All too often, pavement design tends to focus only on thickness design. However, there are numerous other aspects of a CRCP that affect its behavior and performance including the reinforcement, thickness, shoulders, support, and concrete-making materials. The design of the CRCP should therefore consider each of these features, and ideally arrive at an optimum design through an iterative process.

To complete the design, a life-cycle cost analysis is sometimes performed. This allows the designer to take into account the costs associated with various pavement design alternatives along with the benefits in terms of increased pavement performance.

CRCP Design Methods

Various design methods for determination of slab thickness and the amount of reinforcement required in CRCP have been developed in the past. The two most relevant due to their common use and/or level of validation include:

1. AASHTO-86/93 Guide for Design of Pavement Structures
2. AASHTO Interim MEPDG

AASHTO-86/93 Design Procedure

Although the AASHTO-86/93 Guide procedure does not directly consider one of the primary failure mechanisms in CRCP (punchout development), this procedure has been used for design of CRCP by making similar considerations to those for the design of jointed concrete pavements. In addition, because reinforcement keeps cracks tight in CRCP, a slightly improved load transfer coefficient is typically used, which results in a moderate reduction in thickness for this pavement type under similar traffic and environmental conditions.

The AASHTO-86/93 method also includes design procedures for the selection of reinforcement. These procedures are based on a desired range of crack spacing, maximum crack width, and maximum steel stress. It should be noted that it has been reported that this design procedure tends to underestimate the required steel. A summary of the AASHTO-86/93 design procedure is presented in Appendix B since it is currently the most widely used design procedure for design of CRCP(4).

AASHTO Interim MEPDG

Over the last decade, the National Cooperative Highway Research Program (NCHRP) has undertaken a major effort to develop the next generation of pavement design procedure based on mechanistic-empirical methods. This has been conducted under research project 1-37A, and has resulted in the current AASHTO Interim MEPDG. In this design procedure, specific mechanistic-empirical models for prediction of CRCP performance have been developed.(29,53)

A flow diagram of the AASHTO Interim MEPDG CRCP design process is given in Figure 10. The process begins with the selection of a trial design including layer thicknesses, materials, reinforcement, shoulder characteristics, and construction information. Site-specific conditions including environment, foundation, and traffic are also considered. Performance criteria in terms of punchouts and IRI are then specified, along with the reliability level for each criterion. The MEPDG also has limiting design criteria on crack width (over design period), crack spacing, and crack load transfer efficiency (over design period as well).

The procedure explicitly predicts punchout development as a function of the crack width and load transfer efficiency due to aggregate interlock at transverse cracks. Stresses due to loading are predicted as a function of load transfer efficiency, and continuously evaluated and modified throughout the design period. Fatigue damage as a function of the stress level and strength is evaluated and accumulated, and punchout development is subsequently predicted.

IRI is also predicted throughout the design period as a function of the initial smoothness conditions, punchout development, and site-specific conditions. Once the trial design is evaluated, its predicted performance is checked against design criteria at the specified reliability level. If the design requirements are satisfied, the trial design is considered as a viable alternative that can later be evaluated in terms of life-cycle costing. Otherwise, a new trial design is evaluated.
Figure 10. Framework of CRCP design procedure in the AASHTO Interim MEPDG.\(^{(29)}\)
Early-Age Behavior Analysis Tools

In addition to the above design procedures, software tools such as FHWA HIPERPAV (High PERformance concrete PAVing) II and CRSI PowerPave are currently available to engineers for the analysis of CRCP early-age behavior. These tools can be used as a way to fine-tune designs under given site-specific construction scenarios. These tools may be also helpful in developing specifications based on locally available materials and construction procedures.

HIPERPAV II

Developed under sponsorship of the FHWA, this software represents computerized design and construction guidelines for optimization the early-age behavior of CRCP. In HIPERPAV II, the crack spacing, crack width, steel stress, and bond development length are predicted as a function of design, materials, environment, and construction factors, and then evaluated in terms of predefined design thresholds. A sophisticated model for the prediction of concrete pavement temperature is used at the core of the software. The predicted pavement temperature is used for both the prediction of strength and stress development in the concrete. The cracking prediction is based on the CRCP-8 model. Originating from findings under NCHRP research project 1-15, the CRCP-8 model has since been extensively validated through laboratory and field testing.

PowerPave

Developed by CRSI, PowerPave is similar to HIPERPAV II, and is also based on the CRCP-8 model. However, pavement temperature is not predicted but it is rather an input to the software.

Evaluation of Critical Stresses

It is sometimes worth checking the level of stresses of a CRCP in order to evaluate the use of alternate base types. A program based on elastic layered theory (ELSYM5 or BISAR) can be used for this purpose. Alternatively, discrete element (SLAB49) or finite element (ILLISLAB, ISLAB2000, ABAQUS, EverFE2.2) methods can be used. These programs can also be used to model Westergaard’s plate theory and develop composite k-values. Additional discussion of stress evaluation can be found in the AASHTO Interim MEPDG documentation.

Concrete Thickness

Thickness design involves the determination of the minimum required CRCP thickness that will produce an acceptable level of stress in the pavement under traffic and environmental loadings. The assumption being that the targeted stress will reduce the potential for punchouts and other structural distresses, while at the same time maintaining an acceptable level of function (e.g., smoothness).

Reduction of stresses in the pavement slab is not only achieved by increasing thickness but by consideration of numerous other factors including:

- **High load transfer efficiency** – This can be accomplished by keeping transverse cracks tight with the use of an adequate longitudinal steel content to achieve good aggregate interlock. Selecting large size aggregates that are resistant to abrasion will also improve load transfer at the cracks.
- **Sufficient lateral support** – Tied concrete shoulders or widened lanes that extend beyond the wheelpath into the shoulder at least one foot provide improved lateral support over asphalt shoulders, as well as aid in mitigating punchouts.
- **Uniform and stable structural support under the slab** – This may be achieved by stabilizing subgrade if swelling is expected and/or by selecting erosion-resistant bases that minimize erosion and pumping of subgrade materials and through accelerated freeze-thaw and wet-dry testing with strength assessment can be demonstrated to have long-term durability.
- **Prevention of subgrade or base saturation** – This can be achieved by improving drainage features such as selecting non-erodible or permeable moisture insensitive bases.
- **Improved concrete structural properties** – Although excessively high concrete strengths are not desirable, producing concrete with sufficient strength and a low modulus of elasticity will help in reducing stresses due to traffic loading.

