Appendix B

Pavement Rehabilitation Techniques

This appendix provides a summary of each of the following pavement rehabilitation techniques:

Asphalt Pavement Rehabilitation Techniques

- Asphalt patching
- Cold milling
- Hot in-place recycling
- Cold-in place recyling
- Asphalt overlay
- Concrete overlay

Concrete Pavement Rehabilitation Techniques

- Full-depth repair and slab replacement
- Partial-depth repair
- Undersealing (slab stabilization)
- Load transfer restoration
- Diamond grinding
- Asphalt overlay
- Asphalt or concrete overlay of fractured concrete slab
- Bonded concrete overlay
- Unbonded concrete overlay

Additional considerations for the application of some of these techniques to existing asphalt-overlaid concrete pavements are mentioned where appropriate.

Each summary contains the following major sections:

- Appropriate use,
- Limitations (for some techniques),
- Concurrent work,
- Materials,

- Design (for some techniques),
- Construction,
- Performance, and
- References.

The topic of design is addressed for some rehabilitation techniques, such as overlays. It is not within the scope of this Guide to present procedures for overlay thickness design in detail. The available approaches to overlay thickness design are briefly summarized, and the reader is referred to the appropriate overlay thickness design references for more detailed guidance.

It is also not within the scope of this Guide to provide typical unit costs for rehabilitation techniques. An agency´s own experience is the best source of information on the costs of rehabilitation.

Asphalt Patching

Appropriate Use

Full-depth or partial-depth patching of an asphalt pavement is localized repair of distresses related to structural damage, materials problems, or construction problems. The repair may be full depth (down to the subgrade or an intact subbase layer) or partial depth (asphalt surface only), depending on the nature of the distress.

Patching may be done on an asphalt pavement either for maintenance purposes or rehabilitation purposes. Maintenance patching is expedient and temporary repair, sometimes done in cold weather, with cold-mixed patching mixtures, and without careful construction procedures. As a result, maintenance patches generally do not exhibit the same long-term stability and durability as hot-mixed patching mixtures, carefully constructed as part of a rehabilitation strategy. Only hot-mix patching for rehabilitation purposes is described here.

Patching (either full depth or partial depth) is most often thought of as the repair for potholes, but several other asphalt pavement distresses can also be repaired by patching. Fatigue cracking in the wheelpaths and transverse thermal cracking can be repaired by full-depth patching. Partial-depth asphalt patching is appropriate for block cracking, slippage cracking, weathering, and raveling. Both full-depth and partial-depth patching can be applied to bleeding and stripping.

Concurrent Work

Patching for rehabilitation purposes may be done alone, as part of restoration of the existing pavement structure, or as preparation for placement of an overlay or surface treatment.

Materials

Hot-mix asphalt concretes are appropriate for long-term repair of asphalt pavements. Asphalt concrete mixtures for use in patching should be of the same aggregate gradation and asphalt cement stiffness as would be used for asphalt paving or resurfacing.

Design

The major design issue related to asphalt patching is determining the amount of patching to be done in conjunction with an overlay. The greater the percentage of medium- and high-severity distress is repaired by patching, the thinner the overlay may be. However, high-quality asphalt patching requires quite a bit of hand labor, so it might not necessarily be most cost-effective to repair 100 percent of the cracking present. The optimal amount of patching depends on the following:

- The amount of medium- and high-severity potholes, fatigue cracking, and other cracking present;
- The required overlay thickness as a function of the amount of distress repaired;
- The unit cost of patching; and
- The unit cost of the overlay, as a function of thickness.

The condition-based 1993 AASHTO[1] and the deflection-based 1993 AASHTO and Asphalt Insititute[2] asphalt overlay design procedures permit the designer to consider the effect of extent of patching on the required overlay thickness. In the 1993 AASHTO condition-based overlay design procedure, the structural coefficients assigned to the existing surface and stabilized base may be selected to reflect the amount of medium- and high-severity alligator and/or transverse cracking left unrepaired. In the 1993 AASHTO deflection-based overlay design procedure, deflections measured in areas of the pavement that will be patched may be excluded from the calculation of the effective modulus (E_p) of the pavement structure above the subgrade. Similarly, in the Asphalt Institute deflection-based overlay design procedure, deflections measured in areas to be patched may be excluded from the calculation of the representative rebound deflection (RRD) of the pavement.

Construction

Quantity Estimation

For the purpose of developing plans and bid documents, a field survey should be conducted to estimate the total area of patching required. The quantity of repair required may be greater than that originally estimated in the earlier pavement evaluation survey. The Asphalt Institute recommends that patch areas extend 1 foot on all sides beyond the edge of the visible deterioration.[3] The total area of the required repairs – not the total area of cracking to be repaired – should be used in the estimation of the total cost of patching.

Marking Repair Boundaries

Repair boundaries should be straight, to help in achieving adequate compaction of the patch material at the edges. Repair areas do not necessarily have to be rectangular.

Removal of Deteriorated Material

When resurfacing is not part of the rehabilitation strategy, partial-depth sawcuts should be made along the repair boundaries. Partial-depth sawing provides straight edges which look nice and which are easy to seal. Full-depth sawcutting of the boundaries is not recommended, because a rough face achieves better bond between the repair material and the existing pavement. When the pavement is to be resurfaced after patching, the repair boundaries may cut more expediently using flat spades on jackhammers.

After the repair boundary has been cut, the deteriorated material within the repair boundary should be chipped out with jackhammers. Removal should begin at the center of the repair area and progress toward the boundaries, as illustrated in Figure B-1.

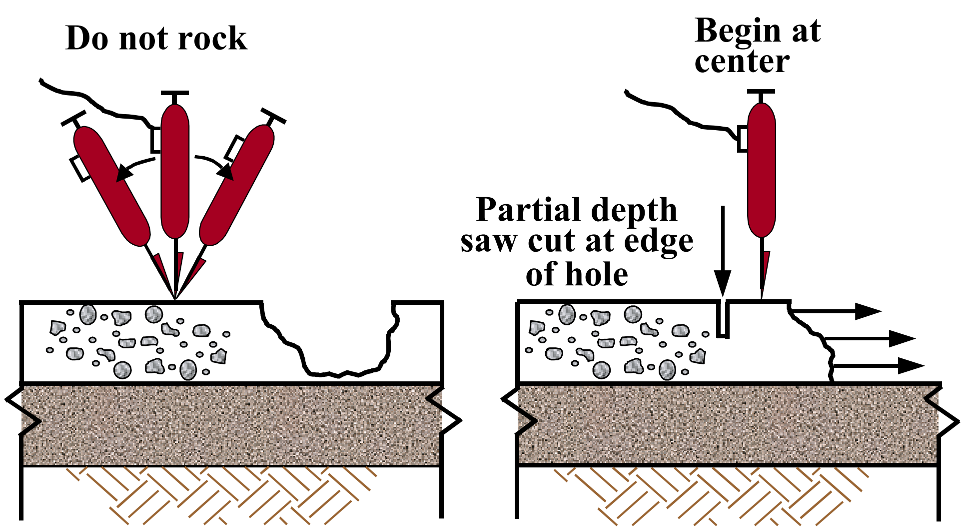

Figure B-1. Removal of material prior to aspalt patching.[4]

Cleaning

The exposed faces of the repair area should be thoroughly cleaned by airblasting to remove loose particles, and should be completely dry prior to placement of the patch material. Any material at the bottom of the repair area which is wet, loose, or soft should be either dried out and recompacted, or removed.

Applying Tack Coat

An emulsion, cutback, or synthetic resin tack coat should be applied to the vertical faces of the repair area, by brushing or low-pressure spraying. If the bottom of the repair area is granular or non-asphaltic treated base material, a prime coat material should be applied to this surface. If the repair extends in depth to the subgrade, it is not necessary to apply a prime coat to the bottom of the repair area.[3]

Material Placement and Compaction

The hot-mix repair material should be placed in the repair and distributed carefully to prevent segregation. The repair material should be placed and compacted in lifts no greater than 6 inches thick. For small repairs, vibratory compaction is adequate (as illustrated in Figure B-2); for larger repairs, a roller should be used for compaction. The best contact between the repair material and repair walls is achieved if compaction begins at the center of the repair area and progresses outward towards the edges. The *Techniques for Pavement Rehabilitation* training course manual[4] recommends that the final surface of the repair after compaction should be about a quarter of an inch higher than the surrounding pavement surface. The Asphalt Institute, on the other hand, recommends that a straightedge be used to check that the final surface of the repair after compaction is level with the surrounding pavement surface.

Sealing Repair Edges

The *Techniques for Pavement Rehabilitation* training course manual[4] recommends that as the last step in construction of an asphalt repair, the edges of the repair be sealed. A liquid asphalt material may be poured along the edges, brushed lightly, and blotted with clean sand.

Additional guidelines on asphalt patching are provided in the Strategic Highway Research Program (SHRP) report, *Asphalt Pavement Repair Manuals of Practice*.[5]

Figure B-2. Vibratory compaction of asphalt patch.[4]

Performance

The key factors influencing the performance of asphalt patching are the appropriateness of its use, the quality of construction, and the properties of the patch material. Asphalt patching may perform well but not be cost-effective for a pavement which has little or no remaining structural life and will soon need resurfacing or reconstruction.

Asphalt patching without an overlay is estimated to have a service life of about 4 to 8 years. Although asphalt patching is perhaps the most commonly used of all rehabilitation techniques, no State DOT estimates of the service life of asphalt patching were found in the review of State DOT publications conducted for the development of this Guide. The American Concrete Pavement Association estimates a service life of 5 to 7 years for asphalt patching without overlay.[6]

Cold Milling

Appropriate Use

Cold milling is the removal of material from an asphalt pavement surface, using carbide bits mounted on a rotating drum. Cold milling may be done across the entire width of a traffic lane, or may be done in smaller areas, as needed. Asphalt concrete can be removed to a depth of a foot or more in a single pass.

Cold milling may be done for one or more of several reasons:

- To texturize the surface prior to resurfacing in order to enhance bond,
- To remove excess asphalt concrete thickness, to remove oxidation at the surface,
- To remove unstable or deteriorated asphalt concrete material,
- To modify the longitudinal and/or transverse grade,
- To maintain or reestablish curb and gutter lines prior to resurfacing,
- To remove rutting, and/or
- To remove bumps.

A cold milling drum is illustrated in Figure B-3. Infrequently, cold milling is done without subsequent resurfacing, to remove rutting and/or bumps, and/or to improve surface friction. In this case, a finer surface texture is obtained using special cold milling drums, with two to three times more carbide bits than on a normal cold milling drum. The surface texture obtained by cold milling depends on the spacing of the carbide bits, the degree of wear of the bits, the rotational speed of the milling drum, and the forward speed of the milling machine.[4]

Figure B-3. Cold milling drum.[4]

Although cold milling is generally considered a rehabilitation technique for asphalt concrete surfaces, it has been done occasionally on concrete pavements. Cold milling is one option for obtaining a rough texture on an existing concrete surface in order to enhance bond between the existing slab and a concrete overlay. It is not considered necessary to apply such aggressive surface preparation methods to a concrete pavement prior to placement of an asphalt overlay. Infrequently, cold milling is done on concrete pavements without subsequent resurfacing.

Limitations

Cold milling of a portion of an asphalt concrete surface will not, of course, completely correct problems which affect the full thickness of the asphalt concrete layer, such as stripping or instability. Complete removal is advisable for an asphalt concrete layer with a serious material problem.

Cold milled asphalt surfaces which are not subsequently overlaid may initially produce considerable tire noise, although this is believed to decline over time as the milled surface is worn down. Cold milled concrete surfaces may be objectionably noisy for longer periods of time, which is one of the reasons that cold milling is not often done on concrete pavements. Another reason is that cold milling produces spalling at joints and cracks in concrete pavements.

Concurrent Work

Cold milling may be done alone, or in conjunction with full-depth repair, partial-depth repair, and/or an asphalt or concrete overlay. Cold milling should be done after any full- and partial-depth patching needed.

Design

One State DOT considers the structural ratio of old asphalt concrete removed by milling to new asphalt concrete to be 3:2. For example, if 1.5 inches of existing asphalt concrete is removed by milling prior to overlay, it must be replaced by 1 inch of new asphalt concrete – in addition to whatever thickness has been selected for the overlay.[7]

Construction

Cold milling of large areas is best done longitudinally, to control the resulting grade and to avoid disrupting traffic in adjacent lanes. Small areas may be milled longitudinally or transversely.

Cold milling drums are available in widths up to 12 ft and down to as little as 1 foot. A normal cold milling drum is covered with blocks, on each of which is mounted one carbide bit. The bits are spaced about 1.5 inches apart. Milling drums which are modified for the purpose of obtaining a finer surface texture have two or three carbide bits mounted on each block. The resulting spacing of the bits is about 0.5 to 0.75 inches.

Careful control of the grade is important in cold milling operations, especially prior to resurfacing. The grade should be held to the same tolerance as for paving operations.

Performance

Although cold milling is widely used to prepare asphalt concrete surfaces for resurfacing, how much cold milling benefits overlay performance has not yet been quantified. The literature on cold milling cites a few examples of asphalt pavements cold milled without subsequent overlay (in Illinois and Washington), and concrete pavements cold milled without subsequent overlay (in Iowa, Oregon, Puerto Rico, and Washington).[4] No State DOT estimates of the service life of cold milling without subsequent overlay were found in the review of State DOT publications conducted for the development of this Guide.

Hot In-Place Recycling

Appropriate Use

Hot in-place recycling is the on-site rejuvenation of aged asphalt concrete material. Hot in-place recycling is usually but not always done in conjunction with resurfacing. Rejuvenating the existing surface prior to placing an overlay enhances bond and discourages reflection cracking. Hot in-place recycling may also be done without a subsequent overlay, to correct surface distresses such as minor corrugations or bleeding.

Surficial distresses which can be corrected by hot in-place recycling of the top inch or so of the existing asphalt concrete surface include bleeding, weathering, and ravelling. Rutting can be removed by hot in-place recycling (although this does not correct any mix deficiency which may have been responsible for the rutting). Hot in-place recycling to a greater depth can be used to address distresses such as fatigue cracking, block cracking, thermal cracking, longitudinal cracking, slippage cracking, shoving, potholes, bumps, settlements, and heaves.

Limitations

Hot in-place recycling only improves the existing asphalt concrete layer to the depth which is recycled. It does not prevent the recurrence of distresses related to deficiencies in the asphalt concrete mix. Hot in-place recycling does not correct stripping in asphalt concrete mixes. Indeed, some studies suggest it may reduce resistance to water damage.[8, 9] Other potential problems cited in some surveys of agencies which use hot in-place recycling include the following:[4]

- Concerns about the finished pavements: problems with inadequate milling depth, insufficient mixing, and/or inadequate compaction, resulting in segregation, low density, rapid reappearance of cracks, and/or inadequate smoothness.

- Equipment-related problems: frequent breakdowns, excessive smoke and steam, excessive cooling of the recycled mix prior to laydown, poor functioning of recycling equipment in windy, cool, or wet weather.

Concurrent Work

Hot in-place recycling should be done after any needed full- and partial-depth patching, and before placement of an asphalt or concrete overlay, if any.

Materials

Liquid rejuvenating agents for scarified asphalt concrete materials are liquids applied at a controlled rate, predetermined by laboratory testing, and controlled in the field by coordination with the rate of forward movement of the recycling train.

The material properties of an aged, brittle mix can be improved through hot in-place recycling with the use of a rejuvenating agent. On the other hand, a mix which is too soft can be also be improved through hot in-place recycling, by heating the material but not adding a rejuvenating agent.[8] For example, a study of ten hot in-place recycling projects in Alberta, Canada found that adding a rejuvenating agent increased the asphalt binder penetration by about 30 percent, while recycling without the addition of a rejuvenating agent reduced binder penetration by about 20 percent.[10]

The presence of chip seals within the planned depth of recycling must be taken into consideration in the mix design for the recycled material. Adjustments may be needed to compensate for the fairly uniform aggregate gradation and high asphalt binder content of the chip seal layer.

The types of distress being addressed by the hot in-place recycling should also be taken into consideration in the mix design of the recycled material. Bleeding is an indication of insufficient air voids in the existing layer. Rutting may be due at least in part to instability in the existing asphalt concrete mix – or it may be a reflection of rutting in underlying granular and/or subgrade layers.

According to the 1994 NCHRP synthesis on hot in-place recycling,[11] mix design procedures have not yet been established specifically for hot in-place recycling mixtures. The synthesis recommends that the mix design guidelines given for central hot plant recycling in NCHRP Report 224[12] may also be applied to hot in-place recycling mix design.

Design

In a 1994 survey of State DOTs' usage of hot in-place recycling,[11] seventeen State DOTs reported that they attribute some structural value to hot in-place recycling. Fourteen States reported that the considered the structural value of hot in-place recycling to be about the same as virgin asphalt concrete material. The other three States attribute to hot in-place recycled material a structural value slightly lower than they attribute to virgin asphalt concrete material.

Construction

The Asphalt Recycling and Reclaiming Association (ARRA)[13] defines three types of hot in-place recycling operations: heater scarification, repaving, and remixing. Each of these is described below.

Heater scarification involves the following steps:

- Heating the existing pavement surface to about 110 to 150ºC, using one ore more propane-fired radiant heaters,
- Scarifying the softened surface to a depth of about one half to three quarters of an inch.
- Applying a liquid rejuvenating agent (if needed),
- Mixing and leveling the loose mixture with an auger and/or laydown machine, and
- Compacting with rollers.

Repaving is heater scarification combined with placement of a new asphalt concrete overlay. The process involves the following steps:

- Heating the existing pavement surface to about 190ºC, using infrared heaters,
- Scarifying the softened surface to a depth about one half to three quarters of an inch,
- Applying a liquid rejuvenating agent (if needed),
- Mixing the loose mixture with an auger,
- Spreading and screeding the recycled mixture,

- Placing a new asphalt concrete layer over the recycled mixture, and
- Compacting with rollers.

Remixing is similar to repaving, but involves mixing mineral aggreagate or new asphalt concrete hot-mix into the scarified, rejuvenating material, rather than placing a layer of new asphalt concrete on top. Remixing not only increases the structural capacity of the pavement, as does repaving, but also permits improvement to the gradation or binder properties of the existing asphalt concrete layer. Remixing involves heating and reworking material to a greater depth than in heater scarification and repaving. The steps in the remixing process are the following:

- Heating the existing pavement surface to about 85 to 105ºC, using one ore more propane-fired radiant heaters,
- Milling the softened surface to a depth of about 1 to 2 inches,
- Mixing the hot milled material, rejuvenating agent, and heated new asphalt concrete material in a pugmill,
- Placing the mixture, and
- Compacting with rollers.

Performance

Hot in-place recycling without an accompanying overlay or addition of new asphalt concrete material is estimated to have a service life of about 4 to 8 years. New York estimates a service life of years for hot in-place recycling of 1 to 1.5 inches, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] Vermont estimates a service life of 6 to 10 years for hot in-place recycling of 1 to 3 inches.[15]

How much hot in-place recycling in conjunction with an overlay or additional asphalt concrete thickness benefits overlay performance has not yet been quantified.

Cold In-Place Recycling

Appropriate Use

Cold in-place recycling (CIPR) involves the reutilization of the asphalt bound layers, and the granular layers in a flexible pavement structure. This process is done on the pavement, without heat being added.[16] The use of a stabilizer additive is optional with emulsions being used most often, with lime and cement being used also.[17]

The Asphalt Recycling and Reclaiming Association (ARRA) differentiates two different CIPR procedures as full depth and partial depth.[18] Partial-depth CIPR involves the recycling of the asphalt bound layers to a depth of 75 to 100 mm (3 to 4 inches). Full-depth CIPR, also termed full-depth reclamation, involves the recycling of the asphalt bound layers and the unbound granular layers in the flexible pavement.

Limitations

CIPR has been performed on all types of roadways with the concentration being on lower-volume roadways. However, full-depth reclamation has successfully been conducted on high volume Interstate pavements. This process can directly address structural problems through the production of an improved stabilized layer when full depth reclamation is used. Partial-depth reclamation is limited to correcting only those distresses which are surface problems in the asphalt layer.

Some agencies have limited use of CIPR to rural areas, and pavements with low traffic volumes. The use of CIPR for production of a base course is the preferred procedure for 95 percent of states with only 5 percent considering it sufficient for use as a surface material. Surfacing options included aggregate surface treatments and hot mix asphalt overlays as the preferred alternatives.

The partial-depth process utilizes two procedures, a single machine, and a single pass equipment train to accomplish the recycling. The single machine has difficulty maintaining proper aggregate size in the milled material which should be watched closely with this option. The single pass train option has been used most extensively, particularly where quality control is of higher concern with higher-traffic-volume pavements.

Concurrent Work

CIPR can be performed with no concurrent work being required prior to the recycling operation. Normally an overlay is placed following the CIPR procedure.

Materials

The materials involved in CIPR include asphalt bound materials, unbound granular materials, and even subgrade soils. Materials added during the process include new hot mix asphalt, new aggregates, liquid asphalt bitumens (primarily emulsions), lime, cement, and flyash. The quantities of these additives are controlled through field sampling and mix designs on samples representative of the materials being recycled. This requires sampling asphalt, granular, and subgrade if the depth of recycling will include each of these materials. Laboratory mix design procedures must be conducted on mixtures constructed in proportion to the materials to be milled.[19]

The presence of different surface materials such as surface treatments, chip seals, cold patches, etc. do not affect the ability to perform CIPR on a pavement. Their impact will be on the amount of additive required, and changes in the final gradation provided by the recycled material which may require additional aggregate to be added to compensate for an improper gradation.

The use of different material additives to accomplish the stabilization in the recycling requires different considerations as to the properties of the final material produced, and the corresponding changes required in the construction process to accommodate the characteristics of materials such as an emulsion and a Portland cement, to ensure the ultimate strengths are achieved. There are new developments in the use of foamed asphalt[20] in the CIPR methods which may provide improved binders with a subsequent improvement in structural properties. This is still a new process, but there are projects under traffic.

The result of a CIPR project is a significant improvement in the structural capacity of the pavement section, particularly with the full-depth reclamation procedure. The material produced in full-depth reclamation highly dependent on the materials being recycled and their proportions. A bituminous recycling operation produces a material that is between hot mix asphalt concrete, and a bituminous stabilized base material. Because of the variability in materials it is recommended that each agency conduct the proper strength tests on representative recycled materials to ascertain their individual strength characteristics for selection of appropriate structural parameters.[21,22]

Construction

The Asphalt Recycling and Reclaiming Association[21] recognizes two methods of CIPR: partial-depth and full-depth reclamation. Each method requires separate considerations to the construction process used.

Partial-depth reclamation involves the use of a single machine or an equipment train to pulverize, add and mix the stabilizing agent, and place the mixture on the roadway. The single machine accomplishes these steps using one piece of equipment. This limits the depth of recycling, and provides little control on the inclusion of oversize RAP in the recycled mixture.

The equipment train provides for the inclusion of peripheral equipment to accomplish select steps such as adding the stabilizer and mixing separate from milling, and crushers and screens to control the size of RAP used in the final mixture.

For partial-depth recycling the depth limitation of the single machine is not an issue normally, however, the inability to control oversize RAP is critical to producing a quality mixture, and equipment trains are recommended for projects dealing with higher- traffic-volume pavements.

