New continuously reinforced concrete pavement designs

Prepared for Research and Development of Standards Division, Highways Agency

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Executive Summary

The aim of this project was to assist the Highways Agency (HA) in re-assessing current designs and specifications for continuously reinforced concrete pavement (CRCP) in the light of performance data available in the UK and in other countries. The findings of this work have been used to develop more economic designs for sustainable long-life roads, with reductions in the maintenance requirements, contract periods and traffic delays, which would give increased value for money and support the Government aims for sustainable construction.

The results from research have indicated a number of options for updating the current CRCP designs to provide sustainable long-life performance with significant economic benefits. The performance and design parameters investigated were crack patterns, concrete strength, steel reinforcement, foundations, hard shoulders and edge strips, traffic loading and terminations. The effects of these parameters on the structural integrity and durability of CRCP were assessed and the results were used to develop new design curves which will enhance pavement performance.

The performance of CRCP is mainly determined by the condition of the surface cracking and defects, with the greatest influence arising from medium, wide and bifurcated cracks, and localised punchouts. It was found that the aggregate type in CRCP has more influence on the cracking pattern than the subbase type. Higher percentages of medium, wide, spalled and bifurcated cracks occurred with coarse aggregate of siliceous gravel than with limestone. Locating the reinforcement at the third-depth of the slab significantly improves the crack pattern of CRCP made with siliceous gravel.

A review of international standards and practices has shown the widespread use of flexural strength rather than compressive strength for design purposes, which is considered to be a more suitable parameter for the structural performance of pavements. Reliable relationships between flexural and compressive strength were established and used to develop new CRCP designs with the possibility of reduced slab thickness compared to the current designs for similar traffic loading.

Cracks in CRCP provide the route for chlorides, from de-icing salts, to penetrate the slab and initiate reinforcement corrosion. Higher levels of chloride concentrations were found at the reinforcement level in the vicinity of cracks. Corrosion mainly occurs in the transverse reinforcement, which tends to be coincident with the transverse cracks, with no evidence of significant corrosion in the longitudinal reinforcement. Therefore, no significant consequences of corrosion damage on the performance of CRCP were found in the UK.

The currently specified cemented subbase under CRCP in the UK has significantly higher strength than that used in other countries. The high strength results in large crack spacings and wide cracks in the subbase, increasing the risk of discontinuity of the foundation support. The new designs consider lowering the subgrade strength requirement for a subbase only, without capping, to 5 per cent CBR, as well as the use of bound materials in Foundation Classes 2, 3 and 4, which should result in significant economic benefits. These foundation classes incorporate a wider range of cement and other hydraulically bound subbases than currently specified.

Recommendations are given against the use of asphalt shoulders alongside CRCP and to retain the maximum cumulative traffic loading given in the current designs to allow for the possibility of an increase in the maximum permitted axle load.

Site data on the thermal movements at CRCP terminations have indicated that the ground beam anchorage system restrains only 40 per cent of the CRCP end movement but requires little maintenance. For the wide-flange beam system, the CRCP end movement is mainly accommodated by the joint between the CRCP and the steel beam, and therefore there is a potential to reduce the number of transition bays to two when adjacent to a flexible pavement. The main problems associated with the performance of wide-flange beam terminations are debonding of the joint seal and fatigue of the beam. Recommendations are given to reduce the amount of thermal movement at terminations by locally increasing the subbase friction and/or reducing the coefficient of thermal expansion of the concrete. A proposal is made to develop a more economic termination system based on bridge-type joints.

1 Introduction

Continuously reinforced concrete pavement (CRCP) was developed to overcome long-term performance problems associated with the old generation of jointed concrete pavements. CRCP is the thinnest concrete option for the same cumulative traffic loading and has the added advantage of eliminating the need for expansion and contraction joints in the main slab. Thermal stresses within the CRCP slab are relieved by transverse cracks, which are held tightly closed by the continuous longitudinal reinforcement to ensure good aggregate interlock. The aggregate interlock results in a high level of load transfer efficiency across the cracks, maintaining the structural performance of the pavement. The amount of water penetrating into the pavement and the associated pumping of fine materials under traffic loading are also reduced, leading to enhanced foundation durability. The enhanced structural performance and durability of CRCP, compared to jointed pavements, has the capability to reduce the stringent requirements for foundations under rigid pavements, allowing the use of a wider range of foundation materials, reducing the initial construction costs, and supporting the Government policy of sustainable construction.

The practice of constructing concrete roads with continuous reinforcement was initiated in the USA in the 1920s and further developed in Belgium in the 1950s. In the UK, the earliest construction of CRCP was the M62 in 1975, using designs that were current in the USA and Belgium but adapted to UK practice.