Taking the above measures will minimize potential for punchout development at a minimum required thickness, thus resulting in a more cost-effective design.

In the past, it was common practice by some States to design CRCP thickness based on jointed concrete pavement methodology, and then reduce the thickness by as much as 20 percent to account for the effect of increased load transfer efficiency at the cracks.
In some cases, this resulted in an under-design, which in turn required expensive maintenance and rehabilitation. As a result, this practice is no longer recommended. Today, typical CRCP thicknesses vary from 7 to 15 in (178 to 381 mm) depending on the level of traffic and environmental conditions, although most common practice is between 10 and 12 inches (254 to 305 mm).

**Longitudinal Reinforcement**

Reinforcement design involves selecting the proper percentage, bar size, and bar configuration for optimum CRCP performance. Reinforcement design is focused on providing the minimum reinforcement necessary to develop the desired crack spacings and widths, while at the same time keeping the steel at an acceptable level of stress. States that have been designing CRCP have established standard details for longitudinal layout and bar size.

**Reinforcement Content**

Longitudinal steel reinforcement content is defined as the ratio of the area of longitudinal steel to the area of concrete ($A_s/A_c$) across a transverse section, often expressed as a percentage. As illustrated in Figure 11, higher amounts of steel reinforcement will result in shorter crack spacings, smaller crack widths, and lower steel stresses. An increase in the percent of longitudinal reinforcement will result in an increase in restraint.

As the level of restraint increases, so does the number of cracks that develop, resulting in shorter crack spacings. In addition, as the amount of reinforcement increases, the average steel stresses are reduced, producing less reinforcement elongation.

As previously mentioned in Section 2.1.1, crack spacings between 3.5 to 8 ft (1.1 and 2.4 m) minimize the potential for development of punchouts and spalling. However, it has been observed that crack spacings as short as 2 ft (0.6 m) have shown good performance as long as good support underneath is provided. Crack widths under 0.024 in (0.6 mm) prevent infiltration of water and incompressibles, and ensure adequate load transfer efficiency between the cracks thus reducing load induced stresses. In addition, keeping the steel working at an acceptable stress level minimizes fracture of the steel or excessive yield that may lead to wide cracks with poor load transfer efficiency.

Longitudinal reinforcement should be designed to meet the following three criteria:

1. Produce a desirable crack pattern (spacing),
2. Keep transverse cracks tightly closed, and
3. Keep reinforcement stresses within allowable levels.

---

*Figure 11. Conceptual representation of steel design for CRCP.*

---

**Steel Stress x 10^3 psi**

**Crack Width x 10^{-2} (in.)**

**Crack Spacing (ft)**

- Maximum Crack Spacing
- Allowable Steel Stress
- Allowable Crack Width
- Minimum Crack Spacing
- Acceptable Design

---

**% Steel Reinforcement**
Although cracking characteristics in CRCP largely depend on the amount of reinforcement, they are also a function of the climatic conditions during placement, materials properties, and construction factors as discussed in Section 3. When designing for longitudinal reinforcement, all these factors need to be taken into consideration.

Specifications for maximum concrete temperatures, low CTE aggregates, and proper curing procedures can help ensure that the intended performance from the reinforcement design will be achieved.

It is also important to consider the effect that excess thickness can have on CRCP performance. Concrete pavement specifications commonly allow for a pay incentive (bonus) for additional pavement thickness due to the resulting increase in structural capacity. However, for CRCP, increasing the thickness (while maintaining the same amount of reinforcement) results in a reduction of the reinforcement percentage. This, in turn, can result in larger crack spacings, wider cracks, and an increase in reinforcement stress. This effect should be considered when specifying an upper limit for thickness pay incentives. For this reason, CRCP should also not be used as a leveling layer.

As a general guideline for conventional steel deformed rebar, reinforcement percentages between 0.60% and 0.80% have been shown to provide acceptable cracking patterns. A minimum of 0.60% is sometimes recommended since lower levels of steel reinforcement may result in wide transverse cracks, large crack spacings, and high tensile stresses in the steel.

On the other hand, steel reinforcement above 0.80% may result in very short crack spacings that later progress into punchout development, particularly under poor support conditions. These recommended limits for steel percentage are based on typical materials properties, and environmental conditions found in the US – particularly the northern states that are subjected to larger temperature extremes.

It should be noted that there exists an optimum steel percentage for any specific project. It should be based on the specified materials and environmental conditions to which the pavement will be subjected. While lower percentages of reinforcement may work in milder climates with ideal materials, it should again be emphasized that lower percentages of reinforcement can still lead to longer crack spacings adjacent to short crack spacings, with the latter ones increasing the probability of punchout development.\(^{(8)}\)

### Bar Size and Spacing

Longitudinal steel is typically designed to meet a minimum spacing in order to achieve good consolidation of concrete during placement. A maximum spacing is also considered to exist in order to ensure adequate concrete bond strength and thus tight crack widths. FHWA Technical Advisory T 5080.14 provides guidelines for minimum and maximum spacing of longitudinal steel as follows:\(^{(56)}\)

- The minimum spacing of longitudinal steel should be the greater of 4 in (100 mm) or 2½ times the maximum aggregate size.
- The spacing of longitudinal steel should be not greater than 9 in (230 mm).

Typical bar sizes used in CRCP range from #4 (0.5 in) to #7 (0.875 in) [#13M (12.7 mm) to #22M (22.2 mm)].

Selection of the steel bar size (diameter) is governed by steel percentage and minimum and maximum spacing permitted.

With the required amount of reinforcement and bar size selected, the reinforcement spacing, \(S\), may be computed as follows:

\[
S = \frac{\phi^2 \cdot \pi}{4 \cdot D \cdot p_s} \cdot 100
\]

where,
- \(S\) = Reinforcement spacing (in or mm),
- \(\phi\) = bar diameter (in or mm),
- \(D\) = Slab thickness (in or mm),
- \(P_s\) = Longitudinal reinforcement percentage (fraction), and
- \(\pi\) = \(\pi\) (3.141593).