Full-depth reclamation can involve either the single machine or the equipment train, but the equipment train is more prevalent for this option to utilize the extra control provided for gradation control, additive mixing, and moisture control for the higher traffic volume pavements. ARRA recognizes that this recycling can be accomplished in a multiple step process or in a two step process.

In the two-step process the pulverization and sizing is done with a single cold milling or pulverization device. The addition of the stabilizer and the mixing is performed with a separate piece of equipment. By contrast, the multiple-step sequence involves breaking the pavement with a motor grader equipped with ripper teeth, often supplemented with a sheepsfoot or grid roller for sizing. A separate piece of equipment for final sizing such as a towed hammermill is required. These are followed by mixers for addition of the stabilizing agent.

The two-step process is conducted in a manner similar to that used in partial-depth recycling, and the equipment is similar.

Apart from the equipment, there are serious material considerations which must be recognized in this process. Moisture control is critical to the long term strengths of these materials. When emulsion is used the moisture must be monitored as this can impact coating and bonding of the asphalt in the final mixture. When a reactive additive such as lime or cement is used, there must be sufficient moisture present when the mixture is compacted to provide water for hydration of the additive for strength gain. This water must be sealed in immediately if the lime or cement is to cure properly. However, the water in the emulsion must be allowed to evaporate sufficiently before compaction, and even moreso following compaction to provide the strength increase. The bituminous material must not be covered immediately.

Performance

Successful projects have been constructed on pavements of all traffic volumes. The case studies presented in the FHWA report on *Recycling Guidelines for State and Local Governments*[16] illustrate how acceptable reconstruction can be achieved using these techniques. Records on performance are, however, highly variable as there has not been a common definition applied to judge the comparative performance levels. Studies report on benefits achieved using CIPR, and list the causes commonly noted for poor performance:[4]

- Use of an excessive amount of recycling agent;
- Application of a surface seal prematurely;
- Recycling only to the depth of an asphalt layer, resulting in delamination from the underlying layer; and/or
- Allowing project to remain open for too long into the winter season.

Asphalt Overlay of Asphalt Pavement

Appropriate Use

An asphalt overlay of an asphalt pavement may be placed to improve ride quality and/or surface friction, or may be placed for the purpose of substantially increasing structural capacity.

A thin asphalt overlay is appropriate for pavements with functional deficiencies only, such as excessive roughness, poor surface friction, excessive rutting, and distresses such as bleeding, weathering, ravelling, bumps, settlements, and heaves. A thicker asphalt overlay is appropriate for pavements with insufficient structural capacity for the traffic anticipated over the design life of the rehabilitation. An asphalt overlay placed to correct a structural deficiency will also correct any functional deficiencies present.

As a general rule, an asphalt pavement is considered to require a structural improvement when 50 percent of the wheelpath area (equivalent to about 10 percent of the total area) of the outer traffic lane has medium- to high-severity alligator cracking.[23] A critical rutting level of one half inch is often cited as indicative of a need for structural improvement. However, rutting may have causes related not only to the load-bearing capacity of the pavement layers, but rather the stability of the mix, so the cause of rutting should be examined before deciding whether or not a structural improvement is the appropriate remedy.

Limitations

Functional asphalt overlays are not appropriate for pavements with little or no remaining structural life, as evidenced by the amount of medium- to high-severity alligator cracking present, and/or measured deflections. Serious materials problems, such as stripping in asphalt concrete layers, or settlement or swelling of foundations soils, can only be temporarily mitigated by an asphalt overlay.

The 1993 AASHTO Guide[1] identifies the following as conditions under which an asphalt overlay may <u>not</u> be a feasible rehabilitation technique for an asphalt pavement:

- The amount of high-severity alligator cracking is so great that complete removal and replacement of the existing surface is dictated.

- Existing surface rutting indicates that the existing materials lack sufficient stability to prevent recurrence of severe rutting.
- An existing stabilized base shows signs of serious deterioration and would require an inordinate amount of repair to provide uniform support for the overlay.
- An existing granular base must be removed and replaced due to infiltration of and contamination by a soft subgrade.
- Stripping in the existing asphalt concrete surface dictates that it should be removed and replaced.

Inadequate vertical clearance at bridges for the required overlay thickness is cited in the 1993 AASHTO Guide as a potential limitation for other combinations of overlay type and pavement type, and is also a concern for asphalt overlays of asphalt pavements. The 1993 AASHTO Guide states that vertical clearance problems may be addressed by reducing the overlay thickness under bridges (although this may result in early failure at these locations), by raising the bridges, or be reconstructing the pavement under the bridges. Thicker asphalt overlays may also necessitate raising signs and guardrails, as well as increasing side slopes and extending culverts. Sufficient right-of-way must be available or obtainable to permit these activities.[23]

Concurrent Work

An asphalt overlay of an asphalt pavement may be used alone, or in combination with full- and/or partial-depth asphalt patching to repair cracks and potholes, and/or in combination with cold milling or hot in-place recycling to remove rutting and improve the top of the existing asphalt concrete layer.

Materials

Asphalt concrete mixes used for overlays are usually of the same mix design as those used for new asphalt pavement construction.

Design

The two most commonly used approaches to structural design of asphalt overlays of asphalt pavements are (1) the structural deficiency approach, exemplified by the 1993 AASHTO procedure[1]; and (2) the deflection-based approach, exemplified by the Asphalt Institute procedure.[2] Much less common is the mechanistic approach, in which fatigue and rutting performance are predicted using mechanistic-empirical models.

Structural Deficiency Approach

The concept of the structural deficiency approach to overlay design is that the overlay satisfies a deficiency between the structural capacity required to support traffic over some future design period, and the structural capacity of the existing pavement. In the 1993 AASHTO overlay design methodology,[1] the former is referred to as the future Structural Number (SN_f) and the latter is referred to as the effective Structural Number (SN_{eff}). The structural deficiency concept is illustrated in Figure B-4. There are eight steps in the process of determining the overlay thickness which will satisfy the structural deficiency, $SN_f - SN_{eff}$:

1. Characterization of existing pavement design: specifically, the thicknesses and material types of each pavement layer, and any available subgrade soil information.

2. Traffic analysis: determination of the future 18-kip ESALs anticipated in the design lane over the analysis period, and if possible, the past 18-kip ESALs over the life of the existing pavement as well.

3. Condition survey: quantities of alligator cracking and transverse cracking, mean rut depth, and evidence of pumping.

4. Deflection testing: recommended to determine the *in situ* subgrade resilient modulus (as a function of deflections measured sufficiently far away from the load plate) and a deflection-based estimate of the effective Structural Number of the pavement structure.

5. Coring and materials testing: recommended to obtain subgrade soil samples for laboratory resilient modulus testing, and to obtain samples of the pavement layers for visual examination of their condition.

6. Determination of the required Structural Number for future traffic (SN_f): done using the 1986 AASHTO model for design of new asphalt pavement, with the design subgrade resilient modulus either obtained from laboratory testing or estimated from the backcalculated *in situ* subgrade modulus.

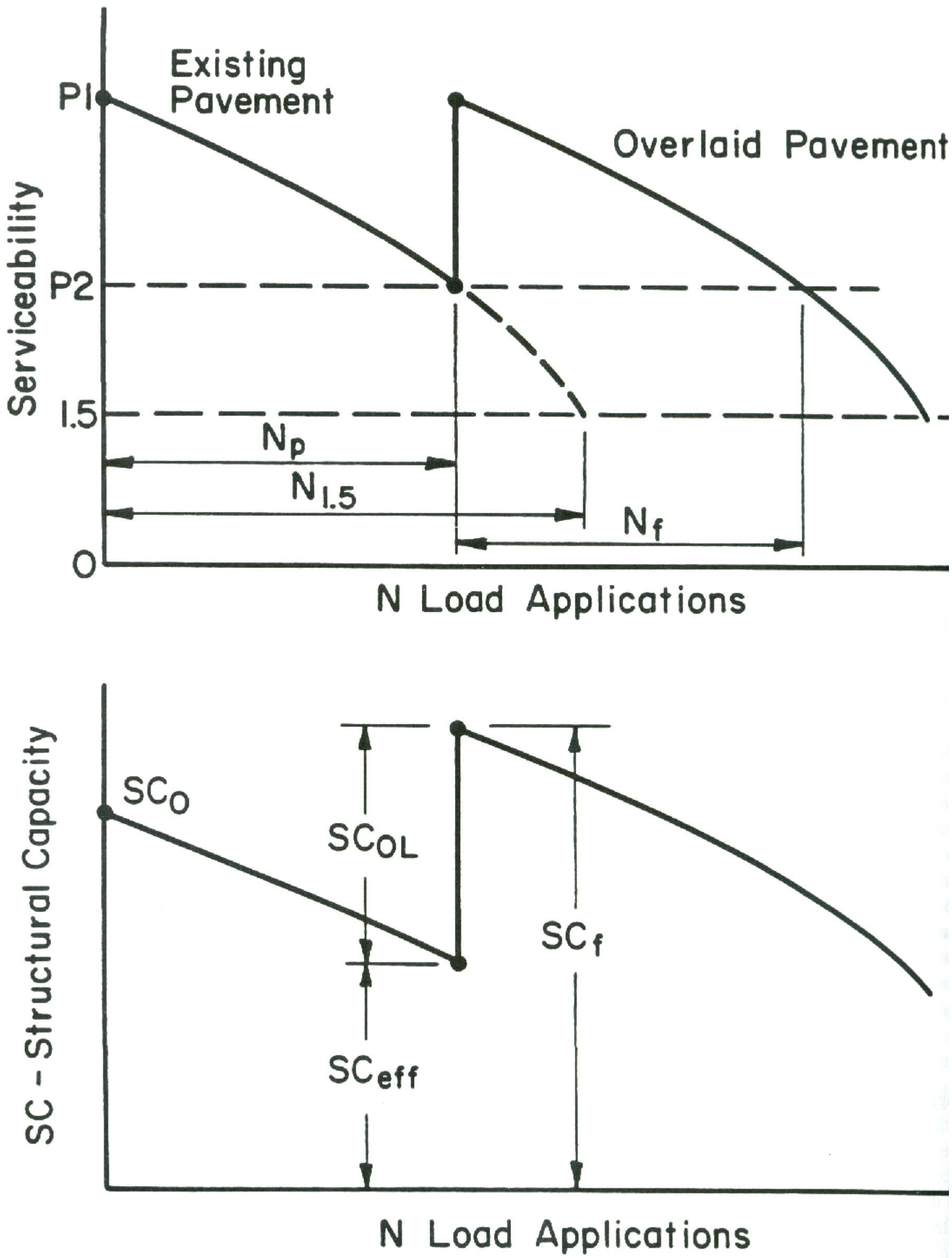

Figure B-4. Structural deficiency concept.[1]

7. Determination of the effective Structural Number of the existing pavement (SN_{eff}) by one or more of three methods:

   - The NDT method – SN_{eff} is estimated as a function of the thickness and effective modulus of the pavement structure. The latter is in turn determined as a function of the mean maximum deflection measured under the load plate (normalized for temperature and load level), the *in situ* subgrade modulus, the thickness of the pavement structure, the load plate pressure, and the load plate radius.

   - The condition survey method – SN_{eff} is estimated by assigning layer coefficients to the existing pavement layers which reflect the types and amounts of deterioration present.

   - The remaining life method – SNeff is estimated as a function of the past 18-kip ESALs carried by the existing pavement as a proportion of the 18-kip ESALs a pavement of its original structural capacity would be expected to be able to carry. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

8. Determination of the required overlay thickness: by dividing the structural deficiency (SN_f – SN_{eff}) by the layer coefficient appropriate for new asphalt overlay material.

The Asphalt Institute overlay design manual also presents a structural deficiency method for asphalt overlay design. Several State DOTs have adapted either the 1993 AASHTO or the Asphalt Institute structural deficiency method to their own purposes for design of asphalt overlays of asphalt pavements.

Deflection-Based Approach

The most widely known deflection-based approach to design of asphalt overlays of asphalt pavements is the Asphalt Institute procedure.[2] State DOTs that have deflection-based procedures for design of asphalt overlays of asphalt pavements include Ohio[7] and Arizona.[24]

The concept of the deflection approach to asphalt overlay design is that the overlay reduces load-induced deflection in the pavement to a level associated with the predicted life of the overlaid pavement. In the Asphalt Institute overlay design methodology, the design deflection level is referred to as the representative rebound deflection (RRD). There are five steps in the process of determining the overlay thickness which will reduce the representative rebound deflection to an acceptable level:

1. Select length of pavement for structural evaluation: divide the project into sections which are uniform with respect to pavement condition, subgrade strength, and drainage conditions. These uniform sections may subsequently be subdivided with respect to measured deflections.
2. Deflection survey: deflections are measured in the outer wheelpath using a Benkelman Beam and static loading. If some other deflection testing device is used (e.g., Falling Weight Deflectomoter, Road Rater, Dynaflect), the Benkelman Beam deflections must be estimated using correlations.
3. Calculate the representative rebound deflection: as the temperature-normalized mean deflection plus two standard deviations, adjusted for some climates by a critical period adjustment factor.
4. Design traffic: the expected 18-kip ESALs over the overlay design period are determined. It should be noted that unlike the 1993 AASHTO overlay design procedure, the Asphalt Institute overlay design procedure does not call for the application of a reliability factor (i.e., a safety factor) to the traffic input. Instead, a safety factor is applied to the deflection input, by designing not for the mean deflection but rather the mean plus two standard deviations.
5. Required overlay thickness: a design chart is provided in the Asphalt Institute overlay design manual[2] which yields the required asphalt overlay thickness as a function of the representative rebound deflection and the design ESALs.

Deflection-based procedures may have been developed using mechanistic-empirical models for fatigue and/or other distresses, or they may have been entirely empirical in their development, but in either case, in their application, they go straight from deflection measurements to required overlay thickness. Other empirical parameters are often incorporated in the formula as well. These other empirical parameters may improve the prediction of the required overlay thickness, but may also limit the applicability of the procedure to other locations outside the range for which the formula was developed.

In Arizona's overlay design procedure, for example, the required overlay thickness is calculated as a function of design ESALs, a seasonal variation factor depending on location, the Mays meter roughness, the spreadability index calculated from preoverlay FWD or Dynaflect deflections, and an outer sensor FWD or Dynaflect deflection.

Mechanistic-Empirical Approach

In a mechanistic-empirical approach to design of asphalt overlays of asphalt pavements, performance of the overlay is predicted using mechanistic-empirical distress models. The distresses considered should include at least fatigue cracking, and ideally rutting and thermal cracking as well. The existing pavement layers and foundation are characterized using nondestructive deflection testing and backcaculation of their elastic moduli. Material properties for the overlay are assumed. The overlay thickness which will yield acceptable performance in terms of the distresses considered is determined by iteration. A conceptual overview of the mechanistic-empirical approach to design of asphalt overlays of asphalt pavements is given by Monismith.[25]

The individual tools used in mechanistic-empirical design of asphalt pavements (fatigue models, rutting models, seasonal adjustment, etc.), can be adapted to some extent to design of asphalt overlays. However, there are additional aspects of the problem that need to be considered in order to develop a full design procedure for asphalt overlays of asphalt pavements. Among these are consideration of the extent, type, and quality of preoverlay repairs, prediction of reflection crack propagation and deterioration (a problem for asphalt overlays of both asphalt and concrete pavements), and calibration of asphalt overlay performance prediction models to the observed performance of asphalt overlays.

Several examples of mechanistic-empirical procedures for design of asphalt <u>pavements</u> exist, such as the Shell procedure,[26] the Asphalt Institute procedure,[27,28] and the NCHRP 1-26 procedure.[29] Fewer example exist, however, of mechanistic-empirical procedures for design of asphalt <u>overlays</u> of asphalt pavements. Among the few State DOTs that have developed a mechanistic-empirical design procedure for asphalt overlays of asphalt pavements are Washington,[30] Idaho, and Nevada.[31,32]

The Washington State DOT procedure uses a model to predict fatigue as a function of horizontal tensile stress at the bottom of the asphalt overlay and at the bottom of the original asphalt layer, as well as a model to predict rutting as a function of vertical compressive stress at the top of the subgrade. The critical stress locations considered are illustrated in Figure B-5. A flowchart of the Washington State procedure is illustrated in Figure B-6. The overlay thickness required to keep fatigue and rutting below critical levels is determined through a process of iteration.

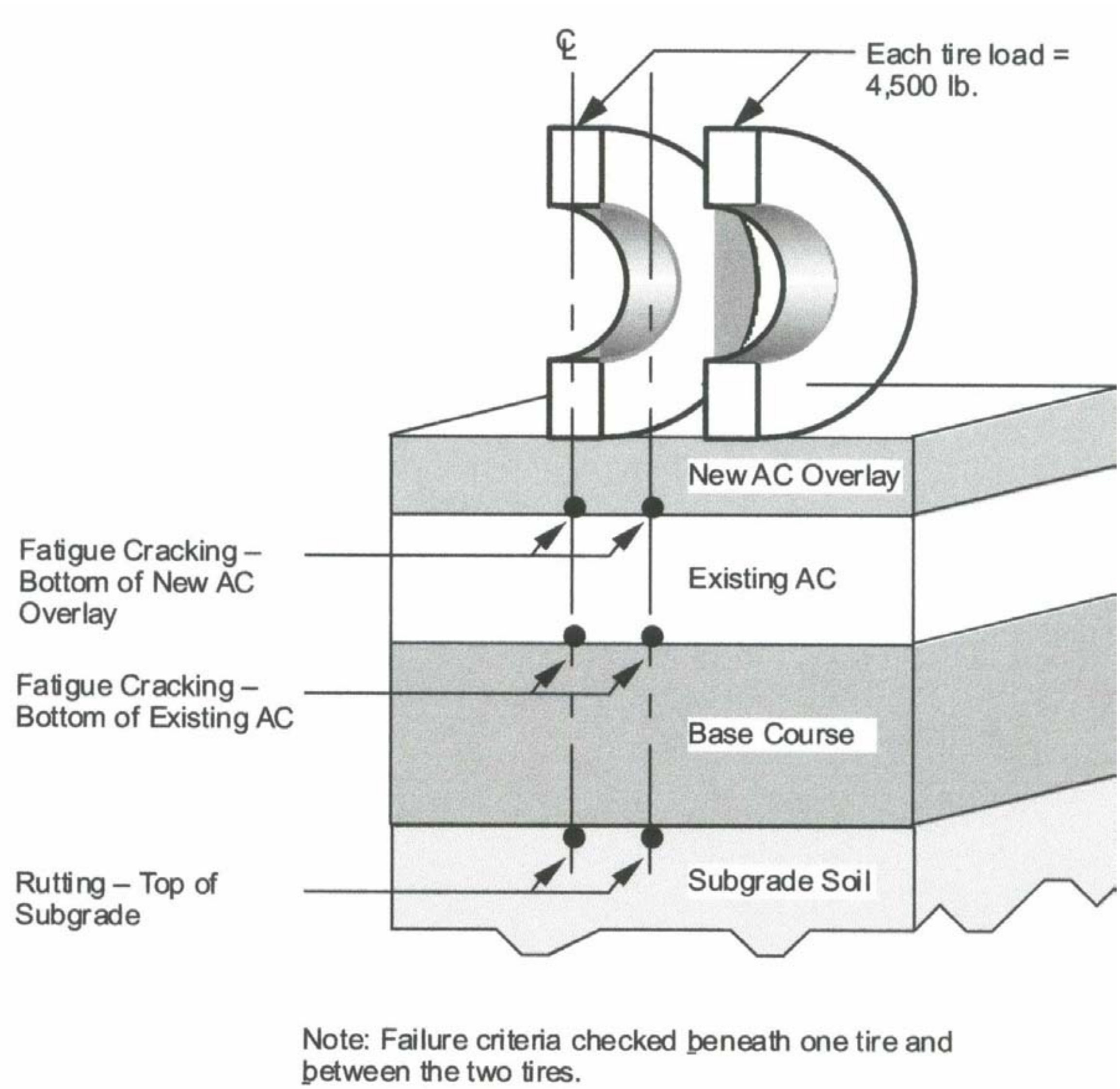

Figure B-5. Critical stress locations considered in Washington State DOT overlay design procedure.

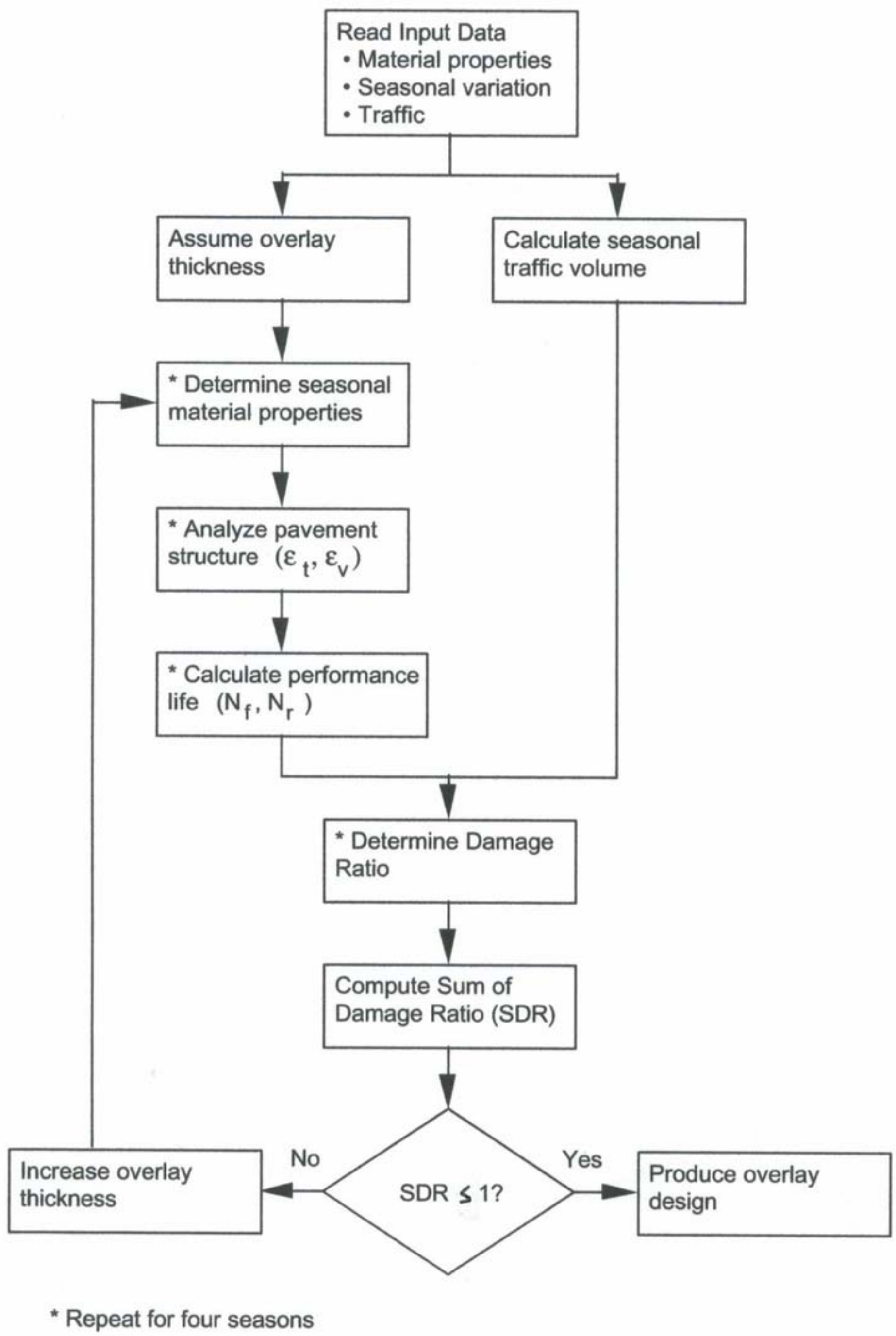

Figure B-6. Washington State DOT overlay design procedure flowchart.