CRCP has many benefits, such as long life durability, low maintenance and contributes to environmental protection and sustainable development. The high stiffness of concrete provides a good distribution of traffic loading leading to low stresses in the underlying materials. The concrete can be made with a wide range of materials, including recycled and secondary aggregates and binders, which makes best use of available materials, conserves primary materials and contributes to sustainable construction. It is a durable material and has a long service life, with the potential to last for an indeterminate period. Concrete has the advantage of being a stable construction material, resistant to fuel spillage, non-flammable and non-toxic. Concrete is not damaged by vehicle fire and it generally maintains its shape and properties. It does not emit harmful fumes, has a high fire safety factor and does not contribute to the fire load. These factors contribute to concrete being the preferred EU surface material for tunnels.

In spite of the many advantages of CRCP, its use in the UK is relatively limited due to the perception of noise associated with concrete pavements and the high initial construction costs. The Government has recognised the noise problem from the traffic/road surface interface and stated in ‘Transport 2010, The Ten Year Plan’ published by the Department of the Environment, Transport and the Regions (DETR, 2000) that new and existing lengths of concrete pavements in England will be required to be covered with a quiet surface. Recent work at TRL has indicated that CRCP with an exposed aggregate concrete surface (EACS) provides a quieter surface than hot rolled asphalt (HRA) surfacing (Chandler *et al.*, 2003). Whilst the new thin asphalt surfacing materials are initially quieter than EACS, their long-term noise reduction may not be maintained as well as that of EACS. The economic issues associated with the initial construction costs are discussed within the context of revising the current designs and the utilisation of alternative materials for CRCP.

The current designs for CRCP given in HD26/01 of Volume 7, 7.2.3, of the Design Manual for Roads and Bridges (DMRB) (Highways Agency *et al.*) are based on the compressive strength of concrete and performance data obtained from jointed concrete pavements prior to the mid 1970s. These data were used by Garnham (1989) to derive the CRCP designs. This report reviews the current designs for CRCP in the light of performance data available both in the UK and in other countries, and highlights sustainability and economic benefits from the use of alternative materials and improved design procedures. The material properties and design parameters considered are the concrete flexural strength, reinforcement corrosion and steel fibre reinforcement, foundations, shoulders and edge strips, traffic loading, and the terminations to CRCP.

2 Performance of CRCP in the UK

A well designed and constructed CRCP can provide excellent performance with minimal maintenance. The performance of a CRCP is influenced by many factors including the properties of the subbase, concrete and reinforcement, the environmental conditions at the time of paving and the method of construction. The condition of CRCP is mainly assessed by readily observed features such as crack pattern, crack spacing, crack width, amount of spalling and bifurcation, and defects such as punchouts.

2.1 Background

Roads constructed as a CRCP have no intermediate expansion or contraction joints and are designed to relieve thermal stresses by a series of transverse cracks as shown in Figure 2.1. Jimenez *et al*. (1992) indicated that the optimum transverse crack spacing in CRCP should be in the range 1.1 to 2.4m. Larger crack spacings increase the probability of the cracks being wide, resulting in reduced aggregate interlock and consequently increased stresses in the pavement slab and longitudinal reinforcement. In contrast, crack spacings smaller than the optimum will result in a series of transverse beams and a load distribution in the transverse direction of the pavement, which could lead to high induced stresses, punchouts and eventually localised structural failure.

There is no clear relationship between crack spacing and performance. Peshkin *et al.* (1993) reported that many CRCP roads in Illinois which have crack spacings between 0.6 and 1.5m have performed exceptionally well. In Belgium, which specifies a high percentage of longitudinal steel, 0.7 to 0.85 per cent, the average transverse crack spacing is small, 0.4 to 0.75m, and the CRCP achieved an excellent 20-year performance (Verhoeven and Van Audenhove, 1994).

Figure 2.1 Transverse cracking in CRCP

Crack width is a more sensitive indicator of the pavement condition than the average crack spacing. Transverse cracks in CRCP are kept tightly closed by the longitudinal steel reinforcement. This ensures structural continuity by good load transfer across the cracks and less surface water penetrating to the underlying layers. Based on considerations of spalling and water penetration through cracks, the American Association of State Highway Transportation Officials (AASHTO, 1986) suggested a maximum allowable crack width of 1mm. Crack widths greater than 1mm result in loss of aggregate interlock and load transfer efficiency. Peshkin *et al.* (1993) showed that a loss of aggregate interlock may occur at openings as small as 0.7mm, and suggested maximum crack widths in the order of 0.5mm to reduce the incidence of punchouts. From a durability viewpoint, McCullough (1997) indicated that surface water can flow through a crack width of only 0.25mm, and that the flow can increase dramatically for crack widths greater than 0.5mm.