It is recommended that the reinforcement spacing determined with the above equation be considered as the maximum to maintain the required longitudinal reinforcement percentage. If this spacing needs to be adjusted, it should be done so by rounding down to a practical spacing according to pavement geometry.

Another option commonly exercised is to space the bars near the slab edge closer, as indicated in TxDOT standard “CRCP (1) 03” (shown in Appendix A).

Figure 12 provides recommended bar spacing for various slab thicknesses and bar sizes as a function of reinforcement percentage.
Other considerations should be made when selecting the bar size including evaluation of the reinforcement surface (bond) area. It has been observed that the average crack spacing decreases with an increase in ratio of reinforcement surface area to concrete volume. A possible explanation for this is that the high tensile stresses in the steel at crack locations are transferred to the concrete as a function of the reinforcement surface area and deformation characteristics of the longitudinal reinforcement.\(^{(55)}\) On the other hand, the greater the bond area, the more restraint to movement of the concrete is imposed by the steel, and therefore, tighter cracks are expected to result.\(^{(57)}\)

For a given reinforcement percentage, higher surface area is achieved using smaller bar sizes. Therefore, reinforcement design should also consider this. For this reason, the ratio of reinforcement surface area to concrete volume, \(R_b\), is typically controlled to take into account the bar size effect. This ratio can be determined with the following relationship:

\[
R_b = \frac{\phi \cdot \pi}{S \cdot D} \quad \text{(sq.in/cu.in or m}\(^2\)/m}\(^3\))
\]

where, \(R_b\) = Ratio of reinforcement surface area to concrete volume,
\(\phi\) = bar diameter (in or mm),
\(\pi\) = pi (3.141593),
\(S\) = Reinforcement spacing (in or mm), and
\(D\) = Slab thickness (in or mm).

A minimum ratio of steel surface area to concrete volume of 0.03 sq.in/cu.in (1.2 m\(^2\)/m\(^3\)) is typically recommended for summer construction and of 0.04 sq.in/cu.in (1.6 m\(^2\)/m\(^3\)) for fall or winter construction.\(^{(9)}\)

---

### Reinforcement Spacing (for one layer of bars)

<table>
<thead>
<tr>
<th>Bar Size, mm (bar size)</th>
<th>15.9 (0.65)</th>
<th>18.1 (0.71)</th>
<th>22.2 (0.87)</th>
<th>25.4 (1.00)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing, mm (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>203 (8)</td>
<td>0.77%</td>
<td>0.92%</td>
<td>0.98%</td>
<td>1.02%</td>
</tr>
<tr>
<td>220 (9)</td>
<td>0.88%</td>
<td>0.92%</td>
<td>0.98%</td>
<td>1.03%</td>
</tr>
<tr>
<td>254 (10)</td>
<td>0.81%</td>
<td>0.91%</td>
<td>0.97%</td>
<td>1.02%</td>
</tr>
<tr>
<td>279 (11)</td>
<td>0.90%</td>
<td>0.90%</td>
<td>0.93%</td>
<td>1.00%</td>
</tr>
<tr>
<td>306 (12)</td>
<td>1.15%</td>
<td>0.87%</td>
<td>0.96%</td>
<td>1.06%</td>
</tr>
<tr>
<td>330 (13)</td>
<td>0.80%</td>
<td>0.77%</td>
<td>0.95%</td>
<td>1.01%</td>
</tr>
</tbody>
</table>

### Pavilion Slab Thickness, mm (in)

<table>
<thead>
<tr>
<th>Pavement Slab Thickness, mm (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.9 (0.65)</td>
</tr>
<tr>
<td>Spacing, mm (in)</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>15</td>
</tr>
</tbody>
</table>

### Reinforcement Spacing (for two layers of bars)

<table>
<thead>
<tr>
<th>Bar Size, mm (bar size)</th>
<th>15.9 (0.65)</th>
<th>18.1 (0.71)</th>
<th>22.2 (0.87)</th>
<th>25.4 (1.00)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing, mm (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>15</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
</tbody>
</table>
Vertical Position of Reinforcement

There are two primary considerations that should be made when selecting the vertical position of the longitudinal reinforcing steel.

On one hand, drying shrinkage and temperature fluctuations are typically more pronounced at the pavement surface, and can result in wider cracks at this location. It is believed that by positioning the reinforcement closer to the surface, narrower crack widths and higher load transfer efficiency can be achieved.

On the other hand, keeping the reinforcement closer to the surface increases the probability of exposure to chlorides from deicing salts, which may lead to corrosion. Future diamond grinding of the pavement surface would further reduce the distance of the reinforcement from the surface. Given these two considerations, it is common to position the reinforcement between one-third and one-half the slab thickness measured from the pavement surface.

To provide sufficient concrete cover, it is typically recommended to specify a reinforcement depth of at least twice the maximum aggregate size. Illinois DOT requires a minimum reinforcement depth of 3.5 inches (8.9 centimeters) from the pavement surface to the top of the longitudinal reinforcement to minimize the possibility of corrosion and to accommodate variations in construction procedures.

It is also recommended that the maximum reinforcement depth be no more than half the slab thickness measured from the surface. As illustrated in Figure 13, placement of reinforcement in two layers has also been used. This is implemented in the TxDOT specifications for pavements thicker than 13 in (330 mm). The TxDOT standard CRCP(2)-03 in Appendix A shows a detail for placement of reinforcement in two layers.

It is believed that placement of the required reinforcement in this way not only helps to maintain the optimum reinforcement bond area to concrete volume ratio, but also ensures proper spacing while at the same time allowing reinforcement to be positioned closer to the CRCP surface where shrinkage strains tend to produce larger crack widths.

Based on long-term field testing in Illinois and on other projects in Belgium and elsewhere, the depth of reinforcement was a major effect on crack width. The higher the position of the reinforcement, the tighter the transverse cracks. Illinois sections with mid-depth steel had much more full depth repair than those with reinforcement above mid-depth over a 20 year period. Illinois recommends a 3.5 in (89 mm) covering over the reinforcement. The highly successful CRCP pavements in Belgium also place reinforcement above mid-depth.