The Nevada DOT uses a falling weight deflectometer to measure asphalt pavement deflections, and collects the surface and air temperature data needed to adjust the measured deflections. The MODULUS backcalculation program[33] is used to determine the elastic moduli of the pavement layers and the foundation. MODULUS is a database backcalculation program in which the deflection basin database is produced by a factorial of CHEVRON elastic layer program runs. In the Nevada DOT overlay design procedure, traffic loadings are expressed in ESALs applied during each season of the year. Overlay performance is predicted in terms of rutting and fatigue cracking. The design procedure yields overlay thicknesses in the range of 2 to 5.5 inches, for design periods from 5 to 20 years.

Construction

Asphalt Patching

High-severity alligator cracking and linear cracking should be repaired by full-depth asphalt patching prior to placement of an asphalt overlay. How much medium-severity cracking should also be repaired to achieve the desired performance depends on the thickness of the overlay, whether or not a reflection crack control treatment such as a fabric interlayer is used, and the unit costs of the preoverlay patching, asphalt overlay, and interlayer material.

Correcting Rutting

Rutting in the existing pavement may be removed by cold milling, filled with asphalt concrete compacted prior to placing the first overlay course, or filled with a leveling course. This last option might result in rutting occurring in the overlay at a faster rate than it would with the former two options.

Tack Coat

A tack coat is be applied to the existing or milled surface prior to the placement of the first leveling or overlay course, unless a reflection crack control treatment is used.

Reflection Crack Control

Cracking which is left unrepaired may reflect through the overlay. The rate at which reflection cracks develop and deteriorate depends on several factors, including the thickness of the overlay, the thickness of the existing pavement, the stiffness of the foundation, the number and magnitude of applied loads, daily and seasonal temperature variations, and the spacing of the unrepaired cracks. As reflection cracks deteriorate to medium and high severity levels, they reduce serviceability and increase maintenance needs. In addition to preoverly repair (patching and crack filling), the following reflection crack control treatment options are identified in the 1993 AASHTO Guide[1] for asphalt overlays of asphalt pavements:

- Synthetic fabrics and stress-absorbing interlayers,
- Crack relief layers greater than 3 inches thick,
- Sawing and sealing joints in the overlay at locations coinciding with straight (e.g., transverse thermal) cracks in the existing asphalt pavement,
- Increased overlay thickness.

Placing the Overlay

The asphalt overlay is placed and compacted in one or more lifts, in the same manner as for new asphalt pavement construction.

Performance

The performance of an asphalt overlay depends primarily on the thickness of the overlay, its asphalt concrete mix design, and the type and extent of preoverlay repair and surface preparation.

Thin asphalt overlays are not likely to perform well or be cost-effective for pavements with little or no remaining structural life. Thicker asphalt overlays will perform well, but not be cost-effective, for pavements which have considerable remaining structural life. One reason for this is that the costs of a thicker asphalt overlay include not only the costs of the overlay itself and preoverlay repairs, but also the costs of raising guardrails and signs, overlaying shoulders, and making other geometric adjustments associated with raising the pavement grade by three or more inches.

An important factor in the performance of asphalt overlays is the extent to which existing cracking is repaired prior to overlay. Asphalt pavements with extensive and severe cracking can be rehabilitated by a combination of patching and asphalt overlay, but at some point, other rehabilitation options which are not as sensitive to the preoverlay condition of the pavement (e.g., whitetopping, reconstruction) may be more cost-effective. At what point this is true for a given pavement depends on the extent and severity of cracking present, the thicknesses of the overlay and/or reconstruction options considered, and the unit costs of the preoverlay repair, overlay, and/or reconstruction options.

An asphalt overlay of 4 or more inches is estimated to have a service life of about 8 to 15 years on an asphalt pavement. Arizona estimates a service life of 10 years for asphalt overlays of asphalt pavements.[24] New York estimates a service life of 15 years for 3 or more inches of new asphalt without cold milling or hot surface recycling, and for 3 or more inches of new asphalt after cold milling to a depth of 1.5 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] Vermont estimates a service life of 6 to 12 years for 2 to 5 inches of asphalt overlay.[15] West Virginia estimates a service life of 8 years for asphalt overlays of asphalt pavements.[34] Wisconsin estimates a service life of 10 to 14 years for asphalt overlays of asphalt pavements.[35]

A thin asphalt overlay is estimated to have a service life in the range of 4 to 8 years. New York estimates a service life of 8 years for 1- to 1.5-inch asphalt overlays of asphalt pavements, and also for 1- to 1.5 inch replacement of asphalt concrete removed by cold milling. New York estimates a service life of 15 years for 1.5 inches of new asphalt after hot in-place recycling to a depth of 1.5 inches, or cold in-place recycling to a depth of 3 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks, except the option involving cold in-place recycling, which is only considered suitable for pavements with less than 4000 ADT per lane.[14] Vermont estimates a service life of 5 to 8 years for thin asphalt overlays.[15] Wisconsin estimates a service life of 6 to 9 years for thin asphalt overlays of asphalt pavements.[35] Indiana estimates a service life of 5 to 8 years for thin asphalt overlay of asphalt, with millling.[36]

Concrete Overlay of Asphalt Pavement

Appropriate Use

A concrete overlay, or whitetopping, of an asphalt pavement is usually done to increase structural capacity. Conventional concrete overlays are constructed with normal Portland cement concrete mixtures and paving methods. The structural design of conventional concrete overlays is comparable to that of new concrete pavements or unbonded concrete overlays. Recommended minimum thicknesses of conventional concrete overlays are 6 inches for Interstate and primary routes, 4 inches for jointed overlays on secondary routes, low-volume roads and parking areas, and 6 inches for continuously reinforced overlays on secondary routes, low-volume roads and parking areas.[37]

A conventional concrete overlay is appropriate for asphalt pavements with insufficient structural capacity for the traffic anticipated over the design life of the rehabilitation. A concrete overlay placed to correct a structural deficiency will also correct any functional deficiencies present.

As a general rule, an asphalt pavement is considered to require a structural improvement when 50 percent of the wheelpath area (equivalent to about 10 percent of the total area) of the outer traffic lane has medium- to high-severity alligator cracking.[23] A critical rutting level of one half inch is often cited as indicative of a need for structural improvement. However, rutting may have causes related not only to the load-bearing capacity of the pavement layers, but rather the stability of the mix, so the cause of rutting should be examined before deciding whether or not a structural improvement is the appropriate remedy.

Design and performance considerations are different for ultrathin whitetopping, which is constructed very short joint spacing, and sometimes with high-early-strength mixes and fast-track paving techniques. The design and construction methods for ultrathin whitetopping are relatively new and in many respects are still in development.

An ultrathin concrete overlay is appropriate for asphalt pavements with functional deficiencies only, such as excessive roughness, poor surface friction, excessive rutting, and distresses such as bleeding, weathering, ravelling, bumps, settlements, and heaves. Most ultrathin concrete overlays have been placed on deteriorated asphalt pavements on city streets, at intersections, at truck weigh stations, and at low- to medium-volume roads.[38] Other applications include general aviation airports and parking areas.[37]

Limitations

Inadequate vertical clearance at bridges is cited in the 1993 AASHTO Guide[1] as a potential limitation to the feasibilty of conventional concrete overlay as a rehabilitation option for asphalt pavements. The 1993 AASHTO Guide further states that vertical clearance problems may be addressed by reducing the overlay thickness under bridges (although this may result in early failure at these locations), by raising the bridges, or be reconstructing the pavement under the bridges. Thicker asphalt overlays may also necessitate raising signs and guardrails, as well as increasing side slopes and extending culverts. Sufficient right-of-way must be available or obtainable to permit these activities.[23]

According to the American Concrete Pavement Association, ultrathin concrete overlays are well suited for pavements with a substantial thickness of asphalt concrete present, such as full-depth asphalt pavements or pavements which have received multiple asphalt overlays.[37] An ultrathin concrete overlay is not recommended for an asphalt pavement with less than 2 inches of sound asphalt concrete present after millling.

Concurrent Work

A concrete overlay of an asphalt pavement may be done alone or in conjunction with full- and/or partial depth patching to repair cracks and potholes, and/or in combination with cold milling to remove rutting and enhance bond. The existing surface could conceivably be prepared by hot in-place recycling in place of cold milling, but this added expense is probably not justified.

Materials

Portland cement concrete mixes used for conventional concrete overlays are usually of the same mix design as those used for new concrete pavement construction. Mixes for ultrathin concrete overlays have a smaller maximum aggregate size, and may include fibers. Ultrathin concrete overlays have been constructed with normal-strength cement mixtures, and with fast-track paving mixtures which have a high cement content and/or high-early strength cement. More information on mix designs appropriate for ultrathin concrete overlays is available in American Concrete Pavement Association's technical bulletin on fast-track paving.[39]

Design

Conventional concrete overlays of asphalt pavements are typically designed as new pavements, with the existing pavement considered a high-strength, high-friction foundation. There are seven steps in the 1993 AASHTO procedure[1] for determining the required thickness for a conventional concrete overlay of an asphalt pavement:

1. Characterization of existing pavement design: specifically, the existing material types and layer thicknesses.

2. Traffic analysis: determination of the future 18-kip ESALs anticipated in the design lane over the analysis period.

3. Condition survey: heaves and swells, signs of stripping in the asphalt concrete, and large transverse cracks.

4. Deflection testing: recommended to determine the effective dynamic k value of the foundation. The method used is different than that for design of overlays of concrete pavements. It is recommended that the *in situ* subgrade resilient modulus be determined from distant sensor deflections, and that this modulus be used to estimate the effective dynamic k value using a nomograph given in Part II, Section 3.2 of the 1993 AASHTO Guide.[1]

5. Coring and materials testing: not required unless some condition such as asphalt stripping warrants investigation.

6. Determination of the required slab thickness for future traffic (D_f): done using the 1986 AASHTO model for design of new concrete pavement, with the design static k value estimated (a) as a function of the effective dynamic k value, determined as described in step 4, or (b) from subgrade soil type, base type, and base thickness information.

7. Determination of the required overlay thickness (D_{ol}): which is equal to the required slab thickness for future traffic, D_f, determined in step 6.

The American Concrete Pavement Association presents a different approach to determination of the design k value for conventional concrete overlay design. In this approach, a k-on-top-of-the-base is determined as a function of the static k value of the subgrade soil and the thicknesses of the asphalt concrete and treated or granular base layers. Nomographs are provided in the American Concrete Pavement Association's whitetopping engineering bulletin[37] for determination of this composite k value for (a) asphalt pavements on granular bases and (b) asphalt pavements on asphalt-treated or cement-treated bases. The ACPA bulletin also presents thickness charts for conventional concrete overlays for different ranges of truck traffic, and recommendations for dowel design and steel reinforcement design for conventional concrete overlays. Dowels are recommended for jointed plain concrete overlays 8 inches thick or more, and for jointed reinforced concrete overlays of any thickness.[37]

Ultrathin concrete overlay design should ideally be done by some mechanistic approach which takes into account the following factors: [40]

- The degree of bond between the concrete overlay and the asphalt layer,
- The quality of support provided by the asphalt concrete layer.
- The effects of overlay slab thickness and joint spacing on curling and warping stress in the overlay, and
- The fatigue performance of high-strength concrete mixtures, including those containing fibers.

Mechanistic design of ultrathin concrete overlays is still in the research and field testing stages. The American Concrete Pavement Association's engineering bulletin on whitetopping offers some tentative ultrathin concrete overlay thickness design charts, for different levels of truck traffic and subgrade support.[37]

Construction

Preoverlay Repair

Conventional concrete overlays of asphalt pavements require little or no preoverlay repair.[1,37] If the concrete overlay is to be continuously reinforced concrete, localized repair of alligator-cracked areas with exceptionally high deflections may be warranted. Potholes may repaired by full- or partial-depth patching, or simply filled with stone, cold mix, or hot mix.

Surface Preparation

The existing asphalt concrete surface may be prepared by sweeping, by cold milling, or by placing a leveling course. If cold milled, the surface should subsequently be swept and cleaned with high-pressure water to remove dust and debris.

Overlay Placement

When paving on hot, sunny days, it is sometimes considered advisable to cool the asphalt surface prior to placement of the concrete overlay. One technique used for this purpose in the past was whitewashing with a white-pigmented curing compound or lime slurry.[41] Whitewashing is recommended when the asphalt temperature is expected to exceed 110°F during paving. This is more likely to be a concern when the concrete overlay is to be placed on a new asphalt surface, i.e., a leveling course.

Another technique for cooling the asphalt surface is fogging with water. This technique is considered adequate when placing the concrete overlay on an old or milled asphalt surface. Water fogging is recommended when the asphalt surface is sufficiently hot that it is uncomfortable to touch with an open palm.[37]

The concrete overlay is placed either with fixed forms or a slipform paver. After placement on the prepared asphalt surface, the concrete is spread, consolidated, finished, textured, and cured. Materials commonly used for curing are white-pigmented liquid membrane curing compunds, waterproof paper or polyethylene sheets, and wet cotton mats or burlap.

Sawing Joints

Timing of the sawcutting operation is critical in concrete overlay construction, especially for ultrathin concrete overlays. For conventional concrete overlays, the timing of sawcutting is similar to that for new concrete construction on a high-friction base, depending also on the weather conditions and mix design. For ultrathin concrete overlays, the high heat of hydration and large surface area-to-volume ratio combine to heighten the risk of premature cracking if joints are not sawed in time.

Lightweight (Soff-Cut) saws may be used to saw the transverse and longitudinal joints early. Joint sawing should begin as soon as the partially hardened concrete can support the weight of the lightweight saw without causing damage to the surface or ravelling of the joint.[42] Secondary sawing for joint sealing purposes, if desired, may be done later using conventional saws.

For conventional concrete overlays, the ACPA recommends that the joint spacing in feet be 1.75 times the overlay slab thickness in inches. For example, the maximum joint spacing for a 6-inch-thick conventional concrete overlay would be 10.5 ft.

For ultrathin concrete overlays, the ACPA recommends that the joint spacing in inches be about 12 to 15 times the overlay slab thickness in inches. For example, the maximum joint spacing for a 3-inch-thick ultrathin concrete overlay would be 45 inches, or 3.75 feet

Performance

The typical range of service life for conventional concrete overlay of asphalt pavement is considered to be the same as for reconstruction in concrete, i.e., 20 to 30 years.

A service life range of 5 to 15 years is estimated for ultrathin concrete overlay of asphalt pavements. This is merely a tentative estimate, as there are no service life estimates to be found in the literature for this technique. Performance estimates for ultrathin concrete overlays tend to be confined to prediction of performance of one specific project at a time. For example, based on the performance of an ultrathin whitetopping project at a weigh station in Florida over its first year of service, the researchers monitoring it estimated that an ultrathin concrete overlay on a medium-volume road or intersection would have a service life of about 10 years.[42]

Full-Depth Repair of Concrete Pavement

Appropriate Use

Full-depth repair is localized repair of distresses which affect the full thickness of the slab. In short-jointed plain concrete pavements, slab replacement may cost less than repair of a portion of a slab. Many distresses may be corrected with full-depth repair:

- Corner breaks,
- Linear cracking,
- Punchouts,
- D-cracking,
- Pressure cracking due to alkali-aggregate reaction,
- Joint spalling,
- Blowups, and
- Bumps, settlements, and heaves.

Some distresses such as joint spalling, map cracking, and scaling may be repaired by either full-depth or partial-depth repair, depending on how much of the slab thickness is affected.

Limitations

The appropriateness of full-depth repair and other nonoverlay restoration techniques depends greatly on the amount of structural distress present. Pavements that have little remaining structural life are not good candidates for restoration. An overlay or reconstruction is usually a more cost-effective rehabilitation alternative in this situation.

Concurrent Work

Full-depth repairs may be done alone, as part of a comprehensive pavement restoration, or in preparation for resurfacing. Full-depth repair should be done after slab stabilization, after or concurrently with partial-depth repair, and before diamond grinding and joint resealing.

Materials

The selection of the full-depth repair material depends on many factors, including: the time available before opening to traffic, the air temperature during construction, the available funds, and the desired service life.

Rapid strength development is important when lane closure time must be minimized. The rate of strength development is influenced by the properties of the repair material as well as the ambient temperature during placement and the curing methods used. Both cement and epoxy materials will gain strength more slowly at low temperatures.

The cost associated with the use of any repair material depends not only on the material cost itself but also labor time for mixing and placement, equipment, curing requirements, and allowable lane closure time. These costs may be different for different repair materials.

Full-depth repairs may be constructed with normal paving concrete mixtures, or with high-early-strength mixtures, if quick opening to traffic is necessary. Normal-strength cement mixtures containing Type I cement can be used when the repairs can be protected from traffic for 24 hours or more. A set accelerator may be added to the mix to reduce the setting time. Normal-strength concrete repair mixtures should not be placed when the air temperature is below 40°F. At temperatures below 55°F, a longer curing period and/or insulation mats may be required.

High-early-strength cement mixtures, usually containing Type III cement with or without admixtures, can gain strengths in excess of 3000 psi within 24 hours. High-early-strength repairs are used when early opening to traffic is required. Specialty cement mixtures contain some kind of cement in place of or in addition to normal Type I or Type III cement. This may be some other hydraulic cement, a gypsum-based cement, magnesium phosphate cement, or mixtures with accelerative admixtures or additives.

Design

The major design issue related to full-depth repair of concrete pavement is determining the amount of repair to be done, especially when in conjunction with an asphalt overlay or bonded concrete. The greater the percentage of medium- and high-severity the thinner the overlay may be. However, high-quality full-depth concrete repair requires quite a bit of hand labor, so it might not necessarily be most cost-effective to repair 100 percent of the deteriorated joints and cracks. The optimal amount of patching depends on the following:

- The amount of medium- and high-severity linear crack deterioration, joint deterioration, and/or punchouts present;
- The required overlay thickness as a function of the amount of distress repaired;
- The unit cost of full-depth repair; and
- The unit cost of the overlay, as a function of thickness.

The condition-based 1993 AASHTO[1] overlay design procedure permits the designer to consider the effect of extent of patching on the required overlay thickness. In the 1993 AASHTO condition-based overlay design procedure, the adjustment factors assigned to the existing slab thickness may be selected to reflect the amount of medium- and high-severity transverse cracking, joint deterioration, and durability distress.

Construction

Selecting Repair Locations and Sizes

For the purpose of developing plans and bid documents, a field survey should be conducted to estimate the total area of full-depth repair required. The quantity of repair required may be greater than that originally estimated in the earlier pavement evaluation survey.

Full-depth repairs should be a full traffic lane wide and at least 6 ft long. The following additional recommendations apply to layout of full-depth repairs in jointed concrete pavements:[43,44]

- If the repair boundary of a 6-ft-minimum-length repair falls within 6 feet of an existing undoweled transverse joint that does not otherwise require repair, the repair should be extended to that transverse joint.
- If the repair boundary of a 6-ft-minimum-length repair falls at an existing doweled transverse joint that does not otherwise require repair, the repair should be extended 1 foot beyond that transverse joint, to permit removal and replacement of the dowels.

- If the repair boundary of a 6-ft-minimum-length repair falls at a crack in a continuously reinforced concrete pavement, the repair should be extended one half foot beyond that crack.
- When two 6-ft-minimum-length repairs are less than about 15 feet are apart, they be combined into one larger repair. However, the length of a single full-depth repair should not exceed the joint spacing of the pavement.

Illustrations of selection of repair areas for jointed plain, jointed reinforced, and continuously reinforced concrete pavements, are shown in Figures B-7, B-8, and B-9, respectively.

Concrete Removal

The transverse boundaries of repairs in jointed concrete pavements may be established by full-depth or partial-depth sawcutting. Full-depth sawcutting facilitates rapid removal of the concrete within the repair boundaries. However, full-depth sawcutting produces smooth vertical faces to the transverse repair joints, with no aggregate interlock.

Partial-depth sawcutting, followed by chipping of the concrete away from the repair boundaries, produces rougher vertical faces to the transverse repair joints. However, the breakup and removal process is slower when the transverse repair boundaries are only sawed partial depth.