Spalling of the arris generally occurs at wide cracks due to the action of passing traffic. This may remain as an insignificant minor defect or may develop into a more severe condition leading to a loss of aggregate interlock and punchout distress. Spalling is also influenced by the type of coarse aggregate. Aggregate with a relatively high bond strength with the mortar, such as limestone, usually exhibits less spalling than a siliceous gravel aggregate. However, as spalling is considered to be predominately caused by the application of wheel loadings across the crack, overlaying new CRCP with a thin surfacing system should overcome this problem.

Localised distress occurs in CRCP where the concrete breaks up into pieces, resulting in a punchout or a punchdown, as shown in Figure 2.2. This may be the result of box cracking, where very closely spaced transverse cracks are intersected by longitudinal cracks, or within an area intersected by bifurcated cracks. Inadequate compaction of concrete, especially under the reinforcement where the longitudinal bars are lapped, is also a primary cause of localised distress. Two specific design features have been associated with the occurrence of punchouts and punchdowns. One is inadequate steel content, which, according to Lee and Darter (1995), often results in more punchout failures. The other is the amount of subbase support, which is critical to the performance of a CRCP as loss of support can lead to a loss of load transfer and punchout distress.

Figure 2.2 Localised failure due to punchouts

2.2 Inspection of UK roads

In the UK, 43 sections at ten CRCP sites were selected for visual condition survey and inspection of cracks and defects. The sites included motorways and trunk roads on the national road network and were constructed with different subbases, coarse aggregate in the concrete and depths of the reinforcement from the slab surface. The surveys were carried out in the nearside traffic lane, Lane 1, along lengths of road between 98m and 662m. For consistency, the results from the survey have been expressed for 100m lengths and are given in Table 2.1. for sections with the reinforcement located at mid-depth.

The results in terms of the number of transverse cracks, average transverse crack spacing, percentages of cracks in each width category, percentages of spalled and bifurcated cracks and a new parameter, the *crack index*, are presented with respect to the different coarse aggregate type; siliceous gravel or limestone, and also subbase type; unbound, asphalt or cemented. The width of a crack was classified hair, narrow, medium or wide in accordance with the HD29/94 of the DMRB 7.3.2 (Highways Agency *et al.*). Hair and narrow cracks are defined as those with a width up to 0.5mm, with hair cracks observed only with difficulty. Medium cracks are between 0.5mm and 1.5mm and wide cracks are greater than 1.5mm. The new parameter, the *crack index,* has been introduced to quantify the summation of all the transverse crack widths, and is expressed in terms of mm of crack opening per 100m of road length (mm/100m). This index was calculated by assigning each category of crack width a representative value of width and multiplying these widths by the number of transverse cracks in each of the categories. The average crack width value assigned to each category of hair, narrow, medium and wide crack was 0.1mm, 0.25mm, 1.0mm and 2.0mm, respectively.

Table 2.1 Transverse cracking per 100m length of CRCP

Aggregate	No. of sections	Subbase type	No. of cracks	Average crack spacing (m)	Percentage of total cracks H	N	M	W	S	B	Crack index (mm)
Siliceous gravel	1	Unbound	67	1.5	0	70	30	0	60	3	32
	2	Asphalt	111	0.9	32	53	15	0	47	25	34
	8	Cemented	95	1.1	14	64	20	2	37	15	38
Limestone	4	Unbound	36	2.7	1	84	15	0	8	6	13
	12	Asphalt	41	2.4	23	75	2	0	5	6	10
	11	Cemented	53	1.8	8	89	3	0	15	10	16

H = Hair, observed only with difficulty. *N = Narrow, up to 0.5mm in width.*
M = Medium, between 0.5 and 1.5mm in width. *W = Wide, greater than 1.5mm in width.*
S = Spalled crack. *B = Bifurcated crack.*

The long-term performance of CRCP is mainly influenced by the medium and wide cracks since these cracks are associated with loss of aggregate interlock, compromising structural integrity, and make the pavement more vulnerable to reinforcement corrosion and deterioration of the foundation, compromising durability. In general, all the inspected CRCP sites exhibited good performance, with an average crack spacing between 0.9m and 2.7m, and with little maintenance having taken place, the exception was some sections of early constructed roads which had been designed prior to the CRCP thickness designs given in HD26/01 of the DMRB 7.2.3 (Highways Agency *et al.*).