Lap Splices

Adequate design of the reinforcement lap splices is necessary to maintain reinforcement continuity. Forces induced in the reinforcement by thermal and shrinkage movements are transferred through the lapped splice from one bar to the other by the surrounding concrete bonded to both bars. A minimum lap length of the splices is therefore necessary to ensure sufficient load transfer. Inadequate design and/or construction of the lap splices can result in failure of the reinforcement and poor CRCP performance, ultimately requiring expensive repairs. Typical lap patterns are shown in Figure 14 and also in Figure 68 of Appendix A.

The effectiveness of the splice relies on achieving sufficient bond development length between the concrete and the reinforcement. Special consideration should be given to ensure that the concrete achieves adequate bond strength during the critical early-age period. This is particularly important during cold weather construction when the concrete gains strength at a lower rate.

Guidelines on splicing length among the different States vary from 25 to 33 bar diameters. Some State specifications require the use of a fixed length of lap splicing varying between 16 and 20 in (406 and 508 mm). An experimental study looking at the bond development length for CRCP reported that lap splices of 33 bar diameters provide good performance, and may be the basis for the larger splice length specified.

FHWA Technical Advisory T 5080.14 recommends a minimum splice length of 25 bar diameters if the splicing is performed in a staggered or skewed pattern. For a staggered
splice pattern, no more than one third of the bars should terminate in the same transverse plane. In addition, the minimum distance between staggers should be 4 ft (1.2 m).

For the skewed splice pattern, the skew angle should be at least 30 degrees from perpendicular to centerline. In practice, an approximate skew configuration may be achieved by skewing the reinforcement by half the pavement width (Figure 14). In any case, it is recommended that a minimum lap splicing of no less than 16 in (406 mm) be provided.

**Corrosion Protection**

Some states require epoxy-coated rebar in CRCP to prevent corrosion of the reinforcement, especially in urban areas where maintenance and rehabilitation activities are strongly discouraged. This step may also be justified in environments with high exposure to chlorides from deicing salts, especially where corrosion has been previously identified as a problem.

The use of solid stainless steel, stainless steel clad, and other proprietary reinforcement non corrosive materials, such as Glass Fiber Reinforced Polymer (GFRP), should be considered in areas with high chloride, with heavy deicer applications, or where long life (50 years or greater) is desired. Corrosion of reinforcing steel in CRCP has been rarely reported though, and is usually attributed to inadequate reinforcement design resulting in wide transverse cracks.

The designer should strive to provide sufficient reinforcement to maintain narrow crack widths and sufficient reinforcement depth. These measures will help to minimize the probability of reinforcement exposure to chlorides from deicing salts. Additionally, increased steel percentages to account for potential corrosion may be also considered during design.

In the case where epoxy-coated bars are employed, the effect of the epoxy coating on the reinforcing steel bond development length should be accounted for. The FHWA Technical Advisory TA 5080.14 recommends an increase of 15 percent in the bond area when epoxy coated rebar is used. Although, some studies have found no significant difference in cracking patterns between the use of uncoated and epoxy-coated reinforcement. It is believed that additional research is needed in order to better understand the effect of epoxy coatings on CRCP behavior and performance.

![Figure 14. Typical layout pattern with laps skewed across pavement.](image)
Transverse Reinforcement

Transverse reinforcement in CRCP serves several purposes, including:

1. To function as tiebars across longitudinal joints (if continuous).

2. To keep uncontrolled longitudinal cracks that may form held tight (which may occur due to shallow saw cuts, late sawing, differential settlement, or heave).

3. To support longitudinal steel in place (ensuring proper spacing and depth according to specifications if mechanical placement of the longitudinal steel is not used).

Illinois and Texas, which both predominantly use CRCP, both use transverse reinforcement – Illinois in urban areas and Texas in all areas. For pavements wider than 7m (24 ft), it is recommended to use continuous transverse reinforcement with an expansion joint adjacent to concrete traffic barriers.

Size and Spacing

As with longitudinal reinforcement, the design of transverse reinforcement consists of determining the required amount of reinforcement per cross sectional area of concrete, and then selecting a corresponding bar size and spacing configuration. The reinforcement design is based on equilibrium of base layer restraint and concrete contraction forces. The required percentage of transverse reinforcement can be obtained with the following relationship:

\[
p_t = \frac{\gamma_c \cdot W_s \cdot F}{2 \cdot f_s} \cdot 100
\]

where, \( p_t \) = Percentage of transverse reinforcement (%), \( \gamma_c \) = Unit weight of concrete (lb/cu.in or kN/m\(^3\)), \( W_s \) = Total pavement width (in or m), \( F \) = Coefficient of friction (unitless), and \( f_s \) = Working stress of steel (75% of yield strength) (psi or kPa).

Friction coefficients for different base materials are provided in Table 4.

Once the required percentage of transverse reinforcement is determined, a bar size is selected and the transverse steel spacing is obtained as follows:

\[
Y = \frac{\phi^2 \cdot \pi}{4 \cdot p_t \cdot D} \cdot 100
\]

where, \( Y \) = Transverse steel spacing (in or mm), \( \phi \) = Bar diameter (in or mm), and \( D \) = Slab thickness (in or mm).

Typical reinforcing recommended in the transverse direction includes # 4 to # 6 (12.7 to 19.1 mm) Grade 60 (Grade 420) deformed bars. It is common for transverse reinforcement to be spaced from 12 in (0.3 m) to 36 in (0.9 m) apart. It should be noted that following this design procedure is very conservative, as the steel stresses are greatest at the tied joints and in the center of the pavement and decrease to zero at the free edges.

Transverse reinforcement may serve a second purpose as tiebars if continuous across the joint. In this configuration, the transverse bars are typically extended half the required tiebar length across the longitudinal joint. For example, for a 24-ft (7.3-m) wide pavement, transverse reinforcement is designed for 12-ft (3.65-m) pavement widths and this reinforcement is extended across the middle longitudinal joint to function as a tiebar.

Since transverse reinforcing bars are typically used to keep longitudinal reinforcement bars in place, they are commonly specified to be positioned below the longitudinal reinforcement.