For repairs of continuously reinforced concrete pavement, transverse repair boundaries must be sawed partial depth, and the remaining concrete around and below the existing reinforcing steel chipped out by hand. This makes full-depth repair a more time-consuming activity for continuously reinforced concrete pavements than for jointed concrete pavements, but is essential to the restoration of reinforcing steel continuity through the repair. Separate full-depth cuts may be made inside these partial-depth cuts, to facilitate concrete removal from the middle of the repair area. These full-depth cuts, if made, should be 24 inches inside of the partial-depth sawcuts, if tied splices are to be used, 4 to 8 inches if welded splices are to be used, and 2 to 4 inches and mechanical splices are to be used.[43,45]

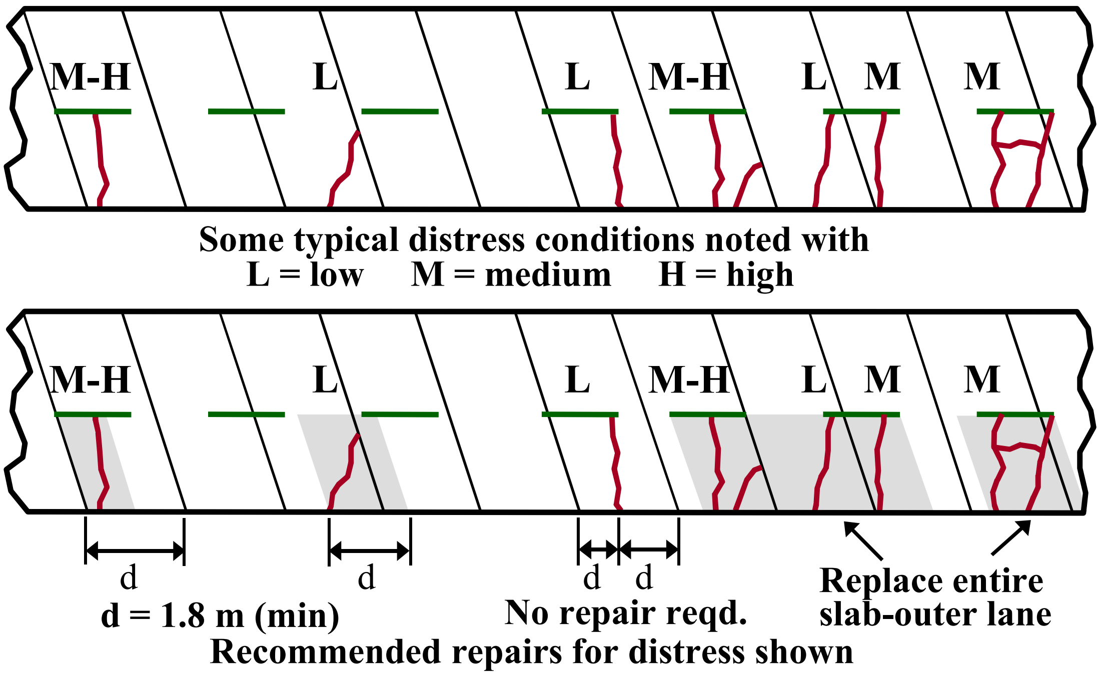

Figure B-7. Selection of full-depth repair boundaries for jointed plain concrete pavement.[4]

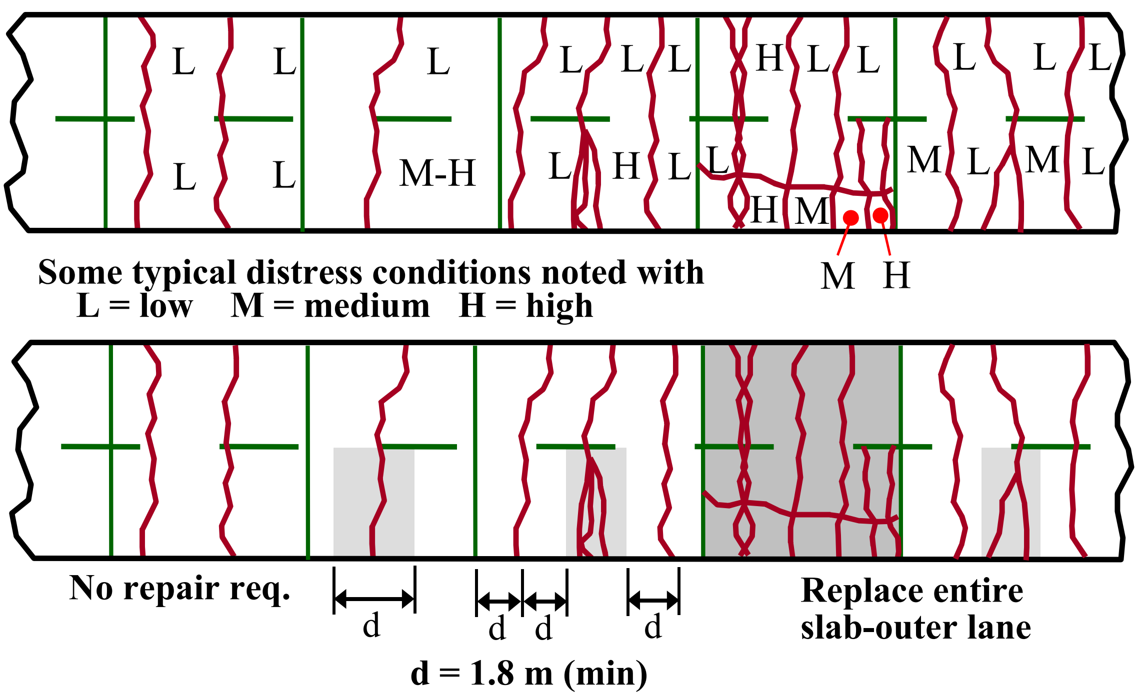

Figure B-8. Selection of full-depth repair boundaries for jointed reinforced concrete pavement.[4]

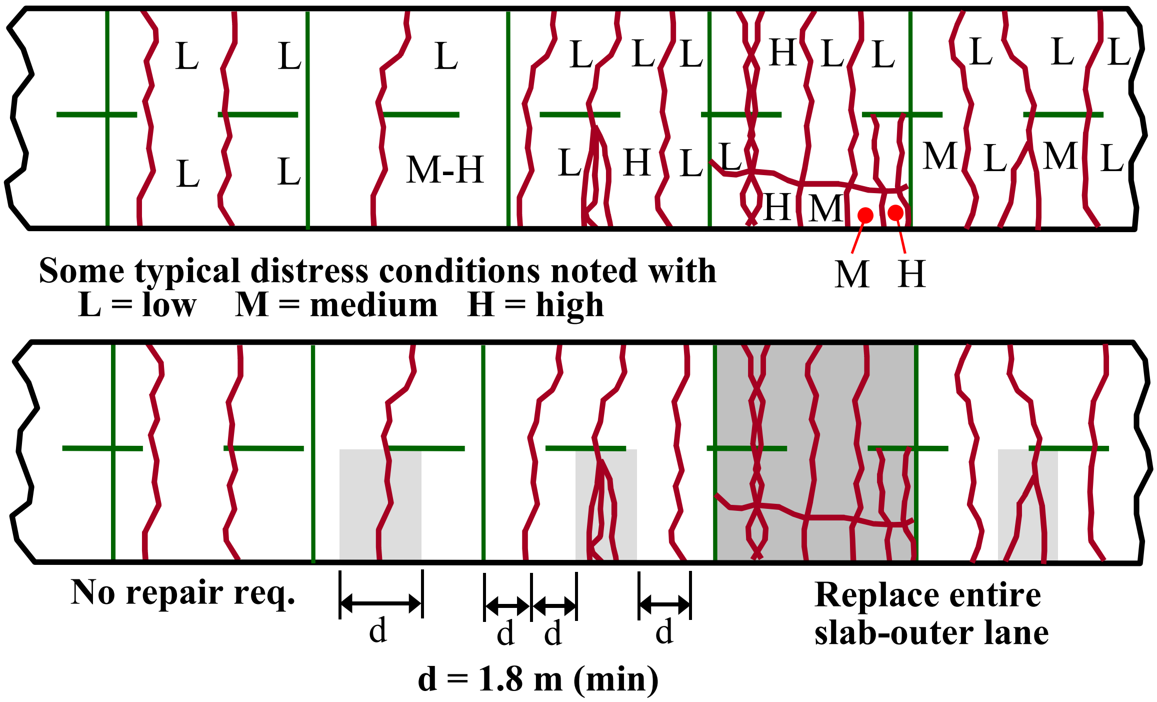

Figure B-9. Selection of full-depth repair boundaries for continuously reinforced concrete pavement.[4]

In all cases, the repair boundary along the longitudinal centerline joint should be sawed full depth to cut through any tie bars present. If the shoulder is tied concrete, these tie bars should be cut by full-depth sawing as well.

The concrete within the repair boundaries may be removed either by liftout or breakup. The liftout method is preferred, if the concrete is sufficiently sound to permit it, because it causes less disturbance to the base material.

Repair Area Preparation

The base material within the repair area should be recompacted, with new granular material added if necessary.

Dowel Installation for Full-Depth Repairs in Jointed Concrete Pavements

Full-depth repairs in jointed concrete pavements should be dowelled at the transverse joints to the adjacent slabs. The transverse joint load transfer system (size, number, and layout of the dowel bars) should be selected considering the truck traffic using the pavement. In general, 1.5-inch-diameter dowels are considered appropriate for Interstate-type truck traffic. 1.25-inch-diameter dowels may be sufficient for low traffic levels, as long the pavement does not have a weak foundation and poor drainage conditions. One-inch-diameter dowel bars have generally not performed well in full-depth repairs.[46,47]

Four dowels per wheelpath is recommended for most support and traffic conditions. Five dowels per wheelpath are recommended for heavy traffic conditions, especially in combination with poor support conditions.

Most agencies specify the use of 18-inch-long dowels, which provide 9 inches of embedment in the existing concrete and 9 inches in the repair. However, in laboratory testing of dowel load transfer, no difference in dowel performance was found between 14-inch-long dowels (7-inch embedment length) and 18-inch-long dowels.[47]

Holes for the dowels may be drilled into the existing slabs on either side of the repair area using individual drills or gang drill rigs. The latter provide better control of the alignment of the dowel holes. Either pneumatic or hydraulic percussion drills may be used for drilling dowel holes. Pneumatic drills may cause more spalling at the joint face, but this does not appear to result in any difference in dowel performance.[47] The dowel hole diameter needed depends on the diameter of the dowel and the type of backfill material to be used. Cement grout requires a hole diameter about 0.25 inch greater than the dowel diameter. Epoxy grout requires a hole diameter only about a sixteenth of an inch greater than the dowel diameter.

After drilling, the dowel holes should be cleaned out by air blasting. A long nozzle should be used to insert the backfill material starting at the back of the hole. The dowel should then be inserted into the hole and rotated about one full turn to distribute the backfill material around the dowel. Many agencies put reusable plastic disks on the dowels, as illustrated in Figure B-10, to prevent any backfill material from flowing out of the dowel holes.

Reinforcing of Full-Depth Repairs in Continuously Reinforced Concrete Pavements

Full-depth repairs in continuously reinforced concrete pavements must be continuously reinforced as well, with the steel connected at the transverse joints to the steel in the adjacent slabs. The connection may be accomplished by tying, welding, or using mechanical connectors. Mechanical connectors require a lap length of 1 to 2 inches between the exposed steel in the existing slab and the steel in the repair, on each side of the repair.[43] Welded connections require a lap length of 4 inches on each side the repair.

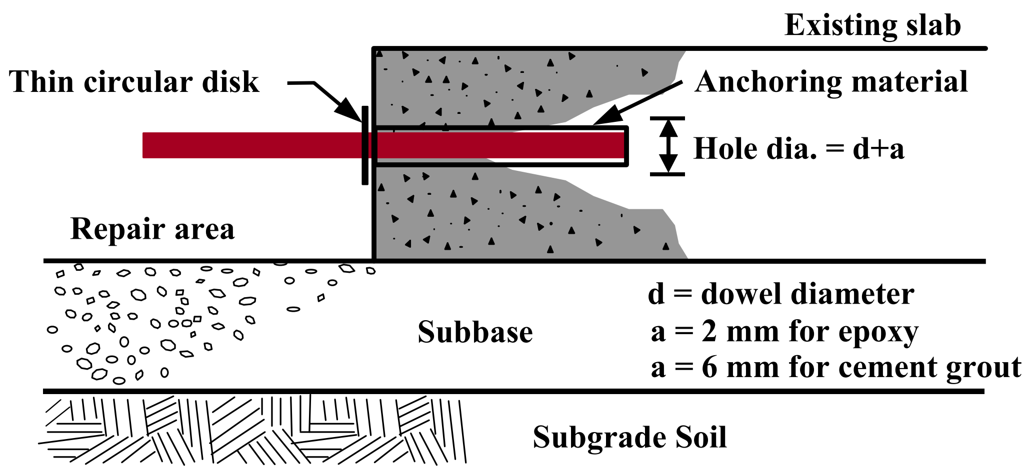

Figure B-10. Diagram of dowel installation for full-depth repair.

The reinforcing steel within the repair should be supported on chairs. To avoid buckling of the steel, if mechanical or welded connections are used, separate pieces of steel should be connected to each piece of exposed steel on each side of the repair, and these pairs of separate pieces tied together in the middle of the slab.

Longitudinal Joint Preparation

Full-depth repairs in both jointed and continuously reinforced concrete pavements should be separated from the adjacent lane slab by a bondbreaker along the longitudinal joint. Tying or doweling the repair across the longitudinal joint will heigten the risk of premature cracking of the repair. The only exception to the rule of separating the repair from the adjacent longitudinal lane is in the case of full slab replacement, at least 15 feet long. In this case, tie bars should be drilled into the adjacent slab.

Repair Material Placement

After preparation of the base, dowels, and reinforcement, the concrete repair material should be placed in the repair area. The repair material should be carefully vibrated around the dowels and reinforcement, to attain good consolidation without segregation and loss of air entrainment. After being vibrated into place, the repair material should be struck off, finished, textured, and cured. Materials commonly used for curing include white-pigmented liquid membrane curing compound, waterproof paper or polyethylene sheets, wet burlap, and insulation mats.

Full-depth repairs in asphalt-overlaid concrete pavements should be constructed of concrete capped with asphalt. The concrete portion of the repair should be dowelled or tied as appropriate for the type of concrete pavement present.

Joint Sawing and Sealing

Once the concrete has gained sufficient strength, the transverse repair boundaries may be sawcut to form reservoirs for joint sealing. It is generally believed that sealing full-depth repair joints will reduce spalling and water infiltration. However, it has not been demonstrated that full-depth repairs with unsealed joints perform any differently than repairs with sealed joints.

Opening to Traffic

The timing of opening full-depth repairs to traffic may be controlled by specifying a minimum compressive or flexural strength, or be specifying a minimum time after placement. The size of the repair, slab thickness, concrete mix design, and ambient conditions influence the time needed for the repair to reach a specified minimum strength. Recommended minimum strengths are 2000 psi in compression, 300 psi in center-point flexural loading, and/or 250 psi in third-point flexural loading.[46] Additional recommendations for minimum strengths for opening full-depth repairs to traffic are given in the ACPA's technical bulletin on full-depth repair.[43]

Performance

The key factors influencing the performance of concrete pavement full-depth repair are the appropriateness of its use, the quality of construction, and the properties of the repair material. Full-depth repair may perform well but not be cost-effective for a pavement which has little or no remaining structural life and will soon need resurfacing or reconstruction.

Concrete pavement restoration is estimated to have a service life in the range of 5 to 15 years, depending on the combination of techniques used, and the extent, materials, and construction quality of each. The American Concrete Pavement Association estimates a service life of 10 to 15 years for full-depth repair on its own.[6]

Partial-depth repair of Concrete Pavement

Appropriate Use

Partial-depth repair is localized repair of distresses which are confined to the upper third of the slab, such as spalling at joints and crack. Partial-depth repair is used to improve pavement rideability, deter further deterioration, and provide suitable edges for effective joint and crack resealing.

Partial-depth repair is appropriate for concrete pavements which have spalling at joints and/or cracks. Spalls of low severity do not affect ride quality and may not require repair. Individual small spalls adjacent to joints or cracks can simply be filled with sealant. However, when several small spalls exist along a joint or crack, it may be preferable to repair the full length of the spalled area.

Limitations

Partial-depth repair is not appropriate for concrete damage caused by material problems such as D-cracking or alkali-aggregate reactivity, or by corrosion or lockup of dowel bars at transverse joints. Such damage is typically more extensive than is evident from visual inspection of the slab surface. Full-depth repair is more appropriate for these distresses. Coring at selected locations may be necessary to determine the depth of spalling before choosing between partial-and full-depth repair.

A partial-depth repair cannot correct a crack through the full thickness of the slab. A full-depth repair is the appropriate repair for cracking. However, surficial spalling at a crack can be corrected by partial-depth repair, as long as the crack itself is reestablished through the partial-depth repair and subsequently sealed.

Partial-depth repairs should not be used to repair spalling caused by corrosion of metal joint-forming inserts or reinforcing steel if the metal is to be left in place. All exposed metal must be removed before placing a partial-depth repair. In general, partial-depth repairs should be applied only to spalls which are confined to the upper third of the slab thickness and do not expose reinforcing steel or load transfer devices.

Pavements that have little remaining structural life, as evidenced by a substantial amount of fatigue cracking and/or rapid crack deterioration, are not good candidates for restoration. An overlay or reconstruction is usually a more cost-effective rehabilitation alternative in this situation.

Concurrent Work

Partial-depth repair may be done alone or as part of a comprehensive concrete pavement restoration strategy. Partial-depth repair should be done after slab stabilization, before or concurrently with full-depth repair, and before diamond grinding and joint resealing. One of the objectives of partial-depth repair is to provide new joint edges suitable for resealing.

Materials

Material Selection

The selection of the partial-depth repair material depends on many factors, including: the time available before opening to traffic, the air temperature during construction, the available funds, the desired service life, and the size and the depth of the repairs. The ideal partial-depth repair material will have good workability, quick mixing time, minimal volume change, fast setting time, rapid strength development, good long-term strength and durability, thermal compatibility with the existing concrete, and reasonable cost. Among the material properties which should be considered are compressive strength, modulus of elasticity, bond strength, freeze-thaw resistance, scaling resistance, abrasion resistance, coefficient of thermal expansion, and shrinkage. Several organizations have developed guide specifications for partial-depth repair materials.[48, 49, 50, 51]

Volume change in partial-depth repair materials occurs due to moisture loss and to temperature changes. Excessive shrinkage can cause the repair material to debond from the surrounding concrete, and may cause cracking within the repair material itself. Some repair materials can be mixed with agents which will decrease the likelihood of debonding and shrinkage cracking. Among the factors which influence the degree of volume change that occurs are the size of the aggregate used, the water/cement ratio (for cementitious mixtures), and retention of heat and moisture during curing.

Rapid strength development is important when lane closure time must be minimized. The rate of strength development is influenced by the properties of the repair material as well as the ambient temperature during placement and the curing methods used. Both cement and epoxy materials will gain strength more slowly at low temperatures.

The cost associated with the use of any repair material depends not only on the material cost itself but also labor time for mixing and placement, equipment, curing requirements, and allowable lane closure time. These costs may be different for different repair materials.

Cementitious Repair Materials

Normal-strength cement mixtures containing Type I cement can be used when the repairs can be protected from traffic for 24 hours or more. A set accelerator may be added to the mix to reduce the setting time. The aggregate used in the repairs should have a maximum size no greater than one half the minimum repair depth.

Cementitious repairs are usually bonded to the existing concrete with a grout, which consists of sand and cement in a 1:1 ratio by volume, and enough water to produce a creamy consistency. The repair material must be placed before the grout dries. If the grout is left exposed long enough to dry, the repair area must be cleaned again by sandblasting and the grout reapplied.

Normal-strength concrete repair mixtures should not be placed when the air temperature is below 40°F. At temperatures below 55°F, a longer curing period and/or insulation mats may be required.

High-early-strength cement mixtures, usually containing Type III cement with or without admixtures, can gain strengths in excess of 3000 psi within 24 hours. High-early-strength repairs are used when early opening to traffic is required. An epoxy bonding agent is usually used with these mixtures. The repair material is placed when the epoxy becomes tacky.

Specialty cement mixtures contain some kind of cement in place of or in addition to normal Type I or Type III cement. This may be some other hydraulic cement, a gypsum-based cement, magnesium phosphate cement, or mixtures with accelerative admixtures or additives.[52]

- **Gypsum-based (calcium sulfate) cement mixtures** can gain strength rapidly and can be used in temperatures above freezing and up to 109°F (43ºC). Some evidence suggests that these materials do not perform well when exposed to moisture and freezing temperatures.[53]

- **Magnesium phosphate cement mixtures** set very rapidly. Therefore, they should be mixed in small quantities and worked rapidly. At temperatures below 80°F, the working time is about 10 minutes. At higher temperatures the working time may be greatly reduced. These mixtures have very high early strength and low permeability. They bond well to any clean and dry surface. However, the strength of these materials is sensitive to the moisture content of the existing concrete. Furthermore, they cannot be used with pavements which contain limestone aggregates. The presence of limestone aggregates may be detected by wetting a freshly exposed concrete surface with vinegar. Bubbles will appear if the concrete contains limestone aggregates.[48,54]

- **Mixtures with accelerating admixtures or additives**. Alumina powder has been used as an admixture with Type I or Type III cement mixtures to counteract shrinkage. However, the reactivity of aluminum powder can be difficult to control in field proportioning, particularly in small batch operations. An alternative is a shrinkage-compensating cement (ASTM C 150, Type K).[55] High-alumina cement, on the other hand, is not recommended, as it is susceptible to a conversion of some of its calcium aluminate hydrate components, which may result in significant strength loss.

Specialty Repair Materials

Rapid-strength proprietary materials must be placed according to the manufacturer's recommendations for bonding, placing, curing, and opening time. Preparation of the repair area should also be done according to the manufacturer's recommendations if any. It is very important to follow the manufacturer's recommendations concerning suitable temperature ranges for placement. Some proprietary materials are very sensitive to temperature and construction procedures.[52]

Polymer concretes are a combination of polymer resin, aggregate, and a set initiator. The aggregate can range in size from sand to 3/8 in stone. Polymer concrete are categorized by the type of resin used, such as epoxies, methacrylates, and polyurethane.[48,54]

- **Epoxy resin mortars or epoxy concretes** have been used since the 1950s. In general, they have excellent adhesive properties and low permeability. However, the setting times, placement temperature ranges, strengths, bonding capabilities, and abrasion resistance properties of epoxy mixtures can vary widely. The particular epoxy mix under consideration should be carefully evaluated in the laboratory before use.

The main disadvantage of epoxy concretes is that they are not thermally compatible with normal concrete. This can sometimes result in early repair failure. The use of larger aggregate increases the volume stability and reduces the risk of debonding. Epoxy concretes should not be used to repair spalls caused by reinforcing steel corrosion because the epoxy can accelerate the corrosion in the steel.[48,54]

The epoxy resin catalyst should be preconditioned before blending. The epoxy components should be mixed in strict compliance with the manufacturer's recommendations before aggregate is added. The material should be blended in a suitable mixer until homogeneous. To avoid waste, only the amount of material that is usable in one hour should be mixed in each batch. If the blended material begins to develop excessive heat, the material should be discarded. Depending on the manufacturer's recommendations, a priming coat of blended epoxy may be required.

- **Methyl methacrylate concretes** have working times of 30 to 60 minutes, have high compressive strengths, and adhere well to clean dry concrete. They can be placed over a wide range of temperatures, from 45 to 130°F. A major concern with methyl methacrylates is their volatility and hazardous nature. The fumes pose a health hazard and can combust if exposed to a spark or flame. High-molecular-weight methacrylate (HMWM) is a new type of methacrylate that possesses many of the same properties as conventional methacrylate but without the volatility or health hazard.[48,54]

- **Polyester-styrene concretes** are similar to methyl methacrylate concretes, possessing many of the same properties, but have a much slower rate of strength gain. This limits their usefulness for partial-depth spall repair.

- **Polyurethane concretes** consist of a two-part polyurethane resin mixed with aggregate. These materials set very rapidly. Two types of polyurethane materials are currently available. The older type is moisture sensitive and foams when it comes into contact with water. The newer ones are claimed to be moisture resistant and suitable for placing on wet surfaces.

- **Other polymeric materials** that have been used in the past or are under development include acrylic concrete and furfuryl-alcohol-polymer concrete. Acrylic concrete has good bond strength, but requires dry aggregate, and can pose environmental and health hazards. Furfuryl-alcohol-polymer concrete was developed for rapid repair of bomb-damaged runways. It develops high early strength, and can be placed in wet conditions and at temperatures between 0 and 125°F. How this material would perform in highway repairs is not known.[54]

Bituminous Materials

Asphalt concrete is sometimes used for partial-depth spall repairs of concrete pavements. However, they are generally considered temporary repairs, when used on concrete pavements that are not to be overlaid. Bituminous materials are more often used to patch spalled joints and cracks in a concrete pavement prior to placement of an asphalt concrete overlay.

Construction

Preliminary Quantity Estimation

For the purpose of developing plans and bid documents, a preliminary field survey should be conducted to estimate the total area of partial-depth repairs required. By the time the repair work begins, the extent of the deterioration may be greater than that shown on the plans, depending on how much time passes between the field survey and the start of construction. The quantity of repair required may be greater than that originally estimated in the preliminary survey.

Marking Repair Boundaries

Just prior to commencement of work, a preconstruction survey should be conducted to mark the repair boundaries. During the preconstruction survey, all areas of delamination should be identified by means of sounding, i.e., striking the concrete surface with a carpenter's hammer or steel rod, or dragging a chain along the surface. A sharp metallic ring will be produced in areas where the concrete is sound, whereas a dull or hollow sound will be produced in areas where the concrete is delaminated.[4,46,48,56]

To ensure that all unsound concrete is removed, the limits of the partial-depth repair should extend 4 in beyond the delaminated or spalled area.[4,46] Repair boundaries should be straight, and sharp angles should be avoided because they may cause cracks in the repair material. Areas less than 2 ft apart should be combined into one repair area. Although this increases the quantity of repair material used, it expedites the construction process and also improves the overall appearance of the partial-depth repair project.

