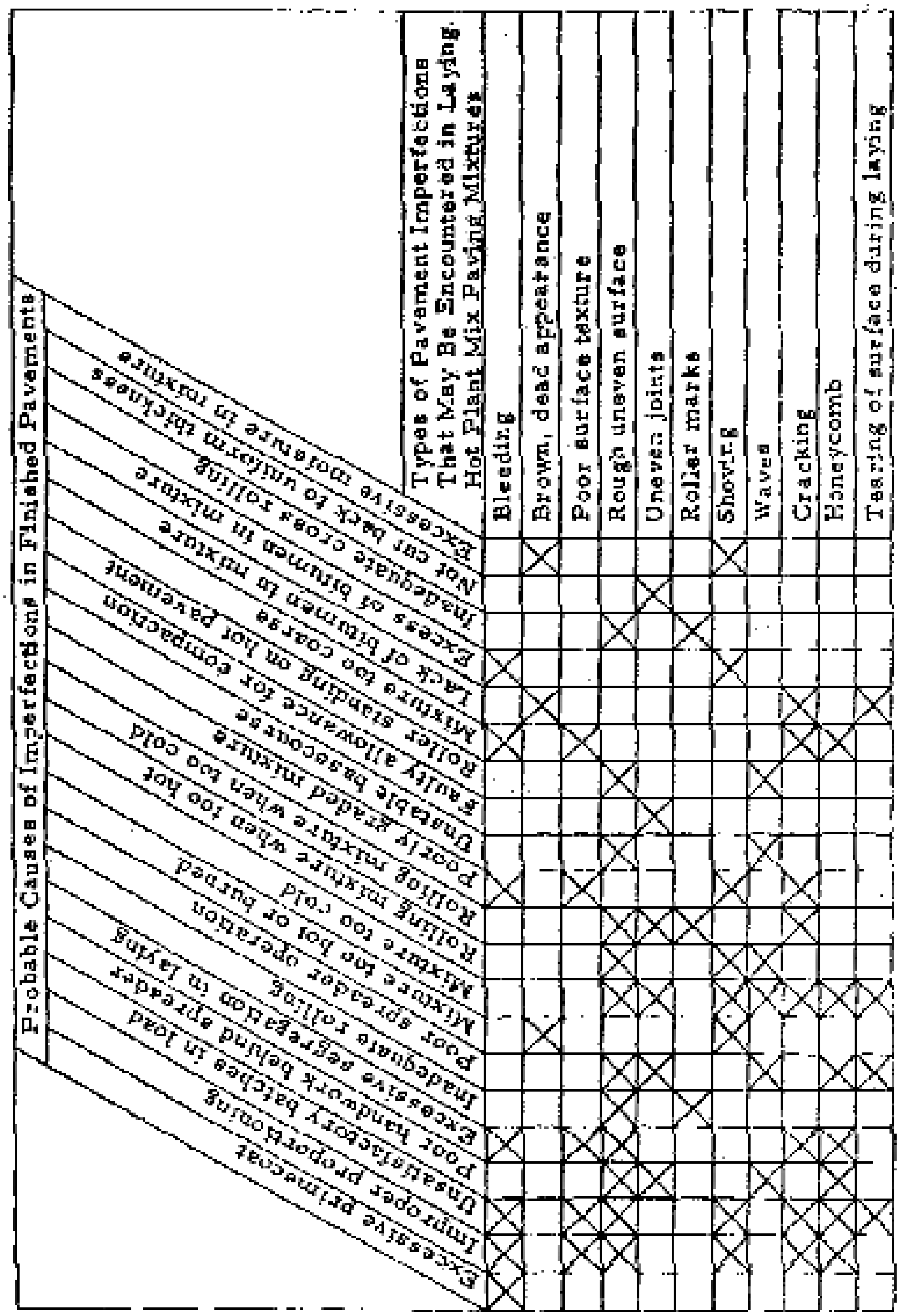

Figure 2-10

Types of hot-mix asphalt pavement imperfections and probable causes

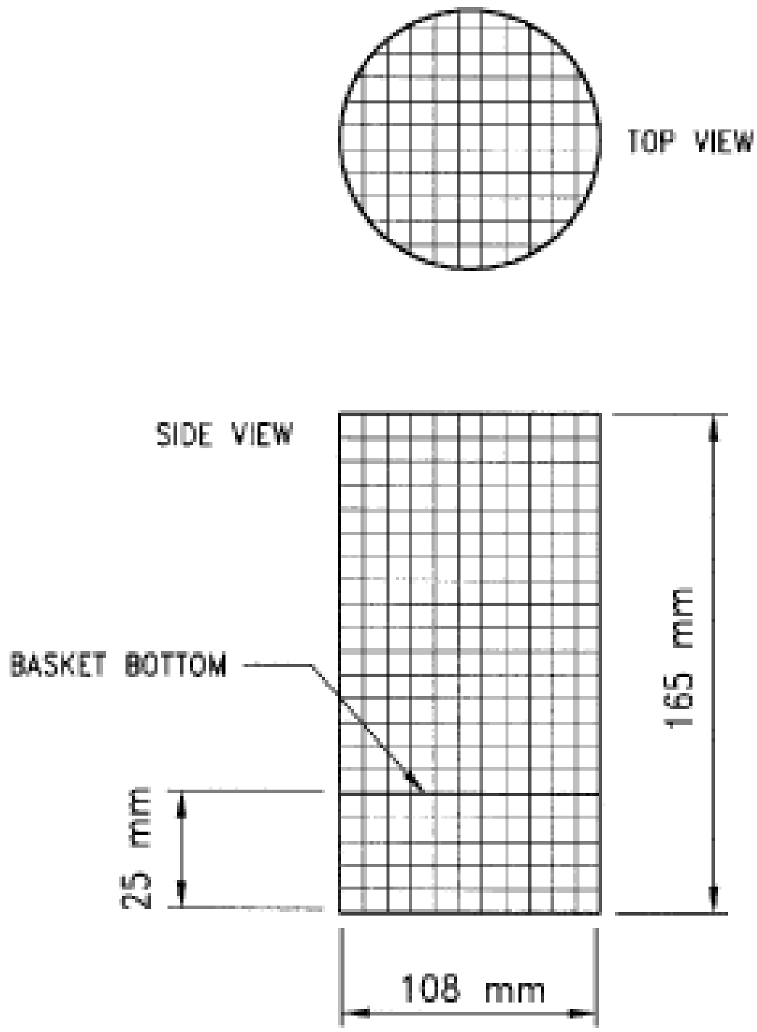

Figure 2-1

Wire basket assembly