2.2.1 Subbase type

Table 2.1 shows that for the siliceous gravel CRCP, the average transverse spacing of the unbound subbase was higher than that of the asphalt and cemented subbases. The average transverse crack spacing was 1.5m, 0.9m and 1.1m for the unbound, asphalt and cemented subbases, respectively. The unbound subbase was shown to have the lowest percentage of bifurcated cracks, only 3 per cent compared with 15 and 25 per cent for the cemented and asphalt subbase, respectively. The highest percentage of cracks categorised as medium and wide, and spalled cracks occurred in the unbound subbase.

For the limestone CRCP, the unbound and asphalt subbases resulted in the largest values of average transverse crack spacing. The average transverse crack spacing was 2.7m, 2.4m and 1.8m for the unbound, asphalt and cemented subbases, respectively. The results also show that the unbound subbase exhibited a much higher percentage of medium cracks than the other subbase types. The percentage of bifurcated cracks was highest for the cemented subbases being 10 percent compared to 6 per cent for the unbound and asphalt subbases. The percentages of spalled cracks were 8, 5 and 15 per cent for the unbound, asphalt and cemented subbases, respectively.

2.2.2 Aggregate type

The results in Table 2.1 show a general trend of larger crack spacing for concrete with the limestone aggregate. Regardless of the subbase type, the average crack spacing for the siliceous gravel aggregate ranged from 0.9 to 1.5m compared to 1.8 to 2.7m for the limestone aggregate.

Table 2.1 also shows that the aggregate type has more influence on the *crack index* than the subbase type. The range of crack index for the siliceous gravel aggregate was between 32 and 38mm/100m, and on average was about two and a half times higher than that of the limestone aggregate, which was between 10 and 16mm/100m. It can also be seen that a higher percentage of defects, in terms of crack spalling and, with the exception of the unbound subbases, bifurcations were more associated with the siliceous gravel than with the limestone aggregate.

The percentages of transverse cracks in each width category have been extracted from Table 2.1 and are given in Figure 2.3. This shows that only 2 per cent of the cracks were wide and were observed in CRCP with the siliceous gravel aggregate, no wide cracks occurred with limestone aggregate. The percentage of medium cracks ranged between 15 and 30 per cent for the siliceous gravel aggregate, which is much higher than the 2 to 15 per cent for the limestone aggregate. Conversely, a higher percentage of narrow cracks were found with the limestone aggregate, 75 to 89 per cent, than with the siliceous gravel, 53 to 70 per cent. Long-term performance of the CRCP is likely to be better when transverse cracks are hair or narrow, rather than medium and wide.

The improved performance of the limestone concrete is mainly attributable to the lower thermal coefficient of expansion of the limestone and the improved bond of the angular limestone aggregate with the mortar compared to that achieved with the smooth surface of siliceous gravel coarse aggregate. During the inspections, it was noticed that in CRCP with siliceous gravel the cracks generally propagated around the aggregate particles, resulting in aggregate becoming loose and plucking out of the matrix, especially in the wheel paths. However, with limestone aggregate the cracks were generally contained either within the mortar or passed through the aggregate particles and the aggregate remained bonded with the mortar matrix.

2.2.3 Reinforcement depth

The effect of reinforcement depth, determined from construction data, on transverse cracking of CRCP is given