Concrete Removal

Spalled or delaminated concrete can be removed within the repair boundaries by sawing and chipping, chipping alone, or milling, each of which is described below.

Sawing and chipping. To remove spalled or delaminated concrete by sawing and chipping, saw cuts are made around the perimeter of the repair area. This provides vertical faces at the repair edges and sufficient depth to prevent spalling of the repair material along the repair perimeter. The saw cuts should be at least 1.5 in deep.[4,46,56,57] Additional sawcuts are often made within the repair area to facilitate chipping out the concrete.

The repair area should be chipped out to a depth of at least 1.5 in with light pneumatic tools, until clean and sound concrete is exposed. A small cold milling machine may be used for large patch areas, but removal to the boundary saw cut would still require using light hand tools. Hammers and other mechanical chipping tools should be operated at an angle less than 45° from the vertical, and fitted with spade bits, as gouge bits can damage sound concrete.

The best results are achieved using 10- to 15-pound hammers. It is harder to control the depth of chipping with heavier hammers. A pneumatic hammer which is too heavy will cause damage in the concrete below the depth actually needed to expose sound material.[4,46,56] Jackhammers heavier than 30 pounds should not be used, because they may break through the slab completely. They may also cause microcracking which can weaken the bond between the existing concrete and the repair material.

Chipping. Spalled or delaminated concrete may also be removed by chipping without first sawing the patch boundaries. The area to be repaired is first chipped out in the center using a light jackhammer, proceeding outwards, and switching to hand tools near the repair boundaries. The chisel point of the jackhammer should be directed inward toward the center of the repair area.[49]

Milling. Removal of spalled or delaminated concrete by cold milling is efficient for transverse joins which need partial-depth spall repair across most or all of the lane width. The milling machine must be equipped with a device for stopping at a preset depth to prevent excessive concrete removal and possible damage to dowel bars or reinforcing steel. After milling, the bottom of the repair area should be checked by sounding to ensure that all unsound material has been removed. Any unsound material must be chipped out. If the depth of unsound concrete exceeds one third of the slab thickness, consideration should be given to full-depth repair of the joint.

Milling may be done either longitudinally across the joint or transversely along the joint. For small individual spalls, milling in either direction is effective. Transverse milling produces more vertical boundaries, but can be a less efficient operation and can potentially interfere with traffic in the adjacent lane.

Cleaning

The exposed faces of the concrete within the repair area should be thoroughly cleaned by sandblasting to remove loose particles, oil, dust, and joint sealant materials. These and any other contaminants will interfere with bonding between the repair material and the existing concrete. The rough texture produced by sandblasting also enhances bonding.

High-pressure waterblasting is an alternative to sandblasting in urban areas where controlling dust is important. Waterblasting equipment for concrete should be capable of producing a jet pressure of 3000 to 6000 psi. However, to avoid damage to sound concrete, the jet pressure must be adjustable.[52]

All residue from sandblasting should be removed by airblasting just prior to placement of the bonding agent. The airblasting equipment should first be checked for oil, because oil sprayed onto the concrete will impede bonding of the repair material to the concrete. The air can be checked by placing a dry cloth over the nozzle and examining the cloth after releasing a small amount of air.

Joint Preparation

If the repair will cross or abut a transverse or longitudinal joint or crack, the joint or crack must be reestablished through the repair, using a compressible insert. The insert keeps the adjacent concrete from bearing directly on the repair material. The insert should extend at least 1 inch deeper and wider than the repair area. A sawcut to a depth of about 1.5 in below the bottom of the repair area facilitates placement of the insert.

A pliable joint insert is needed if the repair abuts a crack. A compressible insert, thin polyethylene strip, or piece of asphalt-impregnated roofing felt should be placed in the crack to confine the repair material. When the repair material has hardened, the insert should be removed and the joint or crack sealed. In the case of a transverse joint this will often require sawing a new joint sealant reservoir along the entire joint before new sealant is placed.

A partial-depth repair which abuts a lane edge adjacent to a shoulder should be confined so that the repair material cannot flow into the lane/shoulder joint or into the shoulder. If the repair material is allowed to penetrate into the shoulder, damage to the repair or the shoulder may result. If the shoulder is portland cement concrete, a compressible insert should be placed in the lane/shoulder joint, and the lane/shoulder joint should be resealed after the repair has hardened and the insert has been removed. If the shoulder is asphalt concrete, the repair material should be confined to the repair area by inserting a thin piece of plywood or other insert in the lane/shoulder joint. This may require removing a small portion of shoulder material. The lane/shoulder joint insert is removed when the repair material has hardened, and the shoulder is then patched.

Material Placement

The repair material should be placed as quickly as possible after preparing the repair area, while the exposed concrete is clean and dry. When a bonding agent or cementitious grout is used, it should be applied in a thin even coat. The best results are obtained when the material is scrubbed into the surface with a stiff bristle brush. The grout should cover the entire repair area including the walls, and should overlap the pavement surface to ensure adequate at the perimeter. Cementitious grouts must not be allowed to dry before the repair material is placed.

Some partial-depth repair materials require epoxy or proprietary bonding agents. Epoxy bonding agents should be mixed carefully according to the manufacturer’s instructions. Bonding agents and grouts should be mixed on site in small quantities. Partial-depth repair materials are also mixed on site in small mobile drum or paddle mixers. On-site mixing minimizes material waste.

The repair area should be slightly overfilled with repair material to allow for some decrease in volume after consolidation. Once the repair material is placed, it should be consolidated with a small spud vibrator to remove entrapped air and eliminate any voids between the repair material and the existing concrete.

The vibrator should be inserted vertically into the repair material. The vibrator should not be dragged through the repair material, nor should it be used to move the repair material around, as these actions may cause segregation of the mix and loss of entrained air. On very small repairs, adequate consolidation can be achieved using hand tools.

Finishing and Texturing

The repair material should be finished flush with the surface of the existing concrete. Finishing from the center of the repair toward the edges is recommended in order to press the repair material into contact with the repair faces.

The sawcut runouts extending beyond the repair boundaries can be filled with excess mortar from finishing. Any repair edges that do not abut joints or cracks to be resealed should also be sealed with a grout made up of equal parts cement and water. These actions will help in preventing water infiltration.

The repair material should be textured in a manner similar to that of the surrounding concrete. However, because of the small size of partial-depth repairs, their surface texture will not have any significant effect on the overall friction characteristics of the pavement surface. Burlap drag and transverse tine surfaces are common.

Curing

Partial-depth repairs must be adequately cured because their large ratio of surface area to volume makes them susceptible to considerable heat and moisture loss. Cementitious repair materials may be cured using a curing compound that meets ASTM C 309 material requirements. The compound impedes mix water evaporation and contributes to thorough cement hydration. Some agencies specify a white-pigmented compound (Type 2, Class A) that is easy to see after application. Other agencies specify a resin-based curing compound that meets ASTM C 309, Type 2, Class B requirements and may not contain a white pigment, but can produce a more effective evaporation barrier. An application rate of about 200 ft^2/gal is sufficient for either material. When specialty repair materials are used, curing should be done according to the manufacturer's recommendations.

When constructing partial-depth repairs in cool weather, and especially when early opening to traffic is required, insulation mats may be used to hold in heat and accelerate strength gain. To prevent moisture loss and to protect the surface, a layer of polyethylene sheeting should be placed between the repair surface and the insulation mats. Insulation mats should not be used in hot weather, because if too much heat is retained in the concrete, shrinkage cracking may occur in the repair material when the insulation is removed.

Joint Sealing

After the partial-depth repairs have gained sufficient strength, the joints should be resealed.

Opening to Traffic

There are two approaches to determining when to open partial-depth repairs to traffic: specified minimum strength, or specified minimum time after placement.[58] For most concrete pavement applications, opening based on strength is preferable. This is not always true, however, for concrete repairs, particularly in cases when quick opening is not critical. Most repair mixtures fall into one of three categories for opening to traffic: 4 to 6 hours, 12 to 24 hours, and the conventional 24 to 72 hours. Contractors often use conventional mixtures in repairs on large projects, in low-traffic areas or in other situations where quick opening is not necessary. In these situations, it is reasonable to specify a minimum time after placement for opening to traffic.[58] For 4-to-6-hour mixtures and 12-to-24-hour mixtures, strength testing using portable cylinder test devices, maturity meters, or pulse velocity devices is preferable to specifying a minimum time requirement.[58]

Performance

The key factors influencing the performance of partial-depth repairs are the appropriateness of their use, the quality of construction, and the properties of the repair material. Partial-depth repairs may perform well but not be cost-effective for a pavement which has little or no remaining structural life and will soon need resurfacing or reconstruction.

Performance reviews of various partial-depth spall repair materials typically have compared the performance of conventional Type I or III materials with proprietary materials. In most cases the conventional concrete mixtures have performed as well as or better than most of the proprietary mixtures and specialty blends.[54]

The American Concrete Pavement Association estimates the service life of partial-depth repair to be in the range of 10 to 15 years.[6] New York estimates a service life of 10 years for partial-depth repair, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] Utah estimates a service life of 10 years for partial-depth repair.[59]

Undersealing

Appropriate Use

Undersealing, also called subsealing or slab stabilization, is the filling of localized voids under slab corners by injection of a filler material, in fluid state, through holes drilled through the slab. The most commonly used filler is cement grout, although asphalt cement has been used as well. The purpose of undersealing is to reduce corner deflections and thereby reduce stresses in the slab, and possibly reduce faulting development as well.

Undersealing differs from slabjacking, which is done with the same equipment and materials, in that undersealing fills voids without raising the slab, whereas slabjacking raises slabs.

Limitations

Undersealing should only be done at slab corners which have voids; the practice of "blanket undersealing" of all corners, whether they have voids or not, can disrupt the uniformity of support and increase stresses in the concrete slab.

Undersealing is not presumed to increase structural capacity, nor stop erosion or faulting. It is only presumed to improve the uniformity of support to the concrete slab and reduce deflections at joints and cracks which have subsurface voids.

According to the American Concrete Pavement Association, some contractors suggest that a minimum project length of 2 miles is necessary to accommodate the different types of equipment needed to conduct an efficient undersealing operation.[60,61]

Concurrent Work

Undersealing is almost always done in conjunction with one or more other rehabilitation techniques. When done as part of a comprehensive concrete pavement restoration effort, undersealing should be done after full-depth repair but before all other techniques (e.g., partial-depth repair, load transfer restoration, diamond grinding, joint resealing).

Materials

The materials most commonly used for undersealing are pozzolan-cement grout and polyurethane.[62] Other materials that have been used include asphalt cement and grout mixtures of cement alone, cement with limestone dust, and cement with sand. Type I, Type II, and Type III cements have been used in grout mixtures for undersealing.

Pozzolan-cement grout is felt to be the best option among the cementitous grouts because the fineness and shape of pozzolanic particles (e.g., fly ash, illustrated in Figure B-11) greatly enhance the fluidity, and because the grout's stiffness is increased by reaction of the pozzolan with lime produced in the hydration of the cement.[60] Both Type C and Type F flyashes are suitable for use in slab undersealing. Typical specifications for pozzolan-cement grout call for a 7-day compressive strength of 600 to 800 psi and a flow-cone test time of 10 to 16 seconds.[62]

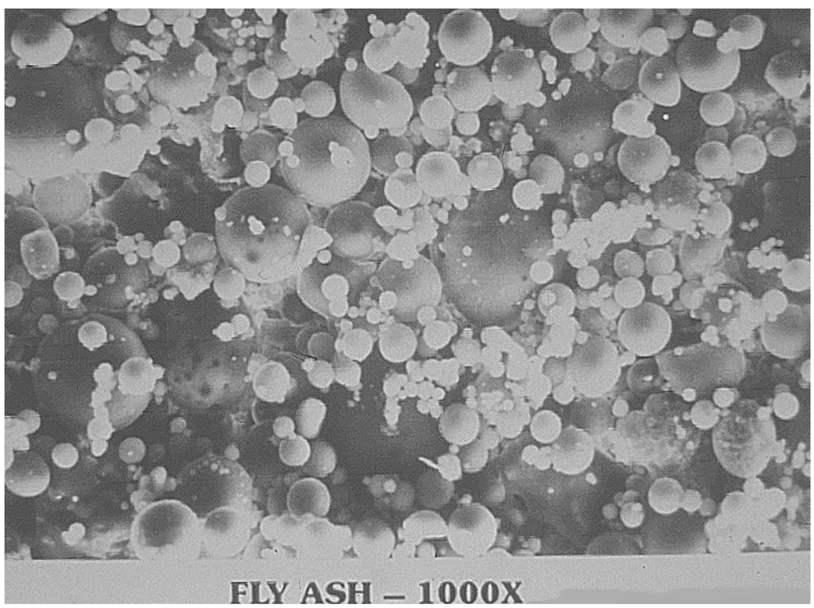

Figure B-11. Magnification of fly ash particles.[4]

Polyurethanes used for slab stabilization are two-part mixtures which react to form a strong, lightweight, foamy material which expands to fill voids under the slabs. Polyurethanes do not flow nearly as well as pozzolan-cement grouts, but they gain strength very quickly. After undersealing with polyurethane, an agency can let traffic back on the pavement in as little as a half hour.[62]

Construction

Void Detection

Four different approaches are available for detecting voids under concrete slabs: visual inspection, nondestructive deflection testing, ground-penetrating radar, and epoxy-core testing.

The visual inspection approach relies on observations of distresses such as corner breaks, joint faulting, pumping, blowholes, and lane/shoulder joint dropoff. This method has very limited accuracy in estimating the locations and sizes of voids.

The traditional method of detecting voids with nondestructive deflection testing method of testing for voids has been to use a load sweep at slab corners and assess the linearity and intercept of the load-deflection plot.[46,63] This has, however, been done on many projects without regard to whether or not the slabs were curled at the time of testing. Improved guidelines for evaluating corner deflections to distinguish curling from loss of support have more recently been developed by Crovetti.[64,65]

Short-pulse, ground-penetrating radar (GPR) works on the principle of wave propagation and reflection and transmission of electromagnetic waves. A brief pulse of electromagnetic energy is directed into the pavement. Dielectric discontinuities in the pavement (for example, changes in material type, moisture content, or density) cause part of the incident wave to be reflected and part to be transmitted to the next layer. This reflected energy is recorded by devices at the surface, and analyzed to determine pavement properties (layer thicknesses, voids, moisture contents, etc.). Ground-penetrating radar has been shown to be capable of detecting voids greater than about an eighth of an inch thick, but much less capable of detecting shallower voids.[60,66]

Epoxy-core testing involves drilling a small hole through the concrete slab and into the base. A two-part epoxy, dyed with food coloring, is poured into the hole. Once the epoxy has hardened, a core is taken from the pavement and examined to assess the presence of a void and its thickness.[67]

Drilling Holes

Pneumatic and hydraulic drills are commonly used to drill holes in concrete pavements for undersealing purposes. To avoid spalling and cracking the concrete, the downward pressure of the drill should not exceed 200 pounds.[60] The size of the grout injection holes depends primarily on the type of stabilization material to be used. Hole diameters of 1.25 to 2 inches are typical for pozzolan-cement grouts, while hole diameters of less than 1 inch are typical for polyurethane.

The hole pattern selected by the contractor depends on the type of concrete pavement, the joint spacing, the condition of the slab, the fluidity of the grout, and the indications from the void detection process about the locations and sizes of the voids. A common drilling pattern has four holes, two on each side of a joint, one in each wheelpath.

Grout Injection

Low flow rates (about 1.5 gallons per minute) and pressures (between 25 and 250 psi) are used to inject grout into the drill holes. The injection equipment should be equipped with some return hose or rapid shutoff valve which can be used to stop the flow of grout as soon as movement of the slab is detected. Once the grout is pumped in, the hole is sealed with a grout packer – a small, tapered pipe which fits into the hole and prevents the grout from backing up.

Monitoring Slab Movement During Undersealing

Uplift beams are typically used to monitor movement of the slab corner during the grout injection process. An uplift beam is something like a simplified Benkelman Beam: a long beam fitted with a dial gauge which rests on the shoulder, with one end on the slab, near the point of injection. Some contractors use laser levels to monitor slab uplift.[60] Undersealing specifications typically call for grout injection to stop at or before the detection of 0.10 inch of slab uplift.

Verification of Undersealing

The effectiveness of undersealing at filling voids may be assessed by nondestructive deflection testing or ground-penetrating radar testing, conducted about 24 to 48 hours after undersealing. For this verification to be meaningful, it should be conducted at the same time of day as was the undersealing – that is, when the temperature gradients in the slab are comparable.

Opening to Traffic

Pavements undersealed with pozzolan-cement grout can usually be opened to traffic within an hour or two, while pavements undersealed with polyurethane can usually be opened to traffic within a half hour.

Performance

The major factors influencing the effectiveness of undersealing are the appropriateness of its timing, the accuracy of the method used to detect voids to be undersealed, the use of appropriate stabilization materials, and the appropriateness of the construction methods used.[60] No service life estimates are available in the literature for undersealing.

Load Transfer Restoration

Appropriate Use

Load transfer restoration is the installation of load transfer devices (either dowel bars or other devices which have been developed for this purpose) across undowelled joints and/or cracks. Load transfer restoration may be done as part of a restoration of the existing pavement structure, or may be done in preparation for resurfacing. Although load transfer retrofitting is a restoration technique, some argue that it increases structural capacity, because it increases load transfer and thereby decreases corner stresses in the adjacent slabs.

Load transfer restoration is best suited for undoweled jointed plain concrete pavements that have very poor load transfer at transverse joints and cracks, but still have considerable remaining structural capacity. Load transfer restoration has also been applied successfully to doweled jointed reinforced concrete pavements, at existing doweled joints and midslab transverse cracks. Load transfer restoration has not yeet been applied to continuously reinforced concrete pavement.

Load transfer restoration may also be done prior to placement of an asphalt overlay or bonded concrete overlay. It is not often used for this purpose, however, since full-depth repairs are often done in preparation for resurfacing and accomplish the same ends as load transfer restoration.

Among the indicators which have been suggested for assessing when it is appropriate to do load transfer restoration are the following:[6]

- Deflection load transfer of 60 percent or less,
- Differential deflections of 10 mils or more,
- Faulting greater than 0.10 inch, or
- Total faulting of 500 mm/km or more.

A limitation of deflection-based indicators of the need for load transfer restoration at individual joints is that they are usually stated without any indication of the temperatures at which they should be measured.

Limitations

Pavements with D-cracking are not good candidates for load transfer restoration. In D-cracked pavements, the concrete in the vicinity of transverse joints and cracks is generally too unsound to make load transfer restoration successful.

For the same reason, pavements with alkali-aggregate distress are not good candidates for load transfer restoration either. However, since pavements with reactive aggregates tend to develop distress due to slab expansion and high compressive stresses at joints and cracks, it is not likely that such pavements would exhibit poor load transfer.

Concurrent Work

When load transfer restoration is done as part of a comprehensive restoration strategy, it should be done after ful-depth repair, undersealing, partial-depth repair, and/or subdrainage retrofitting. Diamond grinding, joint resealing, and crack sealing should be done after load transfer restoration.

Materials

The two devices most often used for load transfer restoration are (a) dowels bars placed in slots sawed across the joint or crack, and (b) cylindrical shear devices placed in core holes drilled across the joint or crack. Other devices that have been used for load transfer restoration, with less success, include miniature I-beams and deformed reinforcing bars.

Dowels used for retroffiting load transfer should be smooth, epoxy coated to prevent inhibit corrosion, and lubricated to permit horizontal unrestrained opening and closing of the joint or crack. The dowels should be fitted on both ends with expansion caps that allow about a quarter inch of horizontal movement. The dowels should be mounted on chairs within the slots sawed across the joint or crack. The joint or crack should be maintained through the slot using a joint-forming insert. The emphasis here is on retrofit dowels, as they are the most commonly used devices. A diagram of a retrofit dowel installation is shown in Figure B-12.

Double-V shear devices are metal cylinders which have a V-shaped piece of metal on each side to produce some expansion pressure against the sides of the core holes. These devices are compressed slightly before insertion into the core holes. A diagram of a double-V shear device is shown in Figure B-13.

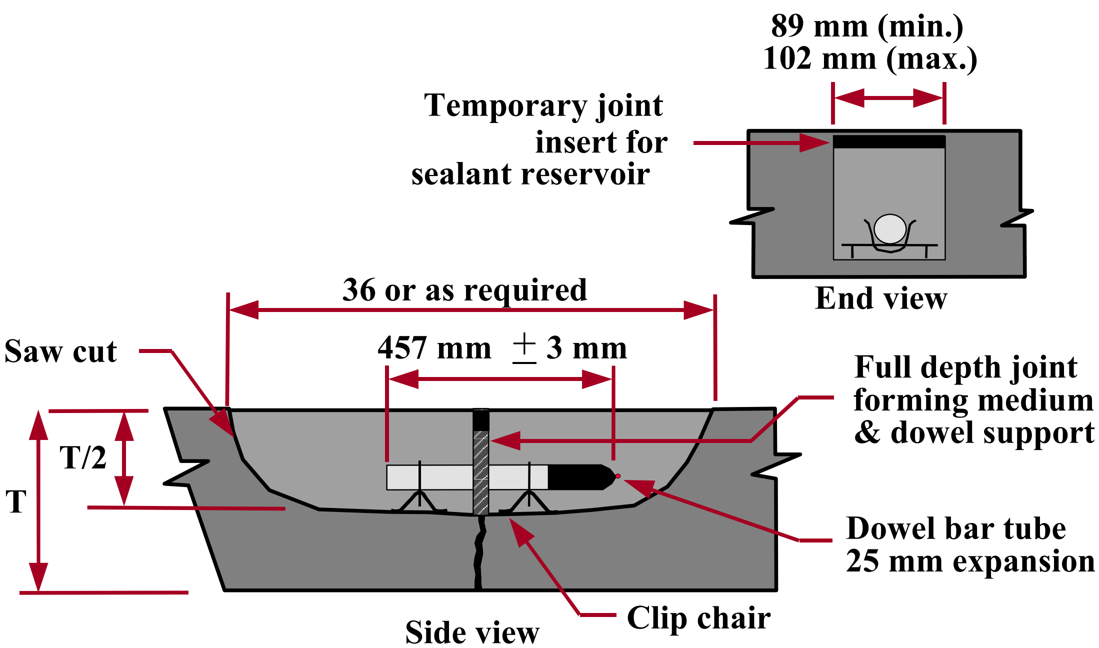

Figure B-12. Diagram of retrofit dowel installation.[4]

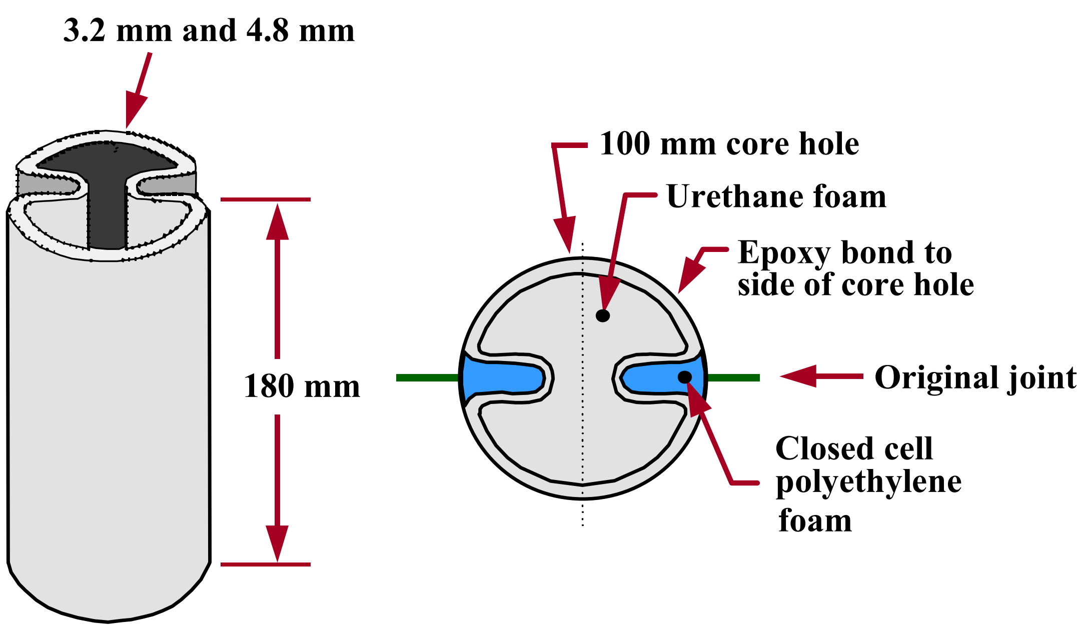

Figure B-13. Diagram of double-V shear device.[4]

Suitable backfill materials are ones which are low in shrinkage, are thermally compatible with the concrete, achieve a strong bond with the existing concrete, and develop strength rapidly. In general, materials used for partial-depth repairs are also good backfill materials for retrofit load transfer devices. These include Type III portland cement concrete, polymer concretes, epoxy concretes, and proprietary materials such as Set 45 and Road Patch.