Tiebars

Tiebars are used along lane-to-lane or lane-to-shoulder longitudinal joints, when placed in separate passes, to keep these joints tight, and to ensure adequate load transfer. The required amount of tiebar reinforcement along longitudinal

<table>
<thead>
<tr>
<th>Type of Material Beneath Slab</th>
<th>Friction Factor (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface treatment</td>
<td>2.2</td>
</tr>
<tr>
<td>Lime stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Asphalt stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Cement stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>River gravel</td>
<td>1.5</td>
</tr>
<tr>
<td>Crushed stone</td>
<td>1.5</td>
</tr>
<tr>
<td>Sandstone</td>
<td>1.2</td>
</tr>
<tr>
<td>Natural subgrade</td>
<td>0.9</td>
</tr>
</tbody>
</table>
joints is determined in a similar way as for transverse reinforcement. However, in this case, the length of pavement for analysis corresponds to the distance from the tied joint to the closest free edge.

A shorter distance to the free edge will result in a lesser amount of reinforcement required to hold the longitudinal joint together. The following equations are used to determine the percentage of tiebar reinforcement and tiebar length required:

\[
Pt = \frac{\gamma_c \cdot W' \cdot F}{f_s} \cdot 100
\]

where, \( P_t \) = Percentage of tiebar reinforcement (%), 
\( \gamma_c \) = Unit weight of concrete (lb/in\(^3\) or kN/m\(^3\)), 
\( W' \) = Distance from tied joint to closest free edge (in or m), 
\( F \) = Coefficient of friction, and 
\( f_s \) = Working stress of steel (75% of yield strength) (psi or kPa).

\[
t = \frac{1}{2} \left( \frac{f_s \cdot \phi}{f_b} \right) + l_a
\]

where, \( t \) = Tiebar length (in or mm), 
\( \phi \) = Bar diameter (in or mm), 
\( f_s \) = Working stress of steel (75% of yield strength) (psi or MPa), and 
\( f_b \) = Allowable bond stress (typically assumed to be 350 psi (2.44 MPa)), 
\( l_a \) = Additional length for misalignment (3 in or 75 mm).

For economy and simplicity, tiebar length is often selected based on available standard manufactured lengths. Typical tiebars consist of Grade 40 or 60 (Grade 300 or 420) steel. Common standard manufactured tiebar lengths include 24, 30, 36, 42, and 48 in (0.61, 0.76, 0.91, 1.07, and 1.22 m). A maximum tiebar spacing of 48 inches (1.22 m) is recommended.

For wide pavements, it is generally more economical to provide an untied longitudinal joint rather than extending transverse bars along the total pavement width. Caltrans, for example, requires at least two lanes but no more than 50 ft (15.2 m) between untied joints. An untied joint may alleviate excessive transverse concrete stresses that could lead to potential longitudinal random cracking. It is recommended that untied joints be located far from the pavement edge to avoid lane separation. Expansion joints adjacent to concrete traffic barriers without providing load transfer are commonly used to prevent the formation of uncontrolled longitudinal cracks in these situations.

For longitudinal construction joints, tiebars are often inserted along the edge while concrete is fresh. It should be emphasized though that bending of the tiebar should be discouraged since it may result in cracking of the steel, and based on experience, there have been several instances where the tiebars have not been unbent prior to placing the adjacent concrete. In addition, when tiebars are epoxy coated, bending may result in damage of the epoxy, giving place to corrosion.

Some State DOTs require the use of multiple-piece tiebars to prevent problems associated with bending bars. This type of tiebar comes in two pieces that are threaded together. Female couplers are inserted along the longitudinal joint in the fresh concrete before paving the adjacent section; the threaded piece is later screwed in place to form a complete tiebar. With the use of these mechanisms, bending of the tiebar is avoided. Multiple-piece tiebars should conform to ASTM A 615 specifications, and the coupler should be required to develop a failure force of 1.25 to 1.5 times the yield strength of the steel.\(^{12}\) Another option is to drill and epoxy tiebars and require a pull-out test as per ASTM E 488. Regardless of the tiebar type and method of placement, it is important to ensure that tiebars are securely anchored so that they provide the pull-out resistance required by design.

**Joints**

CRCP does not require transverse joints in the same way that jointed concrete pavements do. However, joints in CRCP are still necessary. Joints in the longitudinal direction are used between traffic lanes, and between traffic lanes and tied concrete shoulders. Transverse joints are also necessary for construction purposes at the start and end of daily operations. Transition or terminal joints are also necessary for approaches to some types of structures and the transition to other pavement types. This section provides design details that should be considered when designing CRCP joints.
**Longitudinal Joints**

The use of longitudinal joints is recommended for pavements wider than 15 ft (4.6 m). Joints are typically located in between lanes and between a lane and a concrete tied shoulder. Tiebars or transverse reinforcement should be provided along longitudinal joints to prevent separation, and to maintain adequate joint transfer efficiency. Two types of longitudinal joints are commonly used in CRCP: construction and contraction joints.

Better practices for concrete pavement joints can be found in the IMCP manual and elsewhere. However, some typical sections of joints that have been used on CRCP pavements are given herein.

A longitudinal construction joint is illustrated in Figure 15, and is specified when more than one pass is necessary to pave the total pavement width. In the past, it was recommended that trapezoidal or half-round keyways were formed along the longitudinal construction joints to increase its load transfer efficiency. However, some key joints have failed in shear resulting in spalling along the joint. In addition, the area along the keyway may be more susceptible to concrete consolidation problems. It is therefore recommended that tied butt joints be used instead.

Longitudinal contraction (hinged) joints, as illustrated in Figure 16, ensure that longitudinal cracking on wide pavements occur along a fixed (weakened) plane. Longitudinal joints are created by performing a sawcut at the specified joint location to relieve the stresses generated in the transverse direction of the concrete due to volumetric changes. Along these longitudinal joints, either transverse reinforcement or tiebars should be used to tie the adjacent slabs together and to maintain adequate load transfer efficiency. Reinforcement design should follow recommendations provided in Section 4.5.
Transverse Construction Joints

Transverse construction joints are formed at the start and end of paving operations, or whenever paving operations are halted long enough to form a cold joint. Proper design and construction of transverse construction joints is essential to maintain continuity of the CRCP.

Figure 17 shows a transverse construction joint detail. A minimum of 1.0% of longitudinal reinforcement should be provided at transverse construction joints by placing additional reinforcement bars along the joint. Deformed bars 72-in (1.8-m) long and with the same size and grade of the longitudinal reinforcement are typically used to reinforce the transverse construction joint. These tiebars are placed between every other longitudinal bar to provide the required additional reinforcement.

The additional reinforcement is provided to resist the increased shear and bending stresses at the joint, and to provide additional bond area required to accommodate stresses generated during the first few days after construction, before the concrete gains sufficient strength.

In addition, lap splices that fall within 3 ft (0.9 m) behind the construction joint, or lap splices that fall within 8 ft (2.4 m) ahead of the construction joint (in the direction of paving), should be additionally strengthened. It is recommended that the lap either be made double the normal length, or else additional deformed bars 6 ft (1.8 m) long of the same size as the longitudinal reinforcement be spliced in symmetrically with the lap. (2)

This is especially important for transverse construction joints in order to produce high-quality concrete on both ends of the joint. Many transverse construction joints have performed poorly due to inadequate concrete consolidation.

Transition or Terminal Joints

Transition or terminal joints are provided in CRCP to accommodate pavement growth that, if uncontrolled, may close the expansion joint in the approach to structures and induce damage to the adjacent structure. Pavement growth is typically a result of expansion changes in the concrete pavement coupled with intrusion of incompressibles at the cracks. Areas with high precipitation have been found to experience this more frequently since precipitation produces a higher accumulation of incompressibles at the cracks.

Two types of joints are commonly used to prevent excessive CRCP movement: terminal end anchors and wide-flange beams. Terminal end anchors attempt to restrain movement, while wide-flange beams are effective in accommodating movement.

Terminal End Anchor Joints

End anchors were originally developed to restrain movement in jointed pavements. In fact, their use would be more justified on jointed pavements since on this type of pavement there is an increased chance of intrusion of incompressibles at the joints with the consequent pavement growth. The practice of using anchor lugs was carried over from jointed pavements to CRCP.

End anchors are provided by a series of concrete lugs underneath the pavement that anchor to the subgrade and attempt to restrain CRCP movement. The lug depth depends on the soil type and frost conditions. Soil types with low stiffness and thus low resistance to movement may compromise the effectiveness of the end anchors. In such situations, end anchor lugs should not be used. A design standard for terminal end anchors is provided in Figure 65 of Appendix A.
Regardless of the CRCP length, its central portion remains fully restrained by the underlying base and no change in length is experienced with exception of the last few hundred feet [typically 300 to 400 ft (90 to 120 m)] at the ends.

Wide-Flange Beam Joints

Wide-flange beam joints are increasing in popularity as a means to control CRCP end movements. Wide-flange beam joints provide room for CRCP expansion and provide a means to maintain joint load transfer efficiency. Figure 18 shows a typical design detail for a wide-flange beam joint. This joint is typically formed by casting a 10-in (250-mm) thick by 10-ft (3-m) long reinforced sleeper slab used to support the ends of the abutting pavements.

A wide-flange beam is embedded 5 to 6 in (127 to 152 mm) into the sleeper slab so that the top flange is flush with the CRC pavement surface. Compressible joint material such as polystyrene is provided on the side of the beam web, adjacent to the CRCP, to allow for expansion. In addition, a bond breaker is used on top of the sleeper slab to allow the pavement ends to move freely. It is important to provide adequate support for the sleeper slab or it may sink at one end.

It is recommended that corrosion protection be provided for the wide-flange beam, commonly using a corrosion inhibitor. To provide room for additional expansion, one or more slabs with doweled expansion joints are typically constructed between the CRCP transition joint and the bridge approach slab.

Some wide-flange beam joint premature failures have been reported in the past. The type of failure observed was separation of the top flange from the beam web. Because of this, it is now recommended to weld stud connectors to the top flange of the beam to prevent this type of failure. Recommended wide-flange beam sizes for various pavement thicknesses are provided in Table 5.

---

Table 5. Wide-flange (WF) beam (weight and dimensions) (56)

<table>
<thead>
<tr>
<th>Slab Thickness (mm)</th>
<th>Embedment in “Sleeper” slab (mm)</th>
<th>Size x Weight in lb/ft (mm x kg/m)</th>
<th>Flange Width in (mm)</th>
<th>Flange Thickness in (mm)</th>
<th>Web Thickness in (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 (203)</td>
<td>6 (152)</td>
<td>14 x 61 (356 x 91)</td>
<td>10 (254)</td>
<td>5/8 (16)</td>
<td>3/8 (9.5)</td>
</tr>
<tr>
<td>9 (229)</td>
<td>5 (127)</td>
<td></td>
<td>16 x 58 (406 x 86)</td>
<td>8.5 (216)</td>
<td>5/8 (16)</td>
</tr>
<tr>
<td>254 (10)</td>
<td>6 (152)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 (279)</td>
<td>5 (127)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 18. Wide-flange beam joint detail.
A recent FHWA study looked at the performance of existing CRCP sections in several States. In this study, fewer distresses such as spalling and faulting of the transition joint were observed on wide-flange beam joints than on terminal anchors. This may attributed in part to the younger ages of the wide-flange beams.(69)

Worth noting is that success has been achieved in providing continuity between the CRCP reinforcement and that in continuous span bridge structures. (62, 63) This is accomplished by tying the CRCP to both ends of the bridge, with some jointing modifications. It is believed that by successfully tying the CRCP to the bridge structure, the designer can provide for a smooth transition between these elements. One successful implementation of this technique was recently accomplished on Westlink M7 in Sydney, Australia. (63)

**Other Design Details**

The following paragraphs provide information on design details for crossovers, shoulders, ramps, and auxiliary lanes.

**Crossovers**

Crossovers are often used during construction to provide access to thru traffic. When crossovers are needed, it is recommended to pave the crossover in anticipation of the mainline paving requirement. Leaving gaps in the CRCP to provide room for crossovers is not recommended. When these gaps are paved, they are subjected to excessive movement exerted by the CRCP on both ends due to cyclic temperature changes. During the first days after placing, the leave-out concrete will not have reached its full strength, and will be more susceptible to slippage of the reinforcement.

![Figure 19. Layout of reinforcement in leave-out section.](58)
Paving these sections ahead of the mainline paving prevents reinforcement slippage since it is less unlikely that the short length of the paved crossover will exert excessive force on the fresh CRCP. It is recommended that special attention be given to crossovers when planning the paving schedule in order to minimize the need for leave-outs. Some States such as South Dakota include language in their specifications to discourage leave-out gaps.

An option worth considering is the use of a fast-track (high early strength) concrete in areas where a leave in/leave out would otherwise been used. Where this has been tried wet mat curing has been provided for 24 hours. This method would require prior coordination for the delivery of high early strength concrete in a timely manner so that all concrete within the crossover is high early strength.

When leaving a gap in the CRCP cannot be avoided, a minimum of 50 percent additional reinforcement should be required in the leave-out and across the construction joints on both ends of the leave-out. The additional reinforcement should be evenly distributed between every other regular reinforcement bar. Both additional and regular reinforcement bars should extend into the leave-out no less than 7 ft (2.1 m) and should be embedded no less than 3 ft (0.9 m) into the pavement ends adjacent to the leave-out. Splices in the leave-out area should follow the same requirements as those followed at a construction joint.

**Shoulders**

Shoulder types that are commonly used with CRCP include concrete and asphalt shoulders. While asphalt shoulders may require lower initial construction costs than concrete shoulders, the longitudinal joint between the CRCP and asphalt shoulder often requires significant maintenance activities during the life of the pavement. On the other hand, concrete tied shoulders provide enhanced lateral structural support resulting in a reduction in both pavement deflection and stress under traffic loading, leading to improved performance.

In order to provide increased structural support, concrete shoulders should be properly tied to the mainline pavement. While tied concrete shoulders paved in a second pass after the mainline pavement provide good lateral support, additional benefits are obtained from shoulders paved monolithically since a significant improvement in load transfer is achieved by aggregate interlock at the longitudinal joint with the mainline pavement.

Concrete shoulders for CRCP may be designed as either CRCP or JCP. Roller compacted concrete (RCC) has also been used. CRCP shoulders can provide a uniform pavement section that can later be fully utilized when additional lanes are required. Jointed concrete shoulders (with no reinforcement) may provide savings in comparison to CRCP shoulders in terms of initial construction cost, although future maintenance of the CRCP to JCP joint may be required. If JCP shoulders are selected, it is recommended to use short joint spacings (< 15 ft. (5m)) on the shoulder in order to minimize joint movement that could cause additional cracking in the CRCP mainline pavement.

Other factors that require consideration when selecting the shoulder type include the effect the shoulder will have on drainage as well as the effect that the environment will have on shoulder performance.

As an alternative to shoulders, experience has shown that by extending the outer concrete lane by at least 1 ft (0.3 m) into the shoulder to create a “widened lane,” this can significantly reduce edge deflections. The use of a widened lane can provide either additional pavement life or an opportunity to decrease the CRCP thickness. With these considerations, some States have opted to use widened lanes and asphalt shoulders. While this combination may not be as beneficial as monolithic tied concrete shoulders, it provides a trade off between initial construction costs and enhanced performance.

**Ramps and Auxiliary Lanes**

Selection of pavement type for auxiliary lanes between acceleration/deceleration lanes should take into account similar considerations as for shoulders in the previous section. In general, CRCP with the same characteristics as the mainline paving is also recommended for auxiliary lanes.

Pavement ramps, also termed acceleration/deceleration lanes, can be either CRCP or jointed pavement. It is typically recommended that ramps be tied to the mainline paving with the use of tiebars along the longitudinal joint, or by extending the transverse reinforcement from the mainline paving. Reinforcement design along the ramp to lane joint should follow the same guidelines as those provided for conventional longitudinal joints.

Performance of the longitudinal joint between the ramp and mainline paving will also depend on the differential movement between these two elements. If the ramp is constructed with jointed pavement, it is recommended to
provide a short joint spacing similar to jointed concrete shoulders to minimize movement and potential cracking of the CRCP. Furthermore, it is recommended to use jointed concrete pavement on the ramp if the distance between the CRCP end and the gore section of the ramp is less than 200 ft (60 m). This is to prevent excessive movement of the CRCP with respect to the ramp. Recommended layouts for ramp connections and jointing details are provided in Figure 20 and Figure 21.

Figure 20. Recommended layouts for ramp connections.

Figure 21. Jointing details for ramp connections.
Intersections present a unique challenge for CRCP design as maintaining continuity of reinforcement in both directions through the intersection may be required if two CRCP pavements are intersecting. A recent TxDOT research report documents best practices for design and construction of CRCP in transition areas, including intersections. As shown in Figure 22, the report provides design details used by TxDOT for maintaining reinforcement continuity in both directions through an intersection.

Note that the longitudinal reinforcement for the pavement in one direction provides the transverse reinforcement for the pavement in the other direction and vice versa. The report also provides design details for isolating intersecting CRC pavements when maintaining reinforcement continuity in only one direction is necessary, as well as intersection details for CRC pavements intersecting non-CRC pavements.

Figure 22. Design detail for CRCP intersection.\(^{67}\)
APPENDIX B:
AASHTO-86/93 GUIDE
REINFORCEMENT DESIGN
Longitudinal Reinforcement Design Inputs

The inputs required in this procedure include:

- Concrete tensile strength, $f_t$
- Concrete CTE, $\alpha_c$
- Concrete shrinkage at 28 days, $\zeta$
- Reinforcing bar diameter, $\phi$
- Steel CTE, $\alpha_s$
- Design temperature drop, $\Delta T_D$, and
- Wheel load tensile stress, $\sigma_w$

Concrete Tensile Strength

The concrete tensile strength determined through the ASTM C 496 or AASHTO T 198 splitting tensile test performed at 28 days is used in this procedure. A correlation of 86% of the third-point loading flexural strength used for thickness design may be assumed to determine the concrete tensile strength.