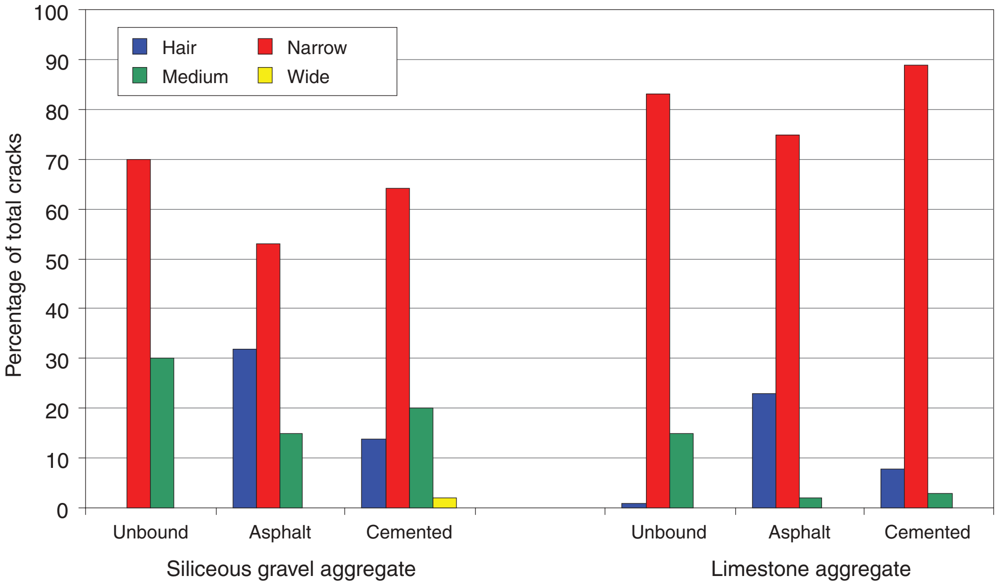

Figure 2.3 Percentage of cracks in each category for gravel and limestone CRCP on different subbases

in Table 2.2. Comparisons were made between two types of aggregate on cemented subbases. For the siliceous gravel aggregate the results show that locating the reinforcement at third-depth, rather than at mid-depth, reduced the average crack spacing from 1.3m to 1.0m. A reduction of at least 75 per cent was achieved for cracks categorised as medium and wide, approximately 50 per cent for spalled, and approximately 25 per cent for bifurcated. As a result, the crack index was significantly reduced from 47 to 32mm/100m.

Table 2.2 shows a similar trend for the limestone aggregate, the average crack spacing was reduced from 1.4m to 1.2m, the 5 per cent of cracks categorised as medium was reduced to 0 per cent, and the spalled and bifurcated were reduced by approximately 50 per cent. No wide cracks were observed in the limestone CRCP. However, the distribution of crack widths has resulted in little difference for the *crack index* between the third-depth and the mid-depth results.

Locating the steel reinforcement nearer to the slab surface, at third-depth rather than at half-depth, decreases the average crack spacing, and has the benefit of reducing the percentage of cracks categorised as medium or wide. It also reduces the percentage of spalled and bifurcated cracks, leading to a better crack pattern. However, in this position the steel could be more vulnerable to corrosion because of the reduced depth of concrete cover, particularly as de-icing salts can penetrate even through narrow cracks.

The type of coarse aggregate in CRCP has more influence on the cracking pattern than the subbase type.

For the same concrete aggregate type, unbound subbases had a higher percentage of medium and wide cracks than asphalt and cement bound subbases.

Higher percentages of medium, wide, spalled and bifurcated cracks occurred in CRCP containing siliceous gravel aggregate than limestone aggregate.

The *crack index* of siliceous gravel CRCP was about two and a half times greater than that of limestone CRCP.

Locating the reinforcement at a third of the slab depth significantly improves the crack pattern of siliceous gravel CRCP. However, there could be an increase in the risk of corrosion from de-icing salts.

Table 2.2 Effect of reinforcement depth on cracking per 100m length of CRCP

Aggregate	*No. of sections*	*Reinforcement location*	*No. of cracks*	*Average crack spacing (m)*	*Percentage of total cracks* H	N	M	W	S	B	*Crack index (mm)*
Siliceous gravel	2	Mid-depth	75	1.3	6	51	37	6	43	13	47
	2	Third-depth	107	1.0	11	80	9	1	22	9	32
Limestone	3	Mid-depth	71	1.4	11	84	5	0	10	10	20
	3	Third-depth	84	1.2	22	78	0	0	5	6	19

H = Hair, observed only with difficulty.
M = Medium, between 0.5 and 1.5mm in width.
S = Spalled crack.
N = Narrow, up to 0.5mm in width.
W = Wide, greater than 1.5mm in width.
B = Bifurcated crack.

3 Concrete strength

The current UK designs for rigid and rigid composite pavements given in HD26/01, DMRB 7.2.3 (Highways Agency *et al*.), are based on empirical data for reinforced and unreinforced concrete pavements published in RR87 (Mayhew and Harding, 1987). These data were obtained before the mid 1970s, and the design approach considered the compressive strength of the concrete to be the criterion to determine the slab thickness. Nowadays, the properties of concrete are improved from that manufactured many years ago, and therefore need to be taken into consideration in revising the current designs.

The structural performance of concrete pavements may be considered to be more related to the flexural strength than the compressive strength. Under axial loads, bending of the concrete results in both compressive and flexural stresses. Concrete is stronger in compression than in flexure, and therefore the flexural strength is more critical and has a greater influence on slab thickness design.