Design

The load transfer system (device type, size, number, and layout) should be selected considering the truck traffic using the pavement. It is recommend that dowels be a minimum of 1.25 inches in diameter, and 18 inches long. The outermost dowel should be located no more than 12 inches from the slab edge.[1,68] Among the recommended dowel layout-diameter combinations that may be used, depending on the slab thickness and traffic level, are the following:

- 2 per wheelpath, 1.625-inch diameter
- 3 per wheelpath, 1.5-inch diameter
- 4 to 6 per wheelpath, 1.25-inch diameter

An experimental load transer retoration project on Interstate 10 in Florda demonstrated that 3 dowels per wheelpath and 5 dowels per wheelpath performed about the same in terms of faulting. For the same number of dowels per wheelpath, 1.5-inch dowels had 30 percent lower corner deflections than 1-inch dowels.[69]

Construction

Cutting Slots

Diamond saw slot cutters and modified milling machines have been used to saw slots across joints and cracks for retrofitting dowels. Diamond saw slot cutters, which are the more commonly used of the two devices, make two parallel cuts for each slot. The best success in cutting well aligned slots is achieved using either (a) ganged sawblades that can cut cut up to eight slots at the same time, or (b) marking the slot ocations on the pavement using a template. Since the length of the slot at the bottom must be as long as the dowel bar plus its expansion caps, and the diamond saw blades are circular, the length of the slots at the surface is about 3 feet. When dowel slots are cut using diamond saw blades, traffic can be allowed back into the lane until it is time for the next step in the construction process: removing the concrete in the slots between the sawcuts.

When modified milling machines are used to saw the slots, the milling head is positioned before the joint crack and moved forward across the joint or crack. An advantage of milling over sawing is that it creates the slots in one step, without subsequent removal of concrete within the slots. Another advantage is that milling produced rougher-textured slot surfaces which achieve better bond with the backfill material. A disadvantage of milling is that traffic cannot be left back into the lane until the retrofit dowel installation is complete. Another disadvantage is that the tips on the cutting head will wear down as the cutting proceeds. As these tips wear down, the slot opening will become narrower. It is important to monitor the slot width to ensure that the opening is adequate for insertion of the dowel bar.   Another disadvantage of milling is that it produces slots which are slightly more spalled around the edges than those produced by sawing.

Preparing Slots

When the slots are cut by diamond saw blades, the concrete left within the slots between the sawcuts must be chipped out using lightweight jackhammers. An experienced construction worker can quickly remove the concrete in a sawed dowel slot in two or three large pieces. The bottom of the dowel slot typically must be leveled by additional chipping with a small jackhammer.

Sawed slots slots should be cleaned by sandblasting and then by airblasting. Milled slots should be cleaned at least by airblasting. Where or not sandblasting is also necessary for milled slots is sill under study.[1] The joint or crack should be caulked along the bottom and sides of the slot, to keep the backfill material from flowing out of the slot.

Placing Dowel Bars

Before being positioned in the slots, the dowels should be fitted with nonmetallic or epoxy-coated expansion caps, joint forming insert, and nonmetallic or epoxy-coated chairs. The chair should be of sufficient height to support the dowel one half inch above the bottom of the slot, so that the backfill material will be able to flow under and around the dowel.

The dowels should be coated with form oil or light grease to prevent them from bonding with the backfill material as it hardens. The dowels should be coated with the bondbreaker before they are placed in the dowel slots, so that excess bondbreaker does not drip down into the slot and interfere with bond between the backfill material and the concrete.

Backfilling Slots

The backfill material should be placed in the slots and consolidated with a small spud vibrator. After the backfill material is consolidated, a curing compound should be applied. Finishing and texturing are not critical for dowel backfill material because of their small surface area.

Opening to Traffic

Lanes with retrofitted dowels installed can usually be opened to traffic within 2 to 6 hours after placement of the backfill material. A typical minimum compressive strength for opening to traffic is 2000 psi. On one retrofit dowel project in Washington State, a compressive strength of 4000 psi was attained in 2 hours. South Dakota requires a compressive strength of 4000 psi in 6 hours.

The best final appearance and smoothness of retrofit dowel slots is attained when diamond grinding is conducted after dowel retrofitting. Diamond grinding also removes any existing faulting present and improves the smoothness of the entire project.

Performance

Load transfer restoration has been conducted with great success on several projects, in Washington,[70,71] Puerto Rico,[72] and other States.[47] However, construction errors can cause failures in load transfer restoration projects. The success of load transfer restoration depends greatly on the quality of construction and the adequacy of the dowel size and layout designs.

The American Concrete Pavement Association estimates the service life of load transfer restoration in the range of 8 to 10 years.[6]

Diamond Grinding

Appropriate Use

Diamond grinding is the removal of a shallow depth of material from a concrete pavement surface. Diamond grinding is most frequently done to remove faulting at joints and cracks, or to remove bumps. In addition to improving the ride quality of the pavement, diamond grinding improves surface texture and friction.

Limitations

The appropriateness of diamond grinding and other nonoverlay restoration techniques depends greatly on the amount of structural distress present. Pavements that have little remaining structural life are not good candidates for restoration. An overlay or reconstruction is usually a more cost-effective rehabilitation alternative in this situation.

Diamond grinding does not eliminate the cause of faulting; it only removes existing faulting. Unless actions are taken to reduce deflections at joints and cracks (such as undersealing and/or load transfer restoration), diamond grinding is likely to recur at a faster rate than it originally developed after construction of the pavement. The reasons for this are the erosion of aggregate interlock and the loosening of dowels, if present, that occur over time.

Concurrent Work

Diamond grinding may be done alone, or as part of a comprehensive concrete pavement restoration strategy. Diamond grinding should be done after full-depth repair, undersealing, nad partial-depth repair. Joint resealing should be done after diamond grinding.

Construction

Diamond grinding is accomplished using a set of closely spaced diamond saw blades mounted on a rotating drum. Traditionally, diamond grinding proceeds longitudinally against the direction of traffic, although it has been suggested that equally good results may be obtained by grinding in either direction.[73] The saw blades are typically spaced at between about 54 and 59 per foot.[74] Grinding heads are typically about 3 feet wide, and thus require four passes per traffic lane to remove faulting and bumps across the whole lane width.

The rate at which a grinding machine can advance depends on the hardness of the aggregate and the size of the diamond particles embedded in the saw blades. Larger diamond particles are better for softer aggregate types, while smaller diamond particles are better for harder aggregate types.

The texture and friction attained by diamond grinding depends on the spacing of the saw blades and the hardness of the concrete. Ideally the blades are spaced such that the pieces of concrete left between the saw blade grooves will break off fairly evenly and leave a uniformly corduroy-like surface appearance, as illustrated in Figure B-14.

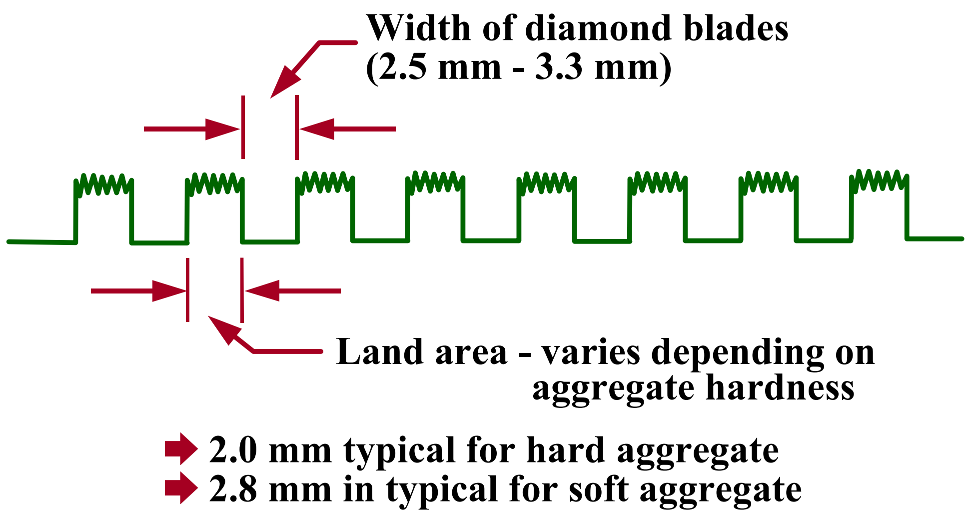

Figure B-14. Spacing of sawblades for diamond grinding.[4]

Smoothness specifications for diamond grinding are equivalent to those used for new construction. The California profilograph is widely used for acceptance testing of the pavement profile after grinding.

Performance

Diamond grinding restores both the smoothness and friction of concrete pavement surfaces to like-new condition. However, the longevity of diamond grinding depends not only on how fast faulting recurs after grinding, but also on how soon other distresses (e.g., cracking, joint deterioration) develop to levels that require some other type of rehabilitation.

The American Concrete Pavement Association estimates the service life of diamond grinding in the range of 10 to 15 years.[6] A recent evaluation of diamond grinding projects conducted for the American Concrete Pavement Association found that diamond-ground projects survived an average of about 14 years or 12 million ESALs before they had to be reground, overlaid, or reconstructed.[75]

New York estimates a service life of 5 years for diamond grinding, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] Utah estimates a service life of 10 years for diamond grinding, when done in conjunction with load transfer restoration and undersealing and/or slabjacking.[59] Wisconsin estimates a service life of 5 to 10 years for diamond grinding when done in conjunction with other repairs.[35] Arizona estimates a service life of 14 years for diamond grinding plus joint resealing.[24]

Asphalt Overlay of Concrete Pavement

Appropriate Use

An asphalt overlay of a concrete pavement or asphalt-overlaid concrete pavement may be placed to improve ride quality and/or surface friction, and/or or may be placed for the purpose of substantially increasing structural capacity.

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[23] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, a jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[23] Low-severity transverse cracks are not considered structural distress.

As a general rule, a continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[23]

Limitations

The 1993 AASHTO Guide[1] identifies the following as conditions under which an asphalt overlay may <u>not</u> be a feasible rehabilitation technique for a concrete or existing asphalt-overlaid concrete pavement:

- The amount of deteriorated slab cracking and joint spalling is so great that complete removal and replacement of the existing surface is dictated.
- Significant deterioration of the concrete slab has occurred due to severe durability problems (e.g., “D” cracking or reactive aggregate distress).
- Vertical clearance at bridges is inadequate for the required overlay thickness.

The 1993 AASHTO Guide further states that vertical clearance problems may be addressed by reducing the overlay thickness under bridges (although this may result in early failure at these locations), by raising the bridges, or be reconstructing the pavement under the bridges. Thicker asphalt overlays may also necessitate raising signs and guardrails, as well as increasing side slopes and extending culverts. Sufficient right-of-way must be available or obtainable to permit these activities.[23]

Concurrent Work

An asphalt overlay of a concrete or asphalt-overlaid pavement may be used alone or in combination with full- and/or partial-depth repair of the existing pavement. If the pavement has an existing asphalt overlay, this surface may be prepared by rut filling, cold milling, or hot in-place recycling prior to placement of the overlay.

Materials

Asphalt concrete mixes used for overlays are usually of the same mix design as those used for new asphalt pavement construction.

Design

The most commonly used approach to structural design of asphalt overlays of concrete pavements and asphalt-overlaid concrete pavements is the structural deficiency approach, exemplified by the 1993 AASHTO procedure[1] The structural deficiency concept is illustrated in Figure B-4, shown previously.

The concept of the structural deficiency approach to overlay design is that the overlay satisfies a deficiency between the structural capacity required to support traffic over some future design period, and the structural capacity of the existing pavement. In the 1993 AASHTO overlay design methodology for overlays of concrete pavements, the former is referred to as the future slab thickness (D_f) and the latter is referred to as the effective slab thickness (D_{eff}). There are eight steps in the process of determining the overlay thickness which will satisfy the structural deficiency, $D_f - D_{eff}$:

1. Characterization of existing pavement design: specifically, the existing slab type and thickness, type of load transfer, and type of shoulder.

2. Traffic analysis: determination of the future 18-kip ESALs anticipated in the design lane over the analysis period, and if possible, the past 18-kip ESALs over the life of the existing pavement as well.

3. Condition survey: quantities of transverse cracking, transverse joint deterioration, punchouts, full-depth asphalt patches, wide joints, D-cracking, reactive aggregate distress, faulting, and pumping.

4. Deflection testing: recommended to determine the effective dynamic k value of the foundation, the elastic modulus of the existing concrete slab, and the degree of deflection load transfer at joints.

5. Coring and materials testing: recommended to obtain concrete samples for indirect tensile testing.

6. Determination of the required slab thickness for future traffic (D_f): done using the 1986 AASHTO model for design of new concrete pavement, with the design static k value estimated as a function of the backcalculated effective dynamic k value.

7. Determination of the effective slab thickness of the existing pavement (D_{eff}) by one or more of two methods:

   - The condition survey method – D_{eff} is estimated by multiplying the existing slab thickness by adjustment factors for the amounts and severities of: (a) joint and crack deterioration, (b) durability-related distress, and (c) fatigue damage.

   - The remaining life method – Deff is estimated as a function of the past 18-kip ESALs carried by the existing pavement as a proportion of the 18-kip ESALs a pavement of its original structural capacity would be expected to be able to carry. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

8. Determination of the required overlay thickness: by multiplying the structural deficiency (D_f – D_{eff}) by an adjustment factor, A, which converts concrete thickness deficiency to asphalt concrete overlay thickness requirement.

A value of 2.5 has traditionally been used for the adjustment factor A. This value was based on the results of accelerated traffic tests conducted by the Corps of Engineers in the 1950's.[76,77] The value 2.5 does not represent the best fit of the relationship of concrete thickness deficiency to asphalt overlay thickness in those field tests, but rather a conservative value suggested by the Corps for use in design.[77] However, an A value of 2.5 can lead to excessive overlay thickness for larger concrete thickness deficiencies. A formula for the A factor as a function of the magnitude of the concrete thickness deficiency was developed by Hall[78] using elastic layer analysis, and is recommended in the 1993 AASHTO Guide,[1] in place of a constant A factor.

The Asphalt Institute overlay design manual[2] also presents a structural deficiency method for design of asphalt overlays of concrete pavements. Several State DOTs have adapted either the 1993 AASHTO or the Asphalt Institute structural deficiency method to their own purposes for design of asphalt overlays of asphalt pavements.

Construction

Preoverlay Repair

In an asphalt pavement of a jointed concrete pavement, reflection cracks typically develop relatively soon after the overlay is placed, often in less than a year. The rate at which they deteriorate depends on the extent and quality of preoverlay repair, as well as the overlay thickness, joint spacing, traffic level, and climate. Thorough repair of deteriorated joints and working cracks with full-depth dowelled concrete repairs reduces the rate of reflection crack occurrence and deterioration.

Permanent patching of punchouts and working cracks will delay for many years the occurrence and deterioration of reflection cracks in asphalt overlays of continuously reinforced concrete pavements. Reflection crack control treatments are not necessary for asphalt overlays of continuously reinforced concrete pavements, as long as continuously reinforced concrete repairs are used for deteriorated areas and cracks.[78]

The importance of high-quality preoverlay repair to the prevention of reflection cracking in an asphalt overlay of a continuously reinforced concrete pavement is demonstrated by the performance of an experimental project on I-57 in Illinois. Every patch, crack, and joint on this project was mapped before the asphalt overlay was placed. Distress surveys and nondestructive deflection tests were performed on the overlaid pavement immediately after placement , and after one year, five years, and ten years. The locations of all repairs were known by station, by photographs, and by repair numbers stamped in the overlay. More than 90 percent of the tied full-depth concrete repairs could not be detected by reflection cracks in the asphalt overlay even after ten years of service.[79]

Correcting Rutting

Rutting in an existing asphalt-overlaid concrete pavement may removed by cold milling, filled with asphalt concrete compacted prior to placing the first overlay course, or filled with a leveling course. This last option might result in rutting occurring in the overlay at a faster rate than it would with the former two options.

Reflection Crack Control

In addition to or in place of preoverlay repair, several different reflection crack control treatments have been used with asphalt overlays of concrete and asphalt-overlaid concrete pavements. Among the more effective reflection crack control measures are the following:

- Sawing and sealing joints in the overlay at locations coinciding with joints in the underlying concrete pavement,
- Placing an asphalt-stabilized large-stone crack relief layer,
- Placing a synthetic fabor or stress-absorbing interlayer prior to or within the asphalt overlay,
- Fracturing the existing concrete slab (discussed further later in this Appendix), or
- Increasing overlay thickness.

Performance

The performance of an asphalt overlay of a concrete or existing asphalt-overlaid concrete pavement depends primarily on the type of existing pavement, the presence of durability distress, the thickness of the overlay, its asphalt concrete mix design, the type and extent of preoverlay repair and surface preparation, and the number and magnitude of applied loads. An asphalt overlay of a concrete pavement is expected to have a service life in the range of 8 to 15 years.

Several agencies have developed asphalt overlay service life estimates which are not a function of the type of concrete pavement. The American Concrete Pavement Association estimates service lives of 8 to 12 years, 10 to 15 years, and 10 to 20 years, depending on design, for asphalt overlays of Interstate, primary/secondary, and municipal concrete pavements respectively.[6] Ohio estimates that an asphalt overlay of a concrete pavement will require rehabilitation (full-depth rigid repairs, milling, and a follow-up overlay) every 8 to 12 years.[7] New York estimates a service life of 15 years for 3- to 4-inch asphalt overlays of concrete pavements, and a service life of 8 years for sawed and sealed joints in asphalt overlays, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] West Virginia estimates a service life of 8 years for first asphalt overlays of concrete pavements.[34] Arizona estimates a service life of 10 years for asphalt overlay of concrete pavement.[24]

Asphalt overlays of jointed concrete pavements exhibit fair to good performance when the concrete is not D-cracked, and poor to fair performance when the concrete is D-cracked. Extensive preoverlay repair (full-depth repair at all deteriorated joints and midslab working cracks) is typically required to achieve the best performance possible.[78] A survival analysis of asphalt overlays on Interstate pavements in Illinois indicated a mean life of 12 years or 18 million ESALs for thin overlays (3 to 3.25 inches) of non-D-cracked jointed reinforced concrete pavement, and 16 years or 45 million ESALs for thick overlays (4 to 6 inches) of non-D-cracked jointed reinforced concrete pavements. The survival analysis indicated a mean life of 7 years or 6 million ESALs for thin overlays (3 to 3.25 inches) of D-cracked jointed reinforced concrete pavement, and 14.5 years or 15 million ESALs for thick overlays (4 to 6 inches) of D-cracked jointed reinforced concrete pavements.[78,80] Indiana estimates a service life of 10 years for asphalt overlay of jointed concrete pavement without sawing and sealing joints, or 13 years with sawing and sealing joints.[36]

Asphalt overlays of continuously reinforced concrete pavements exhibit good to excellent performance when the concrete is not D-cracked and adequate preoverlay repair is done. The service life of asphalt overlays of non-D-cracked CRCP is more likely to be controlled by rutting than by reflection cracking. Asphalt overlays of continuously reinforced concrete pavements exhibit fair to very poor performance when the concrete is D-cracked.[78] It is difficult to place tied CRC repairs in such pavements, and untied concrete or asphalt repairs are often done instead. Illinois' survival analysis indicated a mean life of 10 years or 37 million ESALs for thin overlays (3 to 3.25 inches) of non-D-cracked continuously reinforced concrete pavement. The survival analysis indicated a mean life of 7.5 years or 6 million ESALs for thin overlays (3 to 3.25 inches) of D-cracked continuously reinforced concrete pavement.[78,80] Indiana estimates a service life of 12 to 15 years for asphalt overlay of continuously reinforced concrete pavement.[36]

Second asphalt overlays of existing asphalt-overlaid concrete pavements are estimated to have a service life in the range of 8 to 15 years. New York estimates a service life of 15 years for 3 or more inches of new asphalt without cold milling or hot surface recycling, and for 3 or more inches of new asphalt after cold milling to a depth of 1.5 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] West Virginia estimates a service life of 8 years for the second asphalt overlay of existing asphalt-overlaid concrete pavement.[34] Wisconsin estimates a service life of 4 to 8 years for thin asphalt overlays of existing asphalt-overlaid concrete pavements.[35]

Asphalt Overlay of Fractured Concrete Pavement

Appropriate Use

An asphalt or concrete overlay of a fractured concrete pavement is placed to increase structural capacity. Slab fracturing may be done for two reasons: to attempt to mitigate reflection cracking in the overlay, and/or to dispense with preoverlay repair of a concrete pavement with extensive cracking and/or materials-related deterioration (e.g., "D" cracking, alkali-silica reaction, alkali-carbonate reaction).

Jointed plain concrete pavements are cracked and seated, meaning that the slab is cracked into pieces between about 1 and 3 feet on a side, and seated with a heavy roller. Jointed reinforced concrete pavements are broken and seated, meaning that the slab is broken and the reinforcing steel is ruptured (this may require greater impact force than cracking an unreinforced slab), prior to seating with a heavy roller. Jointed plain, jointed reinforced, and continuously reinforced pavements may be rubblized, meaning that the slab is pulverized into pieces typically about 12 inches or less in size.

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[23] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[23] Low-severity transverse cracks are not considered structural distress.

