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# Structural Design of Interlocking Concrete Pavement for Roads and Parking Lots

## History

The concept of interlocking concrete pavement dates back to the roads of the Roman Empire. They were constructed with tightly-fitted stone paving units set on a compacted aggregate base. The modern version, concrete pavers, is manufactured with close tolerances to help ensure interlock. Concrete pavers were developed in the Netherlands in the late 1940's as a replacement for clay brick streets. A strong, millennia-old tradition of segmental paving in Europe enabled interlocking concrete pavement to spread quickly. It is now established as a conventional means of paving there with some three billion ft$^2$ (300 million m$^2$) installed annually. Concrete pavers came to North America in the 1970's. They have been used successfully in numerous residential, commercial, municipal, port and airport applications.

When considering design and construction, three types of interlock must be achieved: vertical, rotational and horizontal interlock.

*Figure 1. The Roman Appian Way: early interlocking pavement*

## Advantages

The paving system offers the advantages of concrete materials and flexible asphalt pavement. As high-strength concrete, the units have high resistance to freeze-thaw cycles and deicing salts, high abrasion and skid resistance, no damage from petroleum products or indentations from high temperatures. Once installed, there is no waiting time for curing. The pavement is immediately ready for traffic. Stress cracking and degradation of the surface is minimized because the numerous joints, or intentional "cracks," act as the means for load transfer. Like flexible asphalt pavement, an aggregate base accommodates minor settlement without surface cracking. An aggregate base facilitates fast construction, as well as access to underground utilities. Mechanical installation of concrete pavers can further shorten construction time. Pavement reinstatement is enhanced by reusable paving units, thereby reducing waste materials.

## The Principle of Interlock

Interlock is the inability of a paver to move independently from its neighbors. It is critical to the structural performance of interlocking concrete pavement. When considering design and construction, three types of interlock must be achieved: vertical, rotational, and horizontal interlock. These are illustrated in Figure 2. Vertical interlock is achieved by the shear transfer of loads to surrounding units through sand in the joints. Rotational interlock is maintained by the pavers being of sufficient thickness, placed closely together, and restrained by a curb from lateral forces of vehicle tires. Rotational interlock can be further enhanced if there is a slight crown to the pavement cross section. Besides facilitating drainage, the crown enables the units to tighten slightly, progressively stiffening through loads and minor settlement across the entire pavement, thereby increasing structural capacity. When progressive stiffening has stablized, the pavement has reached a state of "lockup."

Horizontal interlock is primarily achieved through the use of laying patterns that disperse forces from

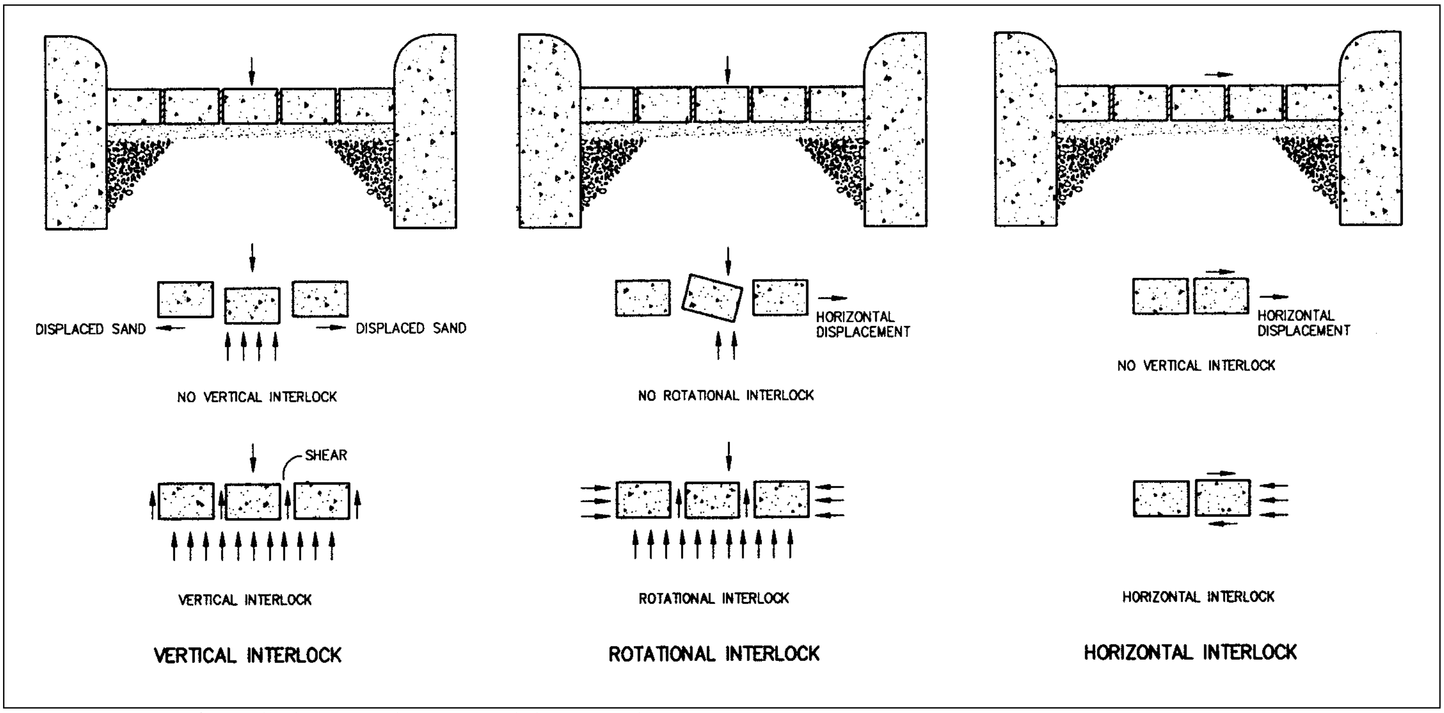


*Figure 2. Types of interlock: vertical, rotational, horizontal*

braking, turning and accelerating vehicles. Herringbone patterns are the most effective laying patterns for maintaining interlock. Testing has shown that these patterns offer greater structural capacity and resistance to lateral movement than other laying patterns (1, 2, 3). Therefore, herringbone patterns are recommended in areas subject to vehicular traffic. See Figure 3. Stable edge restraints such as curbs are essential. They maintain horizontal interlock while the units are subject to repeated lateral loads from vehicle tires. ICPI Tech Spec 3, *Edge Restraints for Interlocking Concrete Pavements* offers guidance on the selection and detailing of edge restraints for a range of applications.

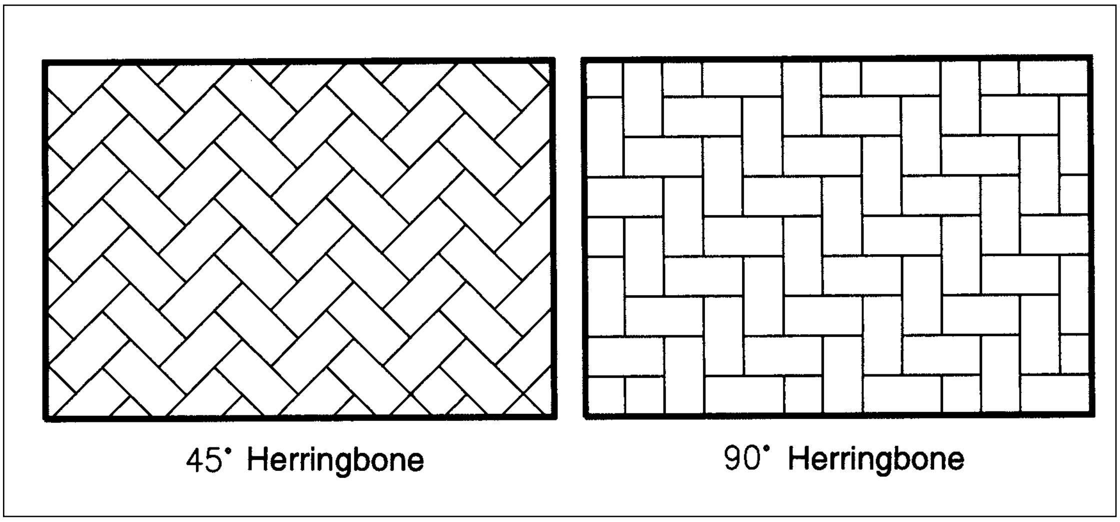
## Typical Pavement Design and Construction

Figure 4 illustrates typical schematic cross sections for interlocking concrete pavement. Both the base and subbase are compacted aggregate. Many pavements for city and residential uses do not require an aggregate subbase except for very heavy use or over a weak soil subgrade. In these situations it may be more economical to use asphalt or cement-stabilized base layers. They are often placed over a subbase layer of unbound compacted aggregate.

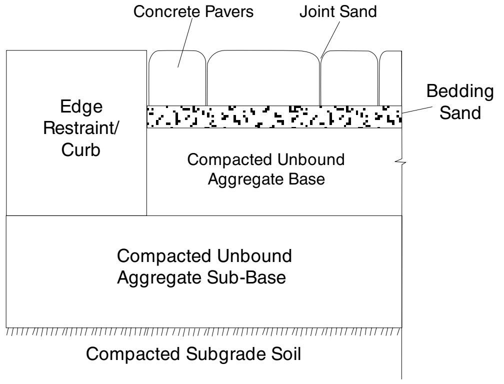
Construction is covered in ICPI Tech Spec 2, *Construction of Interlocking Concrete Pavement*. The steps for preparing the soil subgrade and base materials are similar to those required for flexible asphalt pavements. After the base surface is built to specified elevations and surface tolerances, bedding sand is screeded in an even layer, typically 1 in. (25 mm) thick. The units are placed, manually or mechanically, on the smooth bedding sand constrained by stationary edge restraints.

The pavers are vibrated with a high frequency plate compactor. This action forces sand into the bottom of

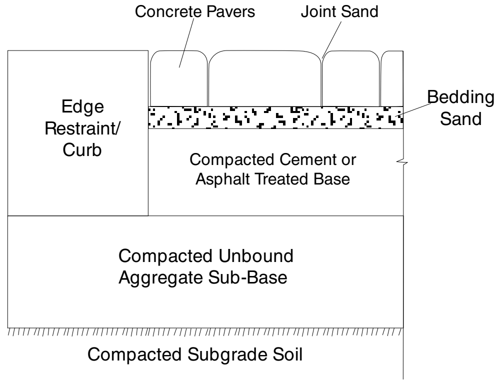
*Figure 3. Laying patterns for vehicular traffic*


45° Herringbone

90° Herringbone



**(A) Unbound Base/Subbase**



**(B) Cement or Asphalt Treated Base**

*Figure 4. Typical schematic cross sections*

the joints of the pavers and begins compaction of the bedding sand. Sand is then spread and swept into the joints, and the pavers are compacted again until the joints are filled. Complete compaction of the sand and slight settlement of the pavers tightens them. During compaction, the pavement is transformed from a loose collection of pavers to an interlocking system capable of spreading vertical loads horizontally. This occurs through shear forces in the joints.

## Structural Design Procedure

The load distribution and failure modes of flexible asphalt and interlocking concrete pavement are very similar: permanent deformation from repetitive loads. Since failure modes are similar, a simplified procedure of the method is adapted from Reference 4 and the American Association of State Highway and Transportation Officials (AASHTO) 1986 and 1993 *Guide for Design of Pavement Structures (5)*. The following structural design procedure is for roads and parking lots. Base design for crosswalks should consider using stablized aggregate or cast-in-place concrete. Stiffer bases will compensate for stress concentration on the subgrade and base where the pavers meet adjoining pavement materials. Design for heavy duty pavements such as port and airport pavements is covered in ICPI manuals entitled, *Port and Industrial Pavement Design for Concrete Pavers* and *Airfield Pavement Design with Concrete Pavers*.

## Design Considerations

The evaluation of four factors and their interactive effects will determine the final pavement thickness and material. These include environment, traffic, subgrade soil strength and pavement materials. The design engineer selects values representing attributes of these factors. The values can be very approximate correlations and qualitative assumptions. Each factor, however, can be measured accurately with detailed engineering studies and extensive laboratory testing. As more detailed information is obtained about each factor, the reliability of the design will increase.

The effort and cost in obtaining information about each should be consistent with the importance of the pavement. A major thoroughfare should receive more analysis of the soil subgrade and traffic mix than a residential street. Furthermore, the degree of analysis and engineering should increase as the subgrade strength decreases and as the anticipated traffic level increases. In other words, pavements for high volume traffic over weak soils should have the highest degree of analysis for each factor as is practical.

**Environment**—Moisture and temperature significantly affect pavement. As moisture in the soil or base increases, the load bearing capacity of the soil or the strength of the base decreases. Moisture causes differential heaving and swelling of certain soils, as well. Temperature can affect the load bearing capacity of pavements, particularly asphalt stabilized layers. The combined effect of freezing temperatures and moisture can lead to two detrimental effects. First, expansion of the water during freezing can cause the pavement to heave. Second, the strength of the pavement materials can be reduced by thawing.

These detrimental effects can be reduced or eliminated in one of three ways. Moisture can be kept from entering the pavement base and soil. Moisture can be removed before it has a chance to weaken the pavement. Pavement materials can be used to resist moisture and movement from swelling or frost. Limited construction budgets often do not allow complete protection against the effects of moisture and freeze-thaw. Consequently, their effects should be mitigated to the highest extent allowed by the available budget and materials.

In this design procedure, the effects of moisture and frost are part of characterizing of the strength of subgrade soil and pavement materials. Subjective descriptions of drainage quality and moisture conditions influence design strength values for subgrade soils and unbound granular materials. In addition, if freeze-thaw exists, then soil subgrade strength is reduced according to the degree of its frost susceptibility.

**Traffic**—When pavement is trafficked, it receives wear or damage. The amount of damage depends

### TABLE 1
### Axle Load Damage Factors

| Single Axle Kips (kN) | Single Axle Damage Factor | Tandem Axle Kips (kN) | Tandem Axle Damage Factor |
|---|---|---|---|
| 2 (9) | 0.0002 | 10 (44) | 0.01 |
| 6 (27) | 0.01 | 14 (62) | 0.03 |
| 10 (44) | 0.08 | 18 (80) | 0.08 |
| 14 (62) | 0.34 | 22 (98) | 0.17 |
| 18 (80) | 1.00 | 26 (115) | 0.34 |
| 22 (98) | 2.44 | 30 (133) | 0.63 |
| 26 (115) | 5.21 | 34 (157) | 1.07 |
| 30 (133) | 10.03 | 38 (169) | 1.75 |
| 34 (157) | 17.87 | 42 (186) | 2.75 |
| 38 (169) | 29.95 | 46 (204) | 4.11 |

### TABLE 2
### Typical Design EALs

| Road Class | EALs* millions | Reliability Factor | Design EALs* millions |
|---|---|---|---|
| Arterial or Major Streets | | | |
| Urban | 7.5 | 3.775 | 28.4 |
| Rural | 3.6 | 2.929 | 10.6 |
| Major Collectors | | | |
| Urban | 2.8 | 2.929 | 8.3 |
| Rural | 1.5 | 2.390 | 3.5 |
| Minor Collectors | | | |
| Urban | 1.3 | 2.390 | 3.0 |
| Rural | 0.55 | 2.390 | 1.3 |
| Commercial/Multi-Family Locals | | | |
| Urban | 0.43 | 2.010 | 0.84 |
| Rural | 0.28 | 2.010 | 0.54 |

**Assume a 20 year design life.*

on the weight of the vehicles and the number of expected passes over a given period of time. The period of time, or design life, is usually 20 years. Predicted traffic over the life of the pavement is an estimate of various vehicle loads, axle and wheel configurations, and the number of loads. The actual amount of traffic loads can often exceed the predicted loads. Therefore, engineering judgement is required in estimating expected sources of traffic and loads well into the future.

Damage to pavement results from a multitude of axle loads from cars, vans, light trucks, buses and tractor-trailers. In order to more easily predict the damage, all of the various axle loads are expressed as damage from an equivalent standard axle load. In other words, the combined damaging effects of various axle loads are equated to the damaging effect of 18-kip (80 kN) equivalent single axle load (ESALs or EALs) repetitions. Damage factors for other axle loads are shown in Table 1. For example, the table shows that a single axle load of 38-kip (169 kN) would cause the same pavement damage as approximately 30 passes of an 18-kip (80 kN) single axle.

For pavements carrying many different kinds of vehicles, greater study is needed to obtain the expected distribution of axle loads within the design period. If no detailed traffic information is available, Table 2 can be used for general guidance by listing typical EALs as a function of road class. EALs in Table 2 can be converted to TI or Traffic Index used by Caltrans in California to characterize axle loads. The following formula converts 18-kip (80kN) equivalent single axle loads (ESALs) to a TI: $TI = 9.0 \times (ESAL/10^6)^{0.119}$. Table 7 correlates ESALs used in Figures 5, 6 and 7 to TIs.

In some situations, the designer cannot know the expected traffic in five, ten or fifteen years into the future. Therefore, the reliability (degree of conservatism) of the engineer's predictions can be modified as follows:

Adjusted EALs = $F_R$ x EALs (estimated or from Table 2) where $F_R$ is the reliability factor. Recommended reliability factors by road class are also given in Table 2, along with the corresponding adjusted EALs and TIs for use in the design.

In some residential development projects, interlocking concrete pavement streets are constructed first and then housing is built. Axle loads from construction-related truck traffic should be factored into the base thickness design. The loads can be substantial compared to the lighter loads from automobiles after construction is complete.

**Soil Subgrade Support** – The strength of the soil subgrade has the greatest effect on determining the total thickness of the interlocking concrete pavement. When feasible, resilient modulus ($M_r$), R-value, or soaked California Bearing Ratio (CBR) laboratory tests should be conducted on the typical subgrade soil to evaluate its strength. These tests should be conducted at the most probable field conditions of density and moisture that will be anticipated during the design life of the pavement. $M_r$ tests are described in AASHTO T-307 (7); R-value in ASTM D 2844 (6) or AASHTO T-190 (7); and CBR in ASTM D 1883 (6) or AASHTO T-193 (7). CBR and R-values are correlated in Reference 9.

In the absence of laboratory tests, typical resilient modulus ($M_r$) values have been assigned to each soil type defined in the United Soil Classification System (USCS), per ASTM D 2487 (6), or AASHTO soil classification systems (see Tables 3 and 4). Three modulus values are provided for each USCS or AASHTO soil type, depending on the anticipated environmental and drainage conditions at the site. Table 3 includes formulas that explain the approximate relationship between $M_r$ and CBR, plus $M_r$ and R-value from Reference 9.

Guidelines for selecting the appropriate $M_r$ value are summarized in Table 5. Each soil type in Tables 3 and 4 has also been assigned a reduced $M_r$ value (far right column) for use *only* when frost action is a design consideration.

Compaction of the subgrade soil during construction should be at least 98% of AASHTO T-99 or ASTM D 698 for cohesive (clay) soils and at least 98% of AASHTO T-180 or ASTM D 1557 for cohesionless (sandy and gravelly) soils. The higher compaction standards described in T-180 or D 1557 are preferred. The effective depth of compaction for all cases should be at least the top 12 inches (300 mm). Soils having an $M_r$ of 4,500 psi (31 MPa) or less (CBR 3% or less/R-value 8 or less) should be evaluated for either replacement with a higher bearing strength material, installation of an aggregate subbase capping layer, improvement by stabilization or use of geotextiles.

### TABLE 3
### Subgrade Resilient Modulus ($M_r$) as a Function of USCS Soil Type

$10^3$ psi = 6.94 MPa

| USCS Soil Group | Resilient Modulus ($10^3$ psi) Drainage Option 1 | Drainage Option 2 | Drainage Option 3 | Reduced Modulus* ($10^3$ psi) |
|---|---|---|---|---|
| GW, GP, SW, SP | 20.0 | 20.0 | 20.0 | N/A |
| GW-GM, GW-GC, GP-GM, GP-GC | 20.0 | 20.0 | 20.0 | 12.0 |
| GM, GM-GC, GC | 20.0 | 20.0 | 20.0 | 4.5 |
| SW-SM, SW-SC, SP-SM | 20.0 | 20.0 | 20.0 | 9.0 |
| SP-SC | 17.5 | 20.0 | 20.0 | 9.0 |
| SM, SM-SC | 20.0 | 20.0 | 20.0 | 4.5 |
| SC | 15.0 | 20.0 | 20.0 | 4.5 |
| ML, ML-CL, CL | 7.5 | 15.0 | 20.0 | 4.5 |
| MH | 6.0 | 9.0 | 12.0 | 4.5 |
| CH | 4.5 | 6.0 | 7.5 | 4.5 |

***NOTE**: Refer to Table 5 for selection of appropriate option.*

**Use only when frost action is a design consideration.*

$M_r$ = *Resilient Modulus, psi*
$M_r$ = *1500 (CBR)  Note: CBR* $\leq$ *20%*
$M_r$ = *1000 + 555R*

### TABLE 4
### Subgrade Resilient Modulus ($M_r$) as a Function of AASHTO Soil Type

$10^3$ psi=6.94 MPa

| AASHTO Soil Group | Resilient Modulus ($10^3$ psi) Option 1 | Option 2 | Option 3 | Reduced Modulus* ($10^3$ psi) |
|---|---|---|---|---|
| A-1-a | 20.0 | 20.0 | 20.0 | N/A |
| A-1-b | 20.0 | 20.0 | 20.0 | 12.0 |
| A-2-4,A-2-5,A-2-7 | 20.0 | 20.0 | 20.0 | 4.5 |
| A-2-6 | 7.5 | 15.0 | 20.0 | 4.5 |
| A-3 | 15.0 | 20.0 | 20.0 | 9.0 |
| A-4 | 7.5 | 15.0 | 20.0 | 4.5 |
| A-5 | 4.5 | 6.0 | 9.0 | 4.5 |
| A-6 | 4.5 | 10.5 | 20.0 | 4.5 |
| A-7-5 | 4.5 | 6.0 | 7.5 | 4.5 |
| A-7-6 | 7.5 | 15.0 | 20.0 | 4.5 |

**Use only when frost action is a design consideration.*

**Pavement Materials**—The type, strength and thickness of all available paving materials should be established. Crushed aggregate bases, or stabilized bases used in highway construction are generally suitable for interlocking concrete pavement. Most states, provinces and municipalities have material and construction standards for these bases. If none are available, then the standards for aggregate bases found in ASTM D 2940 (6) may be used. Minimum recommended strength requirements for unbound aggregate bases should be CBR = 80% and CBR = 30% for subbases.

For unbound aggregate base material, the Plasticity Index should be no greater than 6, the Liquid Limit limited to 25 and compaction should be at least 98% of AASHTO T-180 density. For unbound granular subbase material, the material should have a Plasticity Index less than 10, a Liquid Limit less than 25 and compaction requirements should be at least 98% of AASHTO T-180 density. In-place density should be checked in the field *as this is critical to the performance of the pavement.* If an asphalt-treated base is used, the material should conform to dense graded, well compacted, asphalt concrete specifications, i.e., Marshall stability of at least 1800 pounds (8000 N). For example, a state Superpave intermediate binder course mix required for interstate or primary roads may be adequate. Cement-treated base material should have a 7-day unconfined compressive strength of at least 650 psi (4.5 MPa).

Recommended minimum base thicknesses are 4 in. (100 mm) for all unbound aggregate layers, 3 in. (75 mm) for asphalt-treated bases, and 4 in. (100 mm) for cement-treated bases. A minimum thickness of aggregate base (CBR=80) should be 4 in. (100 mm) for traffic levels below 500,000 EALs and 6 in. (150

### TABLE 5
### Environment and Drainage Options for Subgrade Characterization

| Quality of Drainage | Percent of Time Pavement is Exposed to Moisture Levels Approaching Saturation <1% | 1 to 5% | 5 to 25% | >25% |
|---|---|---|---|---|
| Excellent | 3 | 3 | 3 | 2 |
| Good | 3 | 3 | 2 | 2 |
| Fair | 3 | 2 | 2 | 1 |
| Poor | 2 | 2 | 1 | 1 |
| Very Poor | 2 | 1 | 1 | 1 |

### TABLE 6
### ASTM C 33
### Gradation for Bedding Sand

| Sieve Size | Percent Passing |
|---|---|
| $^3/_8$ inches (9.5 mm) | 100 |
| No. 4 (4.75 mm) | 95-100 |
| No. 8 (2.36 mm) | 80-100 |
| No. 16 (1.18 mm) | 50-85 |
| No. 30 (0.600 mm) | 25-60 |
| No. 50 (0.300 mm) | 10-30 |
| No. 100 (0.150 mm) | 2-10 |
| No. 200 (0.075 mm) | 0-1 |

mm) for EALs over 500,000.

Bedding sand thickness should be consistent throughout the pavement and not exceed 1.5 in. (40 mm) after compaction. A thicker sand layer will not provide stability. Very thin sand layers (less than 3/4 in. [20 mm] after compaction) may not produce the locking up action obtained by sand migration upward into the joints during the initial compaction in construction. The bedding layer should conform to the gradation in ASTM C 33 (6), as shown in Table 6. Do not use screenings or stone dust. The sand should be as hard as practically available.

Joint sand provides vertical interlock and shear transfer of loads. It can be slightly finer than the bedding sand. Gradation for this material can have a maximum 100% passing the No. 16 sieve (1.18 mm) and no more than 10% passing the No. 200 sieve (0.075 mm). Bedding sand may be used for joint sand. Additional effort in filling the joints during compaction may be required due to its coarser gradation. See ICPI Tech Spec 9, *Guide Specification for the Construction of Interlocking Concrete Pavement* for additional information on gradation of bedding and joint sand.

**TABLE 7**

| ESALs | TI |
|---|---|
| $5x10^4$ | 6 |
| $1x10^5$ | 6.8 |
| $3x10^5$ | 7.2 |
| $5x10^5$ | 8.3 |
| $7x10^5$ | 8.6 |
| $1x10^6$ | 9 |
| $3x10^6$ | 10.3 |
| $5x10^6$ | 10.9 |
| $7x10^6$ | 11.3 |
| $1x10^7$ | 11.8 |
| $2x10^7$ | 12.8 |
| $3x10^7$ | 13.5 |

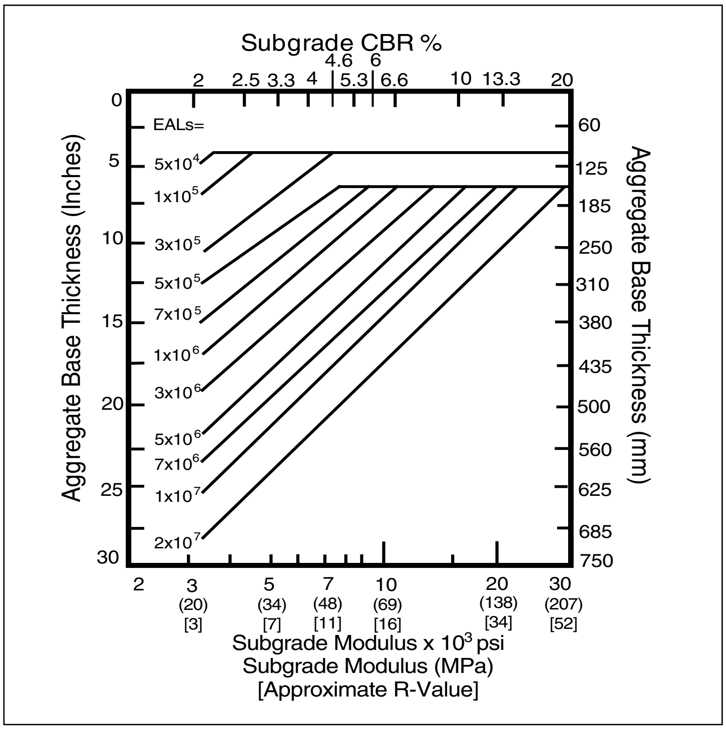


*Figure 5. Thickness design curves–aggregate base*

Concrete pavers should conform to the ASTM C 936 (6) in the U.S. or CSA A231.2 (8) in Canada. A minimum paver thickness of 3.125 inches (80 mm) is recommended for all pavements subject to vehicular traffic, excluding residential driveways. As previously mentioned, the units should be placed in a herringbone pattern. No less than one-third of a cut paver should be exposed to tire traffic.

Research in the United States and overseas has shown that the combined paver and sand layers stiffen as they are exposed to greater numbers of traffic loads. The progressive stiffening that results in "lock up" generally occurs early in the life of the pavement, before 10,000 EALs. Once this number of loads has been applied, $M_r$ = 450,000 psi (3100

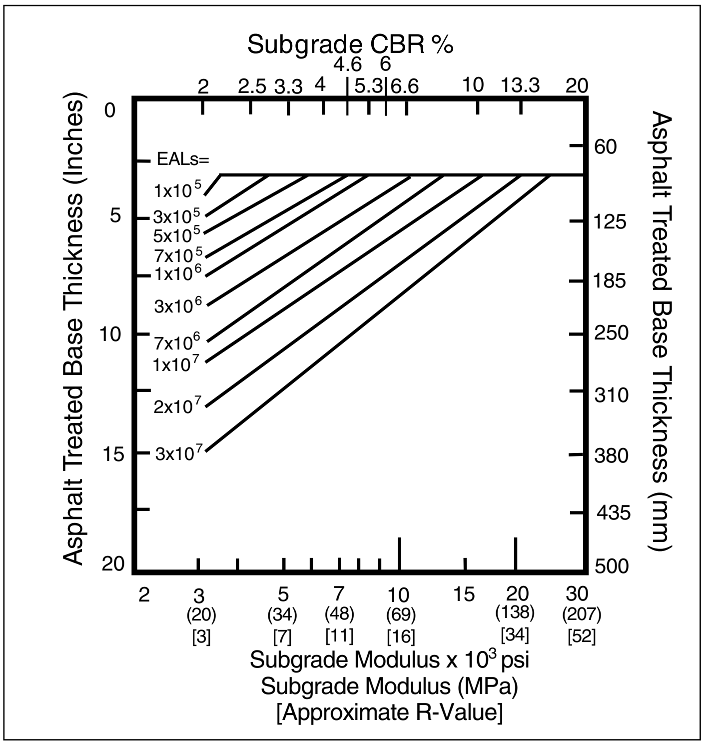


*Figure 6. Thickness design curves–asphalt treated base*

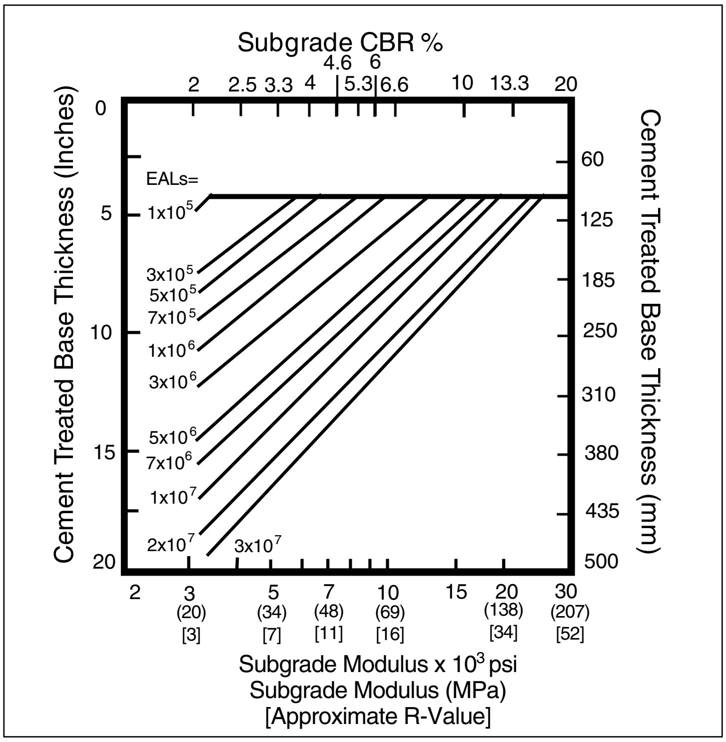


*Figure 7. Thickness design curves–cement treated base*

MPa) for the 3.125 in. (80 mm) thick paver and 1 in. (25 mm) of bedding sand. Pavement stiffening and stabilizing can be accelerated by static proof-rolling with an 8–10 ton (8–10 T) rubber tired roller.

The above modulus is similar to that of an equivalent thickness of asphalt. The 3.125 in. (80 mm) thick pavers and 1 in. (25 mm) thick bedding sand have an AASHTO layer coefficient at least equal to the same thickness of asphalt, i.e., 0.44 per inch (25 mm). Unlike asphalt, the modulus of concrete pavers will not substantially decrease as temperature increases nor will they become brittle in cold climates. They can withstand loads without distress and deterioration in temperature extremes.

## Structural Design Curves

Figures 5, 6, and 7 are the base thickness design curves for unbound aggregate, asphalt-treated and cement-treated materials. The thicknesses on the charts are a function of the subgrade strength ($M_r$, R-value or CBR) and design traffic repetitions (EALs). Use the following steps to determine a pavement thickness:

1. Compute design EALs or convert computed TIs to EALs. Use known traffic values or use the recommended default values given in Table 2. EALs are typically estimated over a 20-year life. Annual growth of EALs over the life of the pavement should be considered.

2. Characterize subgrade strength from laboratory test data. If there is no laboratory or field test data, use Tables 3 and 4 to estimate $M_r$, CBR or R-value.

3. Determine the required base thickness. Use $M_r$, R-value or CBR for subgrade strength and design EALs or TIs listed in Table 7 input into Figures 5, 6 or 7, depending on the base material required. A portion or all of the estimated base thickness *exceeding* the minimum thickness requirements can be substituted by a lower quality, unbound aggregate subbase layer. This is accomplished through the use of layer equivalency values: 1 in. (25 mm) of aggregate base is equivalent to 1.75 in. (45 mm) of unbound aggregate subbase material; 1 in. (25 mm) of asphalt-treated base is equivalent to 3.4 in. (85 mm) of unbound aggregate subbase material; and 1 in. (25 mm) of cement-treated base is equivalent to 2.5 in. (65 mm) of unbound aggregate subbase.

## Example

**Design Data**—A two-lane urban, residential street is to be designed using concrete pavers. Laboratory tests on the subgrade soil indicate that the pavement is to be constructed on a sandy silt; i.e., ML soil type according to the USCS classification system. No field CBR or resilient modulus data are available. From available climatic data and subgrade soil type, it is anticipated that the pavement will be exposed to moisture levels approaching saturation more than 25% of the time. Drainage quality will be fair and frost is a design consideration. Detailed EAL traffic data are not available.

Using the above information, designs are to be developed for the following base and subbase paving materials: unbound aggregate base, asphalt-treated base, and unbound aggregate subbase. All designs are to include a base layer but not necessarily the aggregate subbase layer.

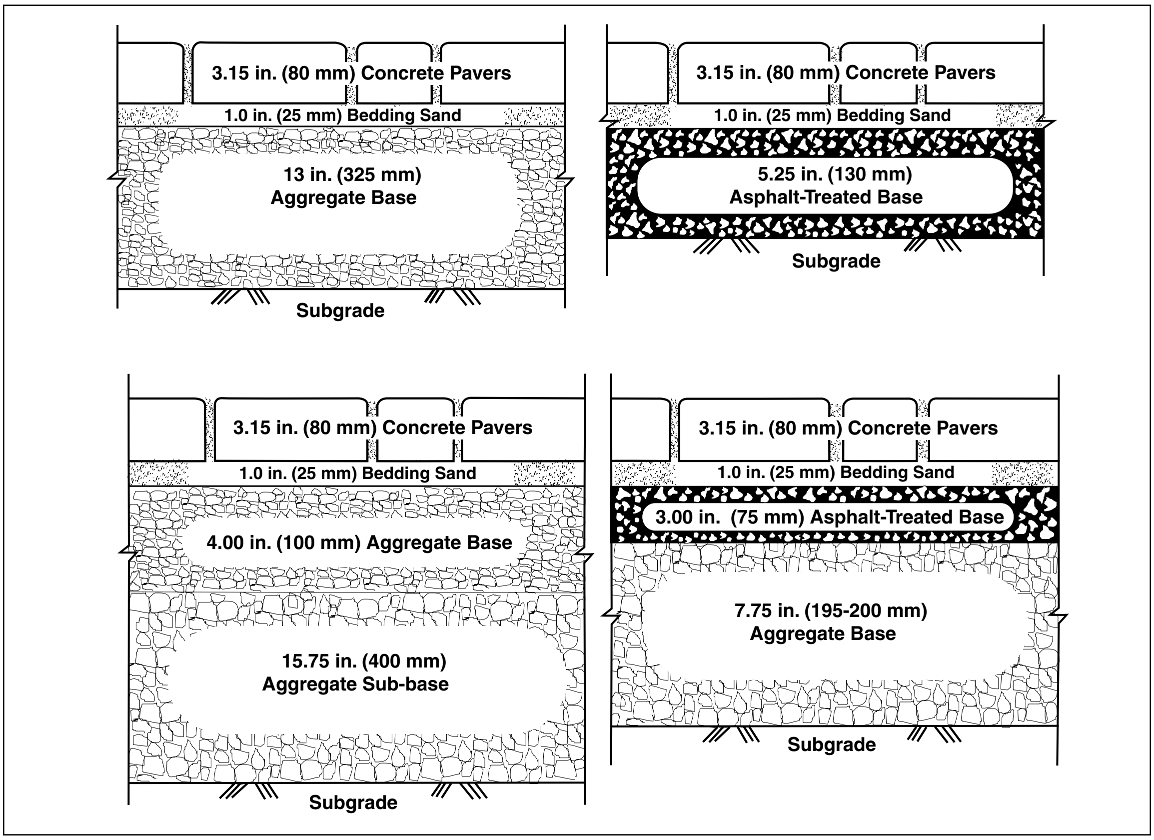


*Figure 8. Alternative cross section solutions for the design example*

## Solution and Results

**1. Estimate design EAL repetitions.** Since detailed traffic information was not available, the value recommended in Table 2 was used: 840,000 design EALs or TI = 8.8.

**2. Characterize subgrade soil strength.** Since only its USCS soil classification is known, Table 3 was used to establish the design strength value. For a USCS ML soil and the given moisture and drainage conditions, the estimated subgrade modulus value is $M_r$ = 7,500 psi (52 MPa), CBR = 5% or R-value = 23. Since frost action is a consideration, the reduced design strength value is $M_r$ = 4,500 psi (31 MPa), CBR = 3% or R-value = 8.

**3. Determine base thickness requirements.** Input of the design traffic (840,000 EALs) and subgrade strength ($M_r$ = 4,500 psi [31 MPa]) values into Figures 5 and 6 yields base thickness requirements of 13 in. (330 mm) for unbound aggregate, or 5.25 in. (133 mm) for an asphalt treated base.

These values can be used to develop subbase thicknesses. Since all designs must include a base layer, only that thickness exceeding the minimum allowable value, 4 in. (100 mm) for aggregate bases and 3 in. (75 mm) for asphalt-treated bases, was converted into subbase quality material. With the aggregate base option, 9 in. (230 mm) or 13 - 4 in. of material can be converted into aggregate subbase quality material, resulting in 15.75 inches (400 mm) or 9 x 1.75 inches. Likewise, for the asphalt-treated base option, 2.25 in. (57 mm) or 5.25 - 3.0 in. of material can be converted into aggregate subbase quality material, resulting in 7.75 in. (197 mm) or 2.25 x 3.40 in.

The final cross section design alternatives are shown in Figure 8 with 3.15 in. (80 mm) thick concrete pavers and a 1.0 in. (25 mm) thick bedding sand layer over several bases. These are a sample of the possible material type and thickness combinations which satisfy the design requirements. Cost analyses of these and other pavement cross section alternatives should be conducted in order to select the optimal design.

## Computerized Solutions

Interlocking concrete pavement can be designed with ICPI Lockpave software, a computer program for calculating pavement base thicknesses for parking lot, street, industrial, and port applications. User designated inputs include traffic loads, soils, drainage, environmental conditions and a variety of ways for characterizing the strength of pavement materials. Parking lot and street pavement thickness can be calculated using the 1993 AASHTO pavement design procedure (an empirical design method) or a mechanistic, layered elastic analysis that computes projected stresses and strains in the pavement structure modified by empirical factors. The AASHTO 2002 *Guide for Design of Pavement Structures* includes procedures for mechanistic analysis of pavement layers.

Outputs include pavement thickness using different combinations of unstabilized and stabilized bases/subbases. Base thicknesses can be calculated for new construction and for rehabilitated asphalt streets using an overlay of concrete pavers. After a pavement structure has been designed, the user can project life-cycle costs by defining initial and lifetime (maintenance and rehabilitation) cost estimates. Design options with initial and maintenance costs plus discount rates can be examined for selection of an optimal design from a budget standpoint. Sensitivity analysis can be conducted on key cost variables on various base designs. For further information on ICPI Lockpave, contact ICPI members, ICPI offices, or visit the web site http://www.icpi.org.

## References

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