Flexural strength has been widely used in other countries and adopted in many specifications for the design of concrete pavements. The British Airport Authority (BAA) specifies only concrete with crushed rock aggregate, because this produces a higher ratio of flexural to compressive strength than when using siliceous gravel (BAA, 1993). The Permanent International Association of Road Congresses (PIARC, 1994) presented data on the standards and practices for concrete roads in many countries, which are given in Table 3.1. This shows that nine of the twelve countries in the review use flexural strength, whereas three use compressive strength only; Great Britain, Belgium and the Netherlands.

Table 3.1 Synoptic Table on strength specifications for concrete roads (PIARC, 1994)

Country	Flexural strength	Other strengths: Compressive	Other strengths: Tensile
Austria	✓	✓	
Belgium		✓	
France	✓		✓
Germany	✓	✓	
Great Britain		✓	
Italy	✓	✓	
Japan	✓		
Netherlands		✓	
Norway	✓	✓	
Spain	✓		
Sweden	✓		✓
Switzerland	✓	✓	

3.1 Flexural strength

The strength properties of concrete are mainly affected by the properties of its constituents; the aggregate and binder, and the interface between them. When comparing the strength properties of concrete and mortars, work by Kaplan (1959) indicated that the inclusion of coarse aggregate reduces the flexural strength and increases the compressive strength of concrete. The reduction of flexural strength was attributed to the increased aggregate/binder interface and the improvement in compressive strength attributed to the mechanical interlocking of the coarse aggregate.

Recent developments in concrete technology have led to the production of ‘high strength concrete’ compared to the old ‘normal strength concrete’. Nowadays, the refinement of the cement manufacture and composition, the use of cement replacement materials, the use of chemical admixtures, such as water reducing admixture, could significantly improve the packing capacity of the mixture, resulting in a dense, high strength concrete. Improving the characteristics of the aggregate/binder interface has greater influence on the flexural and tensile strengths than the compressive strength of concrete.

Aggregate, for use in concrete, is traditionally specified by a combination of physical and mechanical properties with the assumption that the higher the strength of aggregate the higher the strength of the concrete. However, this concept is not always valid and can restrict the wider use of alternative aggregates in concrete. An example of this is that siliceous gravel aggregate usually exhibits superior strength properties and lower porosity than limestone aggregate. However, when incorporated in concrete, the limestone aggregate gives higher strength properties and improved performance compared to siliceous gravel (Hassan *et al.*, 1998). Therefore, the strength and performance properties of concrete are not limited to the strength properties of the aggregate, but rather a combination of surface texture, mineralogy, particle shape and optimisation of the concrete mixture. French experience indicated no strong correlation between the concrete strength and the strength of the aggregate (Voirin *et al*., 2003). A higher rate of strength development was mainly associated with calcareous aggregate due to improved properties at the aggregate/binder interface.

3.2 Flexural and compressive strength relationship

Site and laboratory data were used to establish the relationship between the flexural and compressive strength of concrete. Data were obtained from recently constructed sites and from a laboratory study carried out to investigate the effects of aggregate and binder types on the strength properties of concrete. Two coarse aggregates were used; siliceous gravel and limestone, and four binder types; Portland cement, silica fume, ground granulated blast furnace slag and fly ash. The concrete mixtures were made to satisfy the current UK specifications for pavement quality concrete given in Volume 1, Specifications for Highway Works (SHW), of the Manual of Contract Documents for Highway Works (MCHW1) (Highways Agency *et al*.) and to replicate typical road construction mixtures with low consistence (workability).

Figure 3.1 shows the derived 28-day relationships between flexural and compressive strength for concrete made with either siliceous gravel or limestone aggregates. The results show clearly that for the same compressive strength of concrete, limestone aggregate gives higher flexural strength than the siliceous gravel aggregate, and that this difference reduces as the compressive strength increases.

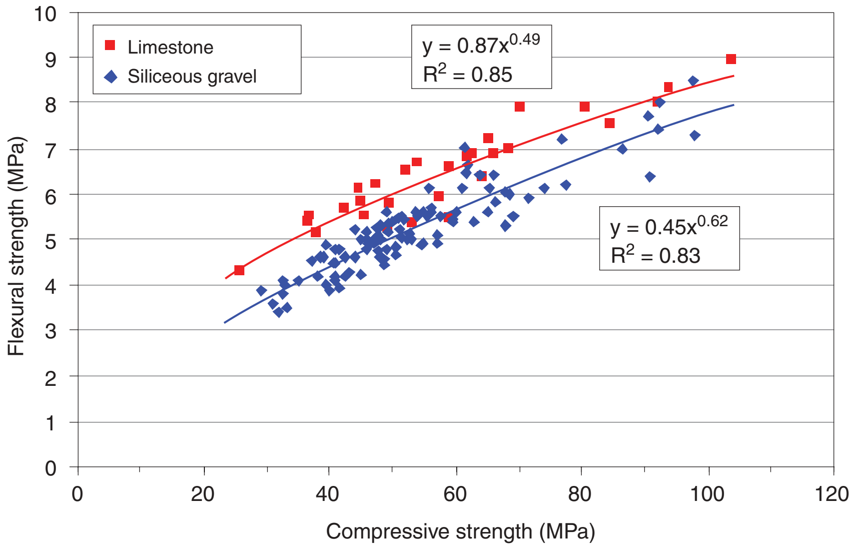

Figure 3.1 Relationship between flexural and compressive strength at 28 days

The relationship between flexural and compressive strength for the data set considered is:

Siliceous gravel $f_f = 0.45\ (f_{c,\ cube})^{0.62}$ (3.1)

Limestone $f_f = 0.87\ (f_{c,\ cube})^{0.49}$ (3.2)

Where: f_f = 28-day flexural strength (MPa).
$f_{c,\ cube}$ = 28-day compressive strength (MPa).

The 28-day strength data for four CRCP sites constructed between 1999 and 2003 was validated against all the data shown in Figure 3.1. One site used siliceous gravel aggregate in the concrete and the other three used limestone aggregate. Depending on the aggregate used, the calculated average flexural strength for a site was obtained from Equation 3.1 or Equation 3.2. Table 3.2 gives results of the average compressive strength and the measured and calculated flexural strength values for each site.

The results show that the percentage difference between the mean measured and the calculated flexural strength was small, and ranged between -4 per cent and +5 per cent. This indicates that the relationships developed between flexural and compressive strength given in Equations 3.1 and 3.2 may be used with confidence for developing the new CRCP design curves based on the flexural strength of concrete.

Table 3.2 Validation of the flexural strength relationships to site data

		Measured strength			
Aggregate type	*No. of specimens*	*Compressive f_c (MPa)*	*Flexural f_f (MPa)*	*Calculated flexural strength* f_{calc} (MPa)*	*Percentage difference $f_{calc} - f_f$ (%)*
Site 1: Siliceous gravel	50	48.0	4.9	5.0	2
Site 2: Limestone	4	55.0	6.1	6.2	2
Site 3: Limestone	6	68.5	6.6	6.9	5
Site 4: Limestone	12	66.5	7.1	6.8	-4

* *From Equation 3.1 or Equation 3.2.*

3.3 CRCP thickness designs based on flexural strength

The CRCP thickness design curve, in the HD26/01 (DMRB 7.2.3), was derived by Garnham (1989) from the regression equations given in RR87 (Mayhew and Harding, 1987), using the compressive strength of concrete. This curve is based on a grade C40 concrete, now designated as Class C32/40 concrete, and was assumed to have a mean 28-day compressive strength of approximately 50MPa.

The data used in RR87 were obtained from sites constructed prior to the mid 1970s, and the concretes were made with siliceous gravel coarse aggregate. A comparison of the flexural to compressive strength relationship for these old concretes with that of the modern concrete with siliceous gravel concrete, given in Equation 3.1, is shown in Figure 3.2. This clearly shows that for the same aggregate and compressive strength, the modern concrete gives a higher flexural strength, between 11 and 14 per cent, than the older concrete. For example, for a compressive strength of 50MPa, the modern concrete has a flexural strength of 5.1MPa compared to 4.5MPa for the old concrete. Therefore, more economic designs for CRCP can now be achieved.

An example of the possible reduction of CRCP thickness is shown in Figure 3.3 by comparing the thickness curve for CRCP given in HD26/01 (DMRB 7.2.3) with the thickness curve for modern concretes. This uses the flexural/compressive relationship for siliceous gravel developed in Equation 3.1 and assumes the mean 28-day compressive strength of the concrete is 50MPa. For a concrete pavement designed to carry a traffic of 400 million standard axles (msa), the new designs reduce the slab thickness given in the HD26/01 (DMRB 7.2.3) by approximately 20mm, when rounded up to the nearest 10mm.