As a general rule, a continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[23]

Limitations

Inadequate vertical clearance at bridges is cited in the 1993 AASHTO Guide[1] as a potential limitation to the feasibilty of asphalt overlay with slab fracturing as a rehabilitation option for concrete pavements. The 1993 AASHTO Guide further states that vertical clearance problems may be addressed by reducing the overlay thickness under bridges (although this may result in early failure at these locations), by raising the bridges, or be reconstructing the pavement under the bridges. Thicker asphalt overlays may also necessitate raising signs and guardrails, as well as increasing side slopes and extending culverts. Sufficient right-of-way must be available or obtainable to permit these activities.[23]

Concurrent Work

The decision to fracture a concrete slab prior to overlaying with asphalt eliminates the need for preoverlay repairs. To fracture a concrete a pavement with an existing asphalt overlay, the overlay should first be milled off completely.

Design

Structural design for an asphalt or concrete overlay of a cracked and seated or broken and seated concrete may be done using either flexible pavement or rigid pavement overlay design methods. Structural design for an asphalt or concrete overlay of a rubblized pavement may be done using overlay design approaches, or may be designed as a new surface on a high-strength granular base.

The approach taken in the 1993 AASHTO Guide[1] to design of asphalt overlays of fractured slabs is a structural deficiency approach, illustrated previously in Figure B-4. The overlay must satisfy the deficiency between the Structural Number (SN_f) required to support traffic over some future design period, and the effective Structural Number (SN_{eff}) of the existing pavement (after fracturing). There are eight steps in the process of determining the overlay thickness which will satisfy the structural deficiency:

1. Characterization of existing pavement design: specifically, the thicknesses and material types of each pavement layer, and any available subgrade soil information.

2. Traffic analysis: determination of the future flexible pavement 18-kip ESALs anticipated in the design lane over the analysis period.

3. Condition survey: distress types and quantities are presumably used in the process of selecting the type of overlay rehabilitation to be done, but condition survey data are not used in the thickness design of an asphalt overlay of a fractured slab.

4. Deflection testing: recommended to determine the *in situ* subgrade resilient modulus (as a function of deflections measured sufficiently far away from the load plate).

5. Coring and materials testing: recommended to obtain subgrade soil samples for laboratory resilient modulus testing, and to obtain samples of the base layers for visual examination of their condition.

6. Determination of the required Structural Number for future traffic (SN_f): done using the 1986 AASHTO model for design of new asphalt pavement, with the design subgrade resilient modulus either obtained from laboratory testing or estimated from the backcalculated *in situ* subgrade modulus.

7. Determination of the effective Structural Number of the existing pavement (SN_{eff}), by multiplying the fractured concrete slab thickness and base thickness by structural coefficients and drainage coefficients which reflect their condition after fracturing.

8. Determination of the required overlay thickness: by dividing the structural deficiency ($SN_f – SN_{eff}$) by the layer coefficient appropriate for new asphalt overlay material.

Perhaps the most contentious aspect of overlay design for fractured slabs by the structural deficiency approach is what structural coefficient should be assigned to the fractured slab. The 1993 AASHTO Guide[1] recommends the following ranges for structural coefficients for different types of slab fracturing:

Rubblized:	0.14	–	0.30
Crack and seat:	0.20	–	0.35
Break and seat:	0.20	–	0.35

Other recommendations for overlay design for fractured slabs, including recommended ranges of structural coefficients and overlay thickness design tables, are provded in the National Asphalt Pavement Association report, *Guidelines for Use of HMA Overlays to Rehabilitate PCC Pavements*. [81]

In a recent study conducted for the American Concrete Pavement Association, a range of 0.15 to 0.25, was recommended for the structural coefficient for use with the 1993 AASHTO method for design of asphalt overlays for all three types of fractured slabs.[82] For a given design serviceability loss, values at the low end of this range are more appropriate for larger future thickness requirements, while values at the high end of this range are more appropriate for smaller future thickness requirements. Structural coefficient values within the recommended range will yield asphalt overlay thicknesses for fractured concrete slabs which are consistent with both the rigid model and flexible model approaches to overlay design in the 1993 AASHTO methodology.

Thompson[83] has demonstrated that a mechanistic-empirical approach to evaluation of the structural capacity of in-service asphalt pavement can be used to determine the required overlay thickness for rubblized concrete pavements. An algorithm to predict the tensile strain at the bottom of the asphalt overlay as a function of a deflection basin parameter called the Area Under the Pavement Profile (AUPP) has been validated with measurements from instrumented full-depth and conventional flexible pavements. Falling weight deflectometer (FWD) data from rubblized concrete pavements with asphalt concrete overlays were used to develop a relationship between AUPP and an overlay stiffness parameter (Eh^3, where E is the asphalt concrete modulus and h is the asphalt overlay thickness). The estimated strain is an input to an asphalt concrete fatigue model. The asphalt overlay thickness is selected to limit the asphalt concrete tensile strain to an acceptable level.

Additional guidelines for the appropriate use, design, and construction of asphalt overlays of cracked or broken and seated concrete pavements are given in NCHRP Synthesis No. 144, *Breaking/Cracking and Seating Concrete Pavements*.[84]

Construction

Pavement Breaking

Among the different types of equipment are available for cracking or breaking jointed concrete slabs are resonant pavement breakers, guillotines hammers, multiple head breakers, hydraulic or pneumatic hammers, whiphammers, pile drivers, and wrecking balls. Among these types of equipments, only resonant pavement breakers and guillotine hammers are effective in rubblizing jointed and continuously reinforced concrete pavements.

Resonant (also called sonic) pavement breakers impart a low impact, low-amplitude, high frequency (2000 pounds force, 1.5 inch amplitude, 44 Hz frequency) vibration to the slab. The resonant pavement breaking machine itself weighs 28 tons and is 7 feet wide by 25 feet long.[81] Multiple passes of the resonant breaker are required to rubblize a single traffic lane. The rubblization achieved by the resonant pavement breaker is very thorough, converting the concrete slab to a layer of particles ranging from sand size at the surface to about 6 inches maximum at the bottom.

The guillotine device has a strike plate of about 4 to 7 inches thick by 5 to 10 feet wide. Pavement breaking is accomplished by lifting the guillotine to a height of 5 to 10 feet and dropping it on the pavement, producing impact forces of up to 10 tons. Multiple head breakers operate similarly but consist of twelve to sixteen hammers. Unlike the resonant pavement breaker, the multiple head breaker can rubblize a full lane width in a single pass.

Hydraulic and pneumatic hammers fracture slabs by dropping a hammer weighing between 1000 and 1500 pounds from a height of 4 to 12 feet. The hammer strikes the concrete surface through a breaking head which is about 2 to 6 inches in diameter. After each drop, the hammer is lifted again by hydraulic or pneumatic pressure, and the machine is moved forward. Because of the smaller contact area of the breaking head, hydraulic/pneumatic hammers tend to produce more spalling at the surface than guillotine or multiple-head breakers.

A whiphammer is a flexible arm made of leaf springs, with a flat cracking head on one end, attached to the back of a truck. The arm is raised hydraullically and swung down to strike the pavement. The arm can also be moved in an arc of about 7 feet from side to side. Each blow to the pavement produces cracking in a radius of about 18 to 24 inches. Like hydraulic/pneumatic hammers, whiphammers may produce excessive spalling at the surface. There is also considerable doubt as to whether they are capable of applying sufficient force to rupture reinforcing steel or separate it from the surrounding concrete. They are therefore considered suitable for cracking and seating of jointed plain concrete pavements, but not breaking and seating of jointed reinforced concrete pavements.

Pile drivers are similar to hydraulic and pneumatic hammers, but instead of the impact force being produced by the free fall of the hammer, it is produced by the controlling the flow of diesel fuel to the hammer. The greater the fuel flow, the greater is the force applied to the pavement. Each impact produces cracking in a radius of about 18 to 24 inches, so several passes are required to fracture a single traffic lane.

Wrecking balls are perhaps the crudest devices for fracturing concrete slabs, but can be used with success, depending on the skill of the operator in controlling the drop height and movement of the ball. The difficulties associated with controlling the movement of the wrecking ball make this fracturing technique inadvisable when traffic flow is being maintained in the adjacent lane.[81]

Preparing Fractured Surface

If the concrete slab was cracked or broken and seated, it should receive two or three passes of a heavy roller (e.g., 35 to 50 tons) to seat the pieces firmly. Any weak areas where the roller breaks through the cracked concrete layer should be excavated and repaired. A tack coat should then be applied to the concrete surface. If the concrete slab was rubblized, it should be compacted with several passes of a 10-ton vibratory roller. Some agencies apply a prime coat to the rubblized surface prior to placement of the overlay.

Placing the Overlay

The asphalt overlay should be placed in one or more lifts, as necessary, and compacted with steel-wheeled rollers, just as for conventional asphalt pavement or overlay construction.

Performance

A rubblized concrete slab with an asphalt overlay is arguably similar to a conventional flexible pavement, in terms of the predominant distress types likely to occur. fatigue cracking and rutting in the wheelpaths. The overlay thickness requirement for a rubblized slab is nearly always controlled by the fatigue criterion (tensile strain at the bottom of the asphalt concrete layer).

However, a rubblized concrete slab does not necessarily exhibit the same material properties as a conventional crushed stone base, in terms of gradation, CBR, or backcalculated elastic modulus. Deflection testing indicates that rubblized concrete layers exhibit stiffer response than granular base layers of the same thickness. It is reasonable to expect that the fatigue performance of an asphalt overlay on a rubblized concrete slab would be better than that of the same thickness of asphalt concrete on a granular base. The full answer to the question of how a rubblized concrete slab compares to a crushed stone base must be determined on the basis of an analytical comparison of the long-term field performance of the two pavement types.

A cracked or broken and seated concrete slab, it is safe to say, is nothing like a conventional flexible pavement base. The predominant distress type expected in an asphalt overlay of a cracked or broken and seated concrete slab is reflection cracking. In this sense, this type of rehabilitation has much more in common with an asphalt overlay of an intact concrete slab. Although cracking or breaking and seating was originally developed as a reflection crack control technique, some field studies have indicated that in the long run, asphalt overlays of cracked or broken and seated concrete display the same quantity of reflection crack deterioration as asphalt overlays of intact concrete. Again, the answer to the proper characterization of cracked and broken concrete lies in the results of comparisons of the long-term field performance of the two pavement types.

Although backcalculation results suggest that a rubblized concrete slab is weaker than a cracked or broken and seated concrete slab, field performance studies suggest that in many cases a given thickness of asphalt overlay over rubblized concrete performs as well as or better than the same thickness of asphalt overlay over cracked or broken and seated concrete. Considering that structural coefficients should reflect the relative contribution of the layer to the performance of the pavement, this suggests that it is not appropriate to use a lower range of structural coefficients for rubblized concrete than for cracked or broken and seated concrete. Analyses of the longer-term performance of these rehabiltated pavement types will shed more light on this issue.

The typical range of service life for asphalt overlay of fractured concrete pavement is expected to be about 15 to 25 years. Ohio estimates that an asphalt overlay of a fractured concrete slab will require a thin overlay (1.25 to 4 inches) in year 8 to 12, and a thick overlay (4 to 8 inches), along with patching and milling, in year 16 to 22.[7] New York estimates a service life of 15 years for either a 5-inch asphalt overlay of a cracked and seated concrete pavement, or a 6-inch asphalt overlay of a rubblized concrete pavement, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[14] West Virginia estimates a service life of 8 years for asphalt overlay of broken and seated or rubblized concrete pavement.[34] Wisconsin estimates a service life of 15 to 18 years for an asphalt overlay of a rubblized concrete pavement.[35] Indiana estimates a service life of 12 to 15 years for an asphalt overlay of a cracked and seated concrete pavement, and a service life of 20 years for an asphalt overlay of a rubblized concrete pavement.[36]

Bonded Concrete Overlay

Appropriate Use

A bonded concrete overlay of a concrete pavement may be placed to increase structural capacity or to increase serviceability of an otherwise structurally sound concrete pavement. Bonded concrete overlays are not used often, because they perform best on pavements in good to fair condition, that is, pavements which are not in urgent need of rehabilitation.

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[23] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, a jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[23] Low-severity transverse cracks are not considered structural distress.

As a general rule, a continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[23]

Limitations

The 1993 AASHTO Guide[1] states that a bonded concrete overlay is a feasible rehabilitation alternative for a concrete pavement except when the conditions of the existing pavement (extensive slab cracking, joint spalling, and/or durability problems) dictate substantial removal and replacement, or when durability problems exist. Nonetheless, the sensitivity of a bonded concrete overlay to underlying pavement condition necessitates exhaustive repair, which is generally not cost-effective (in comparison with other resurfacing options) for pavements with substantial slab cracking, joint spalling, or durability distress.

Concurrent Work

A bonded concrete overlay of a concrete pavement is usually done in conjunction with thorough full- and/or partial depth patching of deteriorated cracks and joints.

Materials

Portland cement concrete mixes used for bonded concrete overlays may be of the same mix design as those used for new concrete pavement construction, or may include fibers, a higher cement content, and/or high-early strength (Type III) cement. More information on mix designs appropriate for bonded concrete overlays is available in American Concrete Pavement Association's technical bulletins on bonded concrete overlays[85] and fast-track paving.[39]

A bonding agent is normally applied to the existing pavement surface to enhance bond between the existing pavement and the overlay. The American Concrete Pavement Association recommends a target bond strength of 200 psi in shear testing.[85] Cement-sand mortars, cement slurries, and epoxies have been used successfully as bonding agents. In some instances, bonded overlays constructed without the use of a bonding agent have performed well.[1]

Design

Structural design of bonded concrete overlays is usually done by the Corps of Engineers method, a structural deficiency approach which is also used in the 1993 AASHTO Guide.[1] The concept of the structural deficiency approach to overlay design (illustrated previously in Figure B-4) is that the overlay satisfies a deficiency between the structural capacity required to support traffic over some future design period, and the structural capacity of the existing pavement. In the 1993 AASHTO overlay design methodology for concrete pavements, the former is referred to as the future slab thickness (D_f) and the latter is referred to as the effective slab thickness (D_{eff}). There are eight steps in the process of determining the overlay thickness which will satisfy the structural deficiency, $D_f - D_{eff}$:

1. Characterization of existing pavement design: specifically, the existing slab type and thickness, type of load transfer, and type of shoulder.

2. Traffic analysis: determination of the future 18-kip ESALs anticipated in the design lane over the analysis period, and if possible, the past 18-kip ESALs over the life of the existing pavement as well.

3. Condition survey: quantities of transverse cracking, transverse joint deterioration, punchouts, full-depth asphalt patches, wide joints, D-cracking, reactive aggregate distress, faulting, and pumping.

4. Deflection testing: recommended to determine the effective dynamic k value of the foundation, the elastic modulus of the existing concrete slab, and the degree of deflection load transfer at joints.

5. Coring and materials testing: recommended to obtain concrete samples for indirect tensile testing.

6. Determination of the required slab thickness for future traffic (D_f): done using the 1986 AASHTO model for design of new concrete pavement, with the design static k value estimated as a function of the backcalculated effective dynamic k value.

7. Determination of the effective slab thickness of the existing pavement (D_{eff}) by one or more of two methods:

   - The condition survey method – D_{eff} is estimated by multiplying the existing slab thickness by adjustment factors for the amounts and severities of (a) joint and crack deterioration, (b) durability-related distress, and (c) fatigue damage.

   - The remaining life method – Deff is estimated as a function of the past 18-kip ESALs carried by the existing pavement as a proportion of the 18-kip ESALs a pavement of its original structural capacity would be expected to be able to carry. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

8. Determination of the required overlay thickness: which is the difference between the required future thickness (D_f) and the existing pavement's effective thickness (D_{eff}).

Construction

Construction of a bonded concrete overlay requires careful preparation of the surface to ensure a strong bond between the existing slab and the overlay. The existing surface should be cleaned and then roughened by shotblasting or milling. Cold milling is used when up to 1 inch of the existing slab thickness needs to be removed (e.g., to maintain curb heights). Cold milling may cause some spalling of the surface which requires followup cleaning by sandblasting or waterblasting.

Opinions differ on whether or not a cement grout is needed to achieve adequate good bond between the existing pavement and the overlay. If a cement grout is used, it is essential that it be applied to the pavement right in front of the paver so that it cannot dry out before the overlay is placed.

According to the ACPA technical bulletin on bonded overlays, key factors influencing the quality of the bond achieved are: (1) the strength and integrity of the existing surface, and (2) the cleanlines of the surface. Other factors that are cited as fostering bond strength development include good consolidation of the overlay, good jointing techniques, and good curing measures. The bulletin also states that 'a cement grout can generate the necessary bond strength and has proven to be effective. However, where modern surface preparation technology is employed, bonding grout is not required to produce adequate bond strengths.' "[85]

Dowels are not normally used in bonded concrete overlays. The existing joint locations must be marked so that matching joints can be sawed in the overlay. The concrete overlay is placed either with fixed forms or a slipform paver. After placement on the prepared concrete surface, the overlay concrete is spread, consolidated, finished, textured, and cured. Materials commonly used for curing are white-pigmented liquid membrane curing compunds, waterproof paper or polyethylene sheets, and wet cotton mats or burlap. Because of the high surface area-to-volume ratio of bonded concrete overlays, and the high heat of hydration that may be generated by fast track mixes, adequate curing is critical to prevent shrinkage cracking.

Timing of the sawcutting operation is critical in bonded concrete overlay construction. The joints must be sawed to match the joints in the underlying pavement. Lightweight (Soff-Cut) saws may be used to saw the transverse and longitudinal joints early. Joint sawing should begin as soon as the partially hardened concrete can support the weight of the lightweight saw without causing damage to the surface or ravelling of the joint.[42] Secondary sawing for joint sealing purposes, if desired, may be done later using conventional saws.

Performance

The key factors influencing the performance of bonded concrete overlays are the appropriateness of their use, the condition of the existing pavement, the extent and quality of repair done, the strength of the bond achieved between the overlay and existing pavement, and the success of the joint sawing operation.

The typical range of service life for bonded concrete overlay of concrete pavement is expected to be about 15 to 25 years. The American Concrete Pavement Association estimates a service life of 15 to 25 years, depending on design, for a bonded concrete overlay.[6] New York estimates a service life of 20 years for bonded concrete overlay, with joint and crack resealing required at 8-year intervals, for highways with ADT between about 12,000 and 35,000, and about 5 percent trucks.[14]

Unbonded Concrete Overlay

Appropriate Use

An unbonded concrete overlay is a concrete overlay which is separated from an existing concrete slab by a new or existing asphalt concrete layer, or some other type of interlayer. An unbonded concrete overlay of a concrete or asphalt-overlaid concrete pavement is placed to increase structural capacity.

Unbonded overlays of all types (jointed plain, jointed reinforced, and continuously reinforced) can be placed on all types of concrete pavements, including those with existing asphalt overlays.

Unbonded concrete pavements are appropriate for pavements with little or no remaining structural life, and/or extensive and sever durability distress. Unbonded concrete overlays require little or no preoverlay repair, and are thus well suited to badly deteriorated concrete and asphalt-overlaid concrete pavements. An unbonded concrete overlay is an attractive alternative to reconstruction when construction duration is a pressing issue (e.g., for high traffic volumes and/or very poor subgrade conditions).

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[23] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, a jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[23] Low-severity transverse cracks are not considered structural distress in jointed reinforced concrete pavements.

As a general rule, a continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[23]

Limitations

An unbonded concrete overlay is usually not competitive with other overlay or nonoverlay rehabilitation alternatives when the pavement being rehabilitated has significant remaining structural life.

The 1993 AASHTO Guide[1] states that vertical clearance problems may be addressed by reducing the overlay thickness under bridges (although this may result in early failure at these locations), by raising the bridges, or be reconstructing the pavement under the bridges. Thicker overlays may also necessitate raising signs and guardrails, as well as increasing side slopes and extending culverts. Sufficient right-of-way must be available or obtainable to permit these activities.[23]

Unbonded concrete overlay may not be a feasible rehabilitation alternative in some situations where geometric restrictions are a major concern. This is especially true in urban areas. One State DOT recommends a preliminary investigation to determine the amount of pavement removal and undercutting necessary to meet at-grade bridges and provide clearance under overhead bridges. If the amount of pavement removal necessary exceeds about 40 percent, assuming none of the bridges are jacked, the alternative is eliminated from consideration.[7]

Concurrent Work

Unbonded concrete overlays require little or no preoverlay repair. The only repairs which may be needed are repair or replacement of severely shattered or rocking slabs, and repair of severe punchouts in continuously reinforced concrete pavement. Preoverlay repairs which are generally not considered warranted for this overlay type include load transfer restoration and full-depth repair of deteriorated joints and cracks.

Some literature on unbonded overlays suggests that removal of faulting in bare jointed concrete pavement, milling or leveling of severe rutting in asphalt-overlaid concrete pavement, and subdrainage improvement should also be considered as preoverlay preparations for an unbonded overlay. However, no performance studies have yet demonstrated that these activities improve unbonded overlay performance.

Materials

The preferred material for separating the old concrete pavement from the concrete overlay is at least 1 inch of asphalt concrete hot mix. In the case of the a concrete pavement with an existing asphalt overlay, the existing overlay, cold milled or left as is, can serve as the separation layer.

Other materials which have been used as separation layers under unbonded concrete overlays include open-graded aggregate (either unstabilized or stabilized), single- or double-layer polyethylene sheeting, chip seals, slurry seals, and wax-based curing compounds. None of these other materials is as effective a separation layer as asphalt concrete hot mix. In addition, thin separation layers require more preoverlay repair, which negates the main advantage of an unbonded overlay, i.e., the ability to rehabilitate a severely deteriored concrete pavement with little or no preoverlay repair.

Design

Traditionally, unbonded concrete overlays have been designed using some form of the familiar "square root" equation shown below:

$$h_{ol} = \sqrt{h_f^{\,2} - h_{eff}^{\,2}}$$

where

h_{ol}	=	unbonded overlay thickness
h_f	=	required slab thickness for future traffic
h_{eff}	=	effective thickness of existing slab

The square root equation dates back to the Bates Road Test[86] in the 1920's, and its use in unbonded overlay design procedures started in the 1940's. Full-scale field tests of concrete overlays conducted by the Corps of Engineers[87] in the 1940's and 50's indicated that the square root equation yielded conservative results.

Although many engineers have the impression that the square root equation (also called the Corps of Engineers equation) for unbonded overlay design is completely empirical, it has a theoretical basis. Several researchers have demonstrated that an overlay slab and a base slab can be represented by an equivalent single slab in a variety of ways. There are, however, some important limitations to the characterization of an unbonded overlay and base slab as an equivalent single slab. One limitation is that the Corps of Engineers equation is a simplified form of the equations for stress in either the base slab or overlay slab equivalent to stress in the equivalent single slab. This simplified equation is only valid when the two slabs are equal in thickness and equal in elastic modulus.

Another important limitation to characterizing an unbonded overlay slab and base slab in terms of an equivalent single slab assumes full contact between the overlay and base slabs. They may bend independently, but they must have the same radius of curvature. To whatever extent the overlay slab curls and/or warps to a different shape than the underlying slab, it will experience different and in some cases much grater stresses under combined load and curling than the equivalent thickness concept implies.