Concrete Coefficient of Thermal Expansion

Table 7 provides typical values of CTE for concrete made with different aggregate types. This information is derived from “Mass Concrete for Dams and Other Massive Structures,” Proceedings, Journal of the American Concrete Institute, Vol. 67, April 1970, as referenced in AASHTO, 1993.(5)

Concrete Drying Shrinkage

Values for 28-day concrete shrinkage as a function of splitting tensile strength may be obtained from Table 8.

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Concrete CTE $\times 10^{-6}$ in/in/$^\circ$F (m/m/$^\circ$C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>6.6 (11.9)</td>
</tr>
<tr>
<td>Sandstone</td>
<td>6.5 (11.7)</td>
</tr>
<tr>
<td>Gravel</td>
<td>6 (10.8)</td>
</tr>
<tr>
<td>Granite</td>
<td>5.3 (9.5)</td>
</tr>
<tr>
<td>Basalt</td>
<td>4.8 (8.6)</td>
</tr>
<tr>
<td>Limestone</td>
<td>3.8 (6.8)</td>
</tr>
</tbody>
</table>

Reinforcing Bar Diameter

Typical bar sizes used in CRCP range from # 4 (0.5 in) to # 7 (0.875 in) [#13 (13 mm) to # 22 (22 mm)].

Steel Coefficient of Thermal Expansion

A steel CTE of $5 \pm 10^{-6}$ in/in/$^\circ$F (9 $\pm 10^{-6}$ m/m/$^\circ$C) is recommended for longitudinal reinforcement design unless information on the steel CTE is available.

Design Temperature Drop

The design temperature drop is determined based on the average concrete curing temperature after placement and the lowest temperature of the year expected in the region where the CRCP will be constructed. The guidance provided in Section 3.5 may be used for this purpose.

Wheel Load Tensile Stress

Wheel load tensile stresses due to construction traffic during the early age may impact the crack spacing pattern and are therefore accounted for in longitudinal reinforcement design. The wheel load stress may be obtained from Figure 75 as a function of the design slab thickness, magnitude of wheel load, and effective k-value.

Longitudinal Reinforcement Design Procedure

The AASHTO-86/93 Guide includes design of the longitudinal reinforcement based on a desirable range of crack spacing, maximum crack width, and maximum steel stress.

Longitudinal reinforcement is designed to meet the following three criteria:

1. Amount of reinforcement required to produce desirable crack spacings. Recommended crack spacings to

<table>
<thead>
<tr>
<th>Indirect Tensile Strength psi (MPa)</th>
<th>Shrinkage in/in (m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300 (2.07) or less</td>
<td>0.00080</td>
</tr>
<tr>
<td>400 (2.76)</td>
<td>0.00060</td>
</tr>
<tr>
<td>500 (3.45)</td>
<td>0.00045</td>
</tr>
<tr>
<td>600 (4.14)</td>
<td>0.00030</td>
</tr>
<tr>
<td>700 (4.83) or greater</td>
<td>0.00020</td>
</tr>
</tbody>
</table>
minimize potential for development of punchouts and spalling range between 3.5 and 8 ft (1.1 and 2.4 m).

2. Amount of reinforcement required to keep transverse cracks tightly closed. Maximum allowable crack widths applicable to this procedure should not exceed 0.04 in (1.0 mm) to prevent spalling and water infiltration (although better practices would limit this to 0.02 in (0.5 mm)).

3. Amount of reinforcement required to keep reinforcement stresses within allowable levels. Keeping the steel working at an acceptable stress level minimizes fracture of the steel or excessive yield that may lead to wide cracks with poor load transfer efficiency. Maximum allowable working stress for steel Grade 60 (Grade 420) recommended by AASHTO-86/93 as a function of indirect tensile concrete strength and reinforcing bar diameter is provided in Table 9.

With the design inputs obtained following the guidelines in Section 3, and the above limiting criteria, the longitudinal reinforcement design is accomplished following the procedure described below:

Figure 75. Chart for estimating wheel load tensile stress.
The amount of longitudinal steel reinforcement to satisfy each limiting criterion is obtained using the design charts in Figure 76, Figure 77, and Figure 78. The minimum required steel percentage \( P_{\text{min}} \) corresponds to the largest obtained among the crack spacing of 8 ft (2.4 m), crack width, and steel stress criteria. The maximum required steel percentage \( P_{\text{max}} \) corresponds to the crack spacing of 3.5 ft (1.1 m). The design charts are accessed by drawing a continuous line intersecting the selected design values on the various scales to determine the percent steel. (In some cases, it may be necessary to extend the turning lines to the top or bottom of the chart.)

- If \( P_{\text{max}} \) is greater than or equal to \( P_{\text{min}} \), proceed to the next step. However, if \( P_{\text{max}} \) is less than \( P_{\text{min}} \), the design is considered unsatisfactory and revision of the selected inputs should be made until \( P_{\text{max}} \) is greater than \( P_{\text{min}} \).

Figure 76. Minimum percent longitudinal reinforcement to satisfy crack width criterion.
Use the following equations to determine the range in the number of reinforcing bars or wires required:

\[
N_{\text{min}} = 0.01273 \cdot P_{\text{min}} \cdot W_s \cdot D \frac{1}{\phi^2}
\]

\[
N_{\text{max}} = 0.01273 \cdot P_{\text{max}} \cdot W_s \cdot D \frac{1}{\phi^2}
\]

where, \( N_{\text{min}} \) = Minimum number of reinforcing bars required,

\( N_{\text{max}} \) = Maximum number of reinforcing bars required,

\( P_{\text{min}} \) = Minimum required steel percentage,

\( P_{\text{max}} \) = Maximum required steel percentage,

\( W_s \) = Total width of pavement section (in or mm),

\( D \) = Slab thickness (in or mm), and

\( \phi \) = Reinforcing bar or wire diameter (in or mm).

Figure 77. Percent of longitudinal reinforcement to satisfy crack spacing criteria.

(1 in = 25.4 mm, 1 ft = 0.3048 m, 1 psi = 6.98 KPa)
• The total number of bars, $N_{\text{design}}$, is determined by selecting a whole number between $N_{\text{min}}$ and $N_{\text{max}}$. The final design is checked against the limiting criteria by converting $N_{\text{design}}$ to percent of steel and working backwards through the design charts.

Figure 78. Minimum percent longitudinal reinforcement to satisfy steel stress criteria.