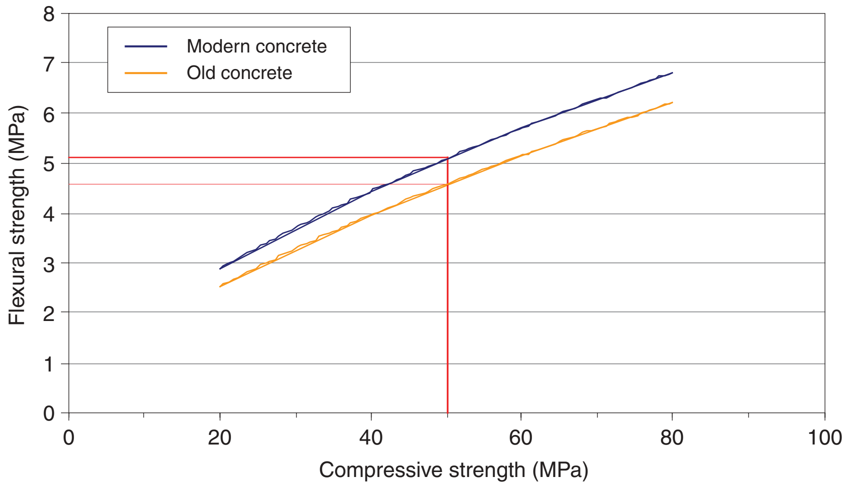

Figure 3.2 Relationship between 28-day flexural and compressive strength for old and modern concrete made with siliceous gravel

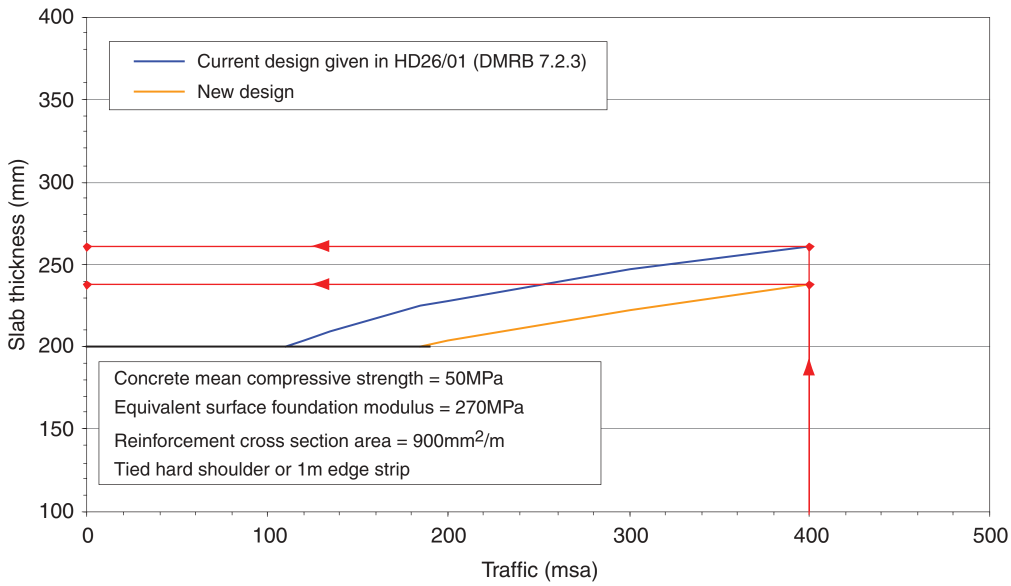

Figure 3.3 CRCP slab thickness for the current and proposed design

The current design curve is based on a single concrete strength and does not give benefit for any concrete of higher strength. To obtain the advantage of a higher flexural strength, designs for various levels of flexural strength between 4.5MPa and 6.0MPa are incorporated in the new design curves as shown in Figure 3.4. These curves are valid for all aggregate types and conservatively use the relationship between flexural and compressive strength for siliceous gravel concrete and are based on the mean flexural strength of the concrete at 28 days.

Figure 3.4 shows the required slab thickness for different flexural strength of the CRCP with a tied shoulder or a one metre edge strip designed to carry a traffic loading up to 500msa. At 400msa and when rounded up to the nearest 10mm, a slab thickness of 270, 240, 220 or 200mm would be required for a mean concrete flexural strength of 4.5, 5.0, 5.5 or 6.0MPa, respectively. A reduction of slab thickness of approximately 70mm could be achieved by increasing the flexural strength from 4.5 to 6.0MPa. If the minimum thickness of pavement is as currently limited to 200mm, a flexural strength higher than the 6.0MPa will not gain any further benefit of thickness reduction up to 500msa.

3.4 Quality control and compliance

Although flexural strength is proposed for the design, it is assumed that early age cube testing will still be used by the contractor for quality control purposes. The flexural strength test is not very practical due to the relatively large specimens needed, especially when the maximum aggregate size is 40mm, which could increase the risks associated with health and safety.

The European Standard EN 13877-2 (2004) requires compliance to be determined from cores, extracted from the finished pavement. Therefore, the contractor will have to establish a reliable relationship between flexural and