The third major limitation of the Corps of Engineers equation is the structural deficiency concept itself, namely, the assumption that an overlay satisfies a structural deficiency between a required single slab thickness and an existing slab's effective (i.e., damage-adjusted) thickness. As can be seen by examining the square root equation, the structural deficiency concept implies that for a given required slab thickness for future traffic, a thicker existing pavement will require a thinner unbonded overlay than a thinner existing pavement in the same condition. Conversely, it implies that a given thickness of unbonded overlay will perform better on a thicker existing pavement than on a thinner existing pavement in the same condition. Field observations do not support the implication that unbonded overlay performance is as sensitive to existing pavement thickness as the structural deficiency concept suggests.

One alternative to the Corps of Engineers equation for design of an unbonded overlay is to design the overlay as if it were a new pavement, with the existing pavement structure characterized as a foundation for the new slab. The elastic modulus, modulus of rupture, and load transfer coefficient inputs to the design model are typically the anticipated values for the overlay slab.

Two key differences exist between this approach and the Corps of Engineers approach. The first difference is that the existing pavement is not considered to contribute any structural capacity to the total structural capacity of the overlaid pavement. The existing pavement is instead considered a foundation for the new slab. This leads to the second major difference between the two methods. The k-value of the foundation beneath the existing pavement is used to determine the required future slab thickness in the Corps of Engineers method, whereas the new pavement design method requires a k-value beneath the overlay. The major difficulty in application of the new design approach thus lies in selection of an appropriate design k-value.

Conventional practice in concrete pavement design for many years has been to assign a k-value to a granular or stabilized base which was considerably higher than the k-value of the subgrade, and which was a function of the thickness and stiffness of the base layer. This convention is still employed for new concrete pavements in the 1993 AASHTO Guide and Portland Cement Association design procedures. Following this logic, an existing concrete pavement with an asphalt concrete surfacing for a separation layer would be assigned a very high k-value, such as 500 psi/in or more for unbonded overlay design.

However, backcalculation results indicate that when an unbonded overlay is designed as a new pavement with the existing pavement as its foundation, it is neither necessary nor appropriate to use an extremely high k-value such as 500 psi/inch or more. A design static k-value in the range of 200 to 400 psi/inch is probably appropriate in most cases. Whenever possible, deflections should be measured on the existing pavement prior to overlay to backcalculate a dynamic k-value for the existing foundation, and to estimate from this a reasonable static k value for design.

Another issue which should be considered is the effect of curling on performance. If a jointed overlay slab is designed as a new pavement with the existing pavemet serving as its foundation, it will experience much higher curling stresses than a conventional concrete pavement on a weaker foundation. These higher curling stresses may be computed using finite element analysis or available equations. However, if the performance model used to determine the required slab thickness was developed for concrete pavements on weak foundations, the detrimental effect of high curling stress will not be adequately reflected in the predicted performance of the overlay. This would be the case if, for example, the 1993 AASHTO design procedure were use to determine the required slab thickness, rather than a fatigue analysis which directly considered the combined effects of load and curling. Either increased slab thickness or reduced joint spacing may be necessary to achieve the performance from the unbonded overlay that is predicted by the model.

Other alternatives to unbonded overlay design involve modeling the overlay and existing slab as either two elastic layers or two plates on a foundation. This is arguably the most realistic of the three design approaches described here, but also the most difficult. [88] The basic approach is the same as for design of the overlay as a new pavement, except that the existing pavement structure is characterized more realistically not as a uniform foundation but as a multilayered system. Among the difficulties associated with this approach are the following:

- Characterization of the existing slab, including deciding how (it at all) to account for existing deterioration;
- Identifying the important structural responses (e.g., overlay stress, overlay deflection, original slab stress, etc.); and
- Identifying the important performance criteria (e.g., cracking in the original slab and/or cracking in the overlay slab).

Construction

After placement of the leveling course, construction of an unbonded concrete overlay is essentially the same as construction of a new concrete pavement. Particular construction issues related to unbonded overlay construction include cooling or whitewashing the asphalt separation layer, timing the sawing of joints, mismatching the joints, and constructing transitions to existing or reconstructed pavement sections.

When paving on hot, sunny days, it is sometimes considered advisable to cool the asphalt surface prior to placement of the concrete overlay. One technique used for this purpose in the past was whitewashing with a white-pigmented curing compound or lime slurry.[41] Whitewashing is recommended when the asphalt temperature is expected to exceed 110°F during paving. This is more likely to be a concern when the concrete overlay is to be placed on a new asphalt surface, i.e., a leveling course.

Another technique for cooling the asphalt surface is fogging with water. This technique is considered adequate when placing the concrete overlay on an old or milled asphalt surface. Water fogging is recommended when the asphalt surface is sufficiently hot that it is uncomfortable to touch with an open palm.

Dowel bar assemblies and reinforcing steel, if used, are positioned and firmly anchored on the separation layer prior to paving of the concrete overlay. After slipform placement of the concrete overlay on the separation layer, the concrete is spread, consolidated, finished, textured, and cured. Materials commonly used for curing are white-pigmented liquid membrane curing compunds, waterproof paper or polyethylene sheets, and wet cotton mats or burlap.

Timing of the sawcutting operation similar to that for new concrete construction on a high-friction base, depending also on the weather conditions and mix design. To avoid premature cracking, lightweight (Soff-Cut) saws may be used to saw the transverse and longitudinal joints early. Joint sawing should begin as soon as the partially hardened concrete can support the weight of the lightweight saw without causing damage to the surface or ravelling of the joint. Secondary sawing for joint sealing purposes, if desired, may be done later using conventional saws.

In jointed unbonded concrete overlays, the joints should be spaced more closely than they would be in a new pavement on a granular base, and the overlay´s transverse joints and the old pavement´s transverse joints should be mismatched to improve load transfer across the overlay joints. Mismatching the joints by at least 1 foot is advisable; several States specify 3 feet.

Dowels are not considered necessary for jointed unbonded overlays less than 8 inches thick. For overlays 8 to 9 inches thick, 1.25-inch-diameter dowels are recommended, and for overlays greater than 9 inches thick, 1.5-inch-diameter dowels are recommended. [89]

Guidelines for constructing transitions between unbonded concrete overlays and existing or reconstructed pavement sections are given in the American Concrete Pavement Association's *Guidelines for Unbonded Concrete Overlays.*[89]

Performance

The typical range of service life for unbonded concrete overlay of concrete pavement is the same as for reconstruction in concrete, i.e., 20 to 30 years. The American Concrete Pavement Association estimates a service life of 30 years or more, depending on design, for an unbonded concrete overlay.[6] West Virginia estimates a service life of 20 years for unbonded concrete overlay.[34]

Additional information on the design and performance of unbonded concrete overlays is provided in and the American Concrete Pavement Association's *Guidelines for Unbonded Concrete Overlays,*[89] the Portland Cement Association's *Guide to Concrete Resurfacing Designs and Selection Criteria*,[90] NCHRP Synthesis 99, *Resurfacing with Portland Cement Concrete*,[91] NCHRP Synthesis No. 204, *Portland Cement Concrete Resurfacing*,[92] and NCHRP Report No. 415, *Evaluation of Unbonded Portland Cement Concrete Overlays.*[93]

References for Appendix B

[1] American Association of State Highway and Transportation Officials, *Guide for Design of Pavement Structures*, Washington, D. C., 1993.

[2] Asphalt Institute, *Asphalt Overlays for Highway and Street Rehabilitation*, Manual Series No. MS-17, Lexington, Kentucky, 1999.

[3] Asphalt Institute, *Asphalt Technology and Construction Practices*, Educational Series ES-1, Lexington, Kentucky, 1983.

[4] National Highway Institute, *Techniques for Pavement Rehabilitation*, sixth edition, 1998.

[5] Wilson, T. P., et al., "Materials and Procedures for Repair of Potholes in Asphalt Pavements: Manual of Practice," *Asphalt Pavement Repair Manuals of Practice*, Strategic Highway Research Program Report No. SHRP-H-348, National Research Council, Washington, D. C., 1993.

[6] American Concrete Pavement Association, *The Concrete Pavement Restoration Guide – Procedures for Preserving Concrete Pavements*, Technical Bulletin, Skokie, Illinois.

[7] Ohio Department of Transportation, *Pavement Design and Rehabilitation Manual*, Columbus, Ohio, June 1999.

[8] Pyrotech Asphalt Equipment Manufacturing Company, *Hot In-Place Recycling – A Technical Training Seminar and Workshop*, British Colombia.

[9] Applied Research Associates, Inc., *Pavement Recycling Guidelines for Local Governments – Reference Manual*, Federal Highway Administration Report No. FHWA-TS-87-230, 1987.

[10] AGRA Earth & Environmental Limited, *Development of Guidelines for the Design of Hot In-Place Recycled Asphalt Concrete Mixtures*, Edmonton, Alberta, 1996.

[11] Button, J. W., Little, D. N., and Estakhri, C. H., *Hot In-Place Recycling of Asphalt Concrete*, NCHRP Synthesis No. 193, Transportation Research Board, National Research Council, Washington, D. C., 1994.

[12] Epps, J. A, Little, D. N., Holmgreen, R. J., and Terrel, R. L., *Guidelines for Recycling Pavement Materials*, NCHRP Report No. 224, Transportation Research Board, National Research Council, Washington, D. C., 1980.

[13] Asphalt Recycling and Reclaiming Association, “Hot In-Place Recycling – First in the Line of Pavement Maintenance,” Hot In-Place Recycling Technical Committee, Annapolis, Maryland, 1992.

[14] New York State Department of Transportation, *Pavement Rehabilitation Manual, Volume II: Treatment Selection*, Materials Bureau, Albany, New York, 1993.

[15] Vermont Agency of Transportation, *Pavement Rehabilitation Design Procedures*, Montpelier, Vermont, 1999.

[16] Federal Highway Administration, Pavement Recycling Guidelines for State and Local Governments, Publication No. FHWA-SA-8-042, 1997.

[17] Epps, J. A., *Cold-Recycled Bituminous Concrete Using Bituminous Materials*, National Cooperative Highway Research Synthesis 160, 1990.

[18] Asphalt Pavement Recycling and Reclaiming Methods for Asphalt Pavement Rehabilitation, Annapolis, Maryland, 1992.

[19] Transportation Research Board, Procedings of the National Seminar on Asphalt Pavement Recycling,Transportation Research Record No. 780, National Research Council, Washington, D.C., 1980.

[20] World Highways/Routes du Monde, “WAM Cuts Emissions,” November-December 2000.

[21] Asphalt Reclaiming and Recycling Association, An Overview of Recycling and Reclaiming Methods for Asphalt Pavement Rehabilitation, Annapolis, Maryland, 1992.

[22] The Asphalt Institute, *Asphalt Cold-Mix Recycling*, Manual Series MS-20, second edtion, 1986.

[23] Hall, K. T. and Darter, M. I., *Structural Evaluation Strategies for Jointed Concrete Pavements, Volume IV – Guidelines for the Selection of Rehabilitation Alternatives*, Federal Highway Administration Report No. FHWA-RD-89-14, 1990.

[24] Arizona Department of Transportation, *Preliminary Engineering and Design Manual*, Phoenix, Arizona, 1991.

[25] Monismith, C. L., "Analytically Based Asphalt Pavement Design and Rehabilitation: Theory to Practice, 1962 – 1992," *Transportation Research Record* No. 1354, 1992.

[26] Shell International Petroleum Company, *Shell Pavement Design Manual*, London, England, 1978.

[27] Shook, J. F., Finn, F. N., Witczak, M. W., and Monismith, C. L., "Thickness Design of Asphalt Pavements – The Asphalt Institute Method," *Proceedings of the Fifth International Conference on the Structural Design of Asphalt Pavements*, University of Michigan and Delft University of Technology, 1982.

[28] Asphalt Institute, *Thickness Design Manual (MS-1)*, ninth edition, College Park, MD, 1981.

[29] Thompson, M. R. and Barenberg, E. J., *Calibrated Mechanistic Structural Analysis Procedures for Pavements*, Phase I Final Report, NCHRP Project 1-26, 1989.

[30] Mahoney, J., Lee, S. W., Jackson, N., and Newcomb, D., *Mechanistic-Based Overlay Design Procedures for Washington State Flexible Pavements*, Washington State DOT Report No. WA-RD-170.1, 1989.

[31] Nevada Department of Transportation, *Pavement Structural Design and Policy Manual*, Carson City, Nevada, 1996.

[32] Sebaaly, P. E., Hand, A., Epps, J., and Bosch, C., "Nevada's Approach to Pavement Management," *Transportation Research Record* No. 1524, 1996.

[33] Lytton, R. L., Germann, F., and Chou, Y. J., *Determination of Asphalt Concrete Pavement Structural Properties by Nondestructive Testing*, NCHRP Report No. 327, 1990.

[34] West Virginia Department of Transportation, "Pavement Design Selection Guide," Memorandum DD-132-1, Division of Highways, Charleston, West Virginia, 1994.

[35] Wisconsin Department of Transportation, *Maintenance and Rehabilitation Committee Final Report (Pavement Preservation Strategy)*, Madison, Wisconsin, 1999.

[36] Indiana Department of Transportation, "Pavement Design," Chapter 52 of *Indiana DOT Design Manual*, Indianapolis, Indiana, 1998.

[37] American Concrete Pavement Association, *Whitetopping – State of the Practice*, Engineering Bulletin EB 210P, Skokie, Illinois.

[38] Speakman, J., Scott, N., III, "Ultra-Thin, Fiber-Reinforced Overlays for Urban Intersections," *Transportation Research Record* No. 1532, Transportation Research Board, National Research Council, Washington, D. C., 1996.

[39] American Concrete Pavement Association, *Fast Track Concrete Pavements*, Technical Bulletin TB004P, Skokie, Illinois.

[40] Mack, J. W., Wu, C.L., Tarr, S., and Refai, T., "Model Development and Interim Design Procedure Guidelines for Ultrathin Whitetopping Pavements," *Proceedings of the Sixth International Conference on Concrete Pavement Design and Materials for High Performance*, Purdue University, West Lafayette, Indiana, 1997.

[41] American Association of State Highway and Transportation Officials*, Guide Specifications for Concrete Overlays of Pavements and Bridge Decks*, Washington, D. C., 1990.

[42] Armaghani, J. M. and Tu, D., "Rehabilitation of Ellaville Weigh Station with Ultrathin Whitetopping," *Transportation Research Record* No. 1654, Transportation Research Board, National Research Council, Washington, D. C., 1999.

[43] American Concrete Pavement Association, *Guidelines for Full-Depth Repair*, Technical Bulletin TB002.02P, Skokie, Illinois.

[44] Yu, H. T., et al., *Concrete Rehabilitation – Users Manual*, Strategic Highway Research Report No. SHRP-C-412, 1994.

[45] Darter, M. I., “Patching of Continuously Reinforced Concrete Pavements,” *Transportation Research Record* No. 800, 1981.

[46] Darter, M. I., Barenberg, E. J., and Yrjanson, W. R., *Joint Repair Methods for Portland Cement Concrete Pavements*, NCHRP Report No. 281, 1985.

[47] Snyder, M. B., Reiter, M. J., Hall, K. T., and Darter, M. I., *Rehabilitation of Concrete Pavements, Volume 1 – Repair Rehabilitation Techniques*, Federal Highway Administration Report No. FHWA/RD-88/071, 1989.

[48] Patel, A. J., C. A. G. Mojab, and A. R. Romine, *Materials and Procedures for Rapid Repair of Partial-Depth Spalls in Concrete Pavements – Manual of Practice*, Strategic Highway Research Report No. SHRP-H-349, National Research Council, Washington, DC, 1993.

[49] Evans, L. D. and A. R. Romine, *Materials and Procedures for the Repair of Joint Seals in Concrete Pavements – Manual of Practice*, Strategic Highway Research Report No. SHRP-H-349, National Research Council, Washington, DC, 1993.

[50] Jerzak, H., *Rapid Set Materials for Repairs to Portland Cement Concrete Pavement and Structures*, California Department of Transportation, 1994.

[51] Waterways Experiment Station, U.S. Army Corps of Engineers, *REMR Notebook, (CS-MR-7.3, Rapid-Hardening Cements and Patching Material, CM-PC-2.2, Fast-Setting Patching Materials: Pavement Blended Cements; CM-PC-2.4, Fast Setting Patch Materials: Bonsal Rapid Patch; CM-PC-2.5, Rapid-Setting Patching Materials: Rapid Set Concrete Mix).*

[52] American Concrete Pavement Association, *Guidelines for Partial-Depth Spall Repair*, Technical Bulletin TB003P, Skokie, IL, 1998.

[53] National Cooperative Highway Research Program, *Rapid Setting Materials for Patching of Concrete*, Synthesis of Highway Practice No. 45, Transportation Research Board, National Research Council, Washington, DC, 1977.

[54] Smith, K. L. et al., *Innovative Materials and Equipment for Pavement Surface Repairs*, Volumes I and II, Strategic Highway Research Report No. SHRP-M/UFR-91-504, National Research Council, Washington, DC, 1991.

[55] American Society for Testing and Materials, *Standard Specification for Expansive Hydraulic Cement*, ASTM C 845, Philadelphia, PA.

[56] American Association of State Highway and Transportation Officials, *Guide Specifications for Highway Construction,* Washington, DC, 1988.

[57] Federal Highway Administration, *Field Inspection Guide for Restoration of Jointed Concrete Pavements*, Demonstration Projects Program, 1987.

[58] Whiting, D., et al., *Synthesis of Current and Projected Highway Technology*, Strategic Highway Research Program, Report No. SHRP-C-345, National Research Council, Washington, DC, 1993.

[59] Utah Department of Transportation, *Pavement Management and Pavement Design Manual*, Salt Lake City, Utah, 1998.

[60] American Concrete Pavement Association, *Slab Stabilization Guidelines for Concrete Pavements*, Technical Bulletin TB 018P, Skokie, Illinois, 1994.

[61] Federal Highway Administration, *Construction Handbook on PCC Pavement Rehabilitation*, Washington, D. C., 1984.

[62] Taha, R., Salim, A., Hasan, S., and Lunde, B., "Evaluation of Highway Undersealing Practices of PCC Pavements," Annual Meeting of the Transportation Research Board, Washington, D.C., 1994.

[63] Crovetti, J. A. and Darter, M. I., "Void Detection for Jointed Concrete Pavements*,"* *Transportation Research Record* No. 1041, 1985.

[64] Crovetti, J. A., *Design and Evaluation of Jointed Concrete Pavement Systems Incorporating Free Draining Base Layers*, Ph.D. thesis, University of Illinois at Urbana-Champaign, 1994.

[65] Crovetti, J. A. and Crovetti, M. R. T., "Evaluation of Support Conditions Under Jointed Concrete Pavement Slabs," *Nondestructive Testing of Pavements and Backcalculation of Moduli*, ASTM STP 1198, H. L. Von Quintus, A. J. Bush III, and G. Y Biladi, editors, 1994.

[66] Clemena, G. G., Sprinkel, M. M., and Long. R. R., “Use of Ground-Penetrating Radar for Detecting Voids Under Jointed Concrete Pavements,” *Transportation Research Record* No. 1109, 1987.

[67] Chapin, L. T. and White, T. D., “Validating Loss of Support for Concrete Pavements,” Proceedings, *Proceedings of the Fifth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, West Lafayette, Indiana, 1993.

[68] Gulden, W. and Brown, D., *Improving Load Transfer in Existing Jointed Concrete Pavements*, Federal Highway Administration Report No. FHWA/RD-82/154, 1987.

[69] Hall, K. T., Darter, M. I., and Armaghani, J. M., “Performance Monitoring of Joint Load Transfer Restoration,” *Transportation Research Record* No. 1388, 1993.

[70] Pierce, L. M., *Dowel Bar Retrofit in Washington State – Summary of Findings*, Washington State Department of Transportation, 1999.

[71] Pierce, L. M., “Portland Cement Concrete Pavement Rehabilitation in Washington: Case Study,” *Transportation Research Record* No. 1449, 1994.

[72] Federal Highway Administration, “Design Review Report on Retrofit Dowels,” Puerto Rico Division, 1991.

[73] Federal Highway Administration, *Concrete Pavement Restoration Performance Review*, Pavements Division and Demonstration Projects Division, 1987.

[74] International Grinding and Grooving Association, “A Level Road Rides Better, Lasts Longer,” 1989.

[75] ERES Consultants, Inc., *The Longevity and Performance of Diamond-Ground Pavements*, for American Concrete Pavement Association, 1998.

[76] Chou, Y. T., “Asphalt Overlay Design for Airfield Pavements, *Proceedings of the Association of Asphalt Paving Technologists*, Volume 53, April 1984.

[77] Mellinger, F. M. and Sale, J. P., "The Design of Non-Rigid Overlays for Concrete Airfield Pavements," *Journal of the Air Transport Division*, American Society of Civil Engineers, Volume 82, Number AT 2, May 1956.

[78] Hall, K. T., *Performance, Evaluation, and Rehabilitation of Asphalt-Overlaid Concrete Pavements*, Ph.D. thesis, University of Illinois at Urbana-Champaign, 1991.

[79] Hall, K. T. and Darter, M. I., "Rehabilitation Performance and Cost Effectiveness: A Ten-Year Case Study," *Transportation Research Record* No. 1215, 1990.

[80] Hall, K. T., Darter, M. I., and W. M. Rexroad, *Performance of Bare and Resurfaced JRCP and CRCP on the Illinois Interstate Highway System – 1991 Update*, Illinois Highway Research Report No. 532-1, Federal Highway Administration Report No. FHWA-IL-UI-244, University of Illinois and Illinois Department of Transportation, 1993.

[81] National Asphalt Pavement Association, *Guidelines for the Use of HMA Overlays to Rehabilitate PCC Pavements*, Information Series No. 117, Lanham, Maryland, 1994.

[82] Hall, K. T., *Structural Coefficients for Fractured Concrete Slabs*, for American Concrete Pavement Association, 1999.

[83] Thompson, M. R., "Hot-Mix Asphalt Overlay Design Concepts for Rubblized Portland Cement Concrete Pavements," *Transportation Research Record* No. 1684, 1999.

[84] Thompson, M. R., *Breaking/Cracking and Seating Concrete Pavements*, NCHRP Synthesis No. 144, 1989.

[85] American Concrete Pavement Association, *Guidelines for Bonded Concrete Overlays*, Technical Bulletin TB-007P, Arlington Heights, Illinois, 1990.

[86] Older, C., "Highway Research in Illinois," *Transactions of the American Society of Civil Engineers*, Volume 87, 1924.

[87] Mellinger, F. M., "Structural Design of Concrete Overlays," *Journal of the American Concrete Institute*, Volume 60, No. 2, February 1963.

[88] Hall, K. T., Darter, M. I., and Seiler, W. J., "Improved Design of Unbonded Concrete Overlays," *Proceedings of the Fifth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, West Lafayette, Indiana, 1993.

[89] American Concrete Pavement Association, *Guidelines for Unbonded Concrete Overlays*, Technical Bulletin TB-005.0D, Skokie, Illinois, 1990.

[90] Portland Cement Association, *Guide to Concrete Resurfacing Designs and Selection Criteria*, Engineering Bulletin EB087.01P,1981.

[91] Hutchinson, R. L., *Resurfacing with Portland Cement Concrete*, NCHRP Synthesis No. 99, 1982.

[92] McGhee, K. H., *Portland Cement Concrete Resurfacing*, NCHRP Synthesis No 204, 1994.

[93] ERES Consultants, Inc., *Evaluation of Unbonded Portland Cement Concrete Overlays*, NCHRP Report No. 415, 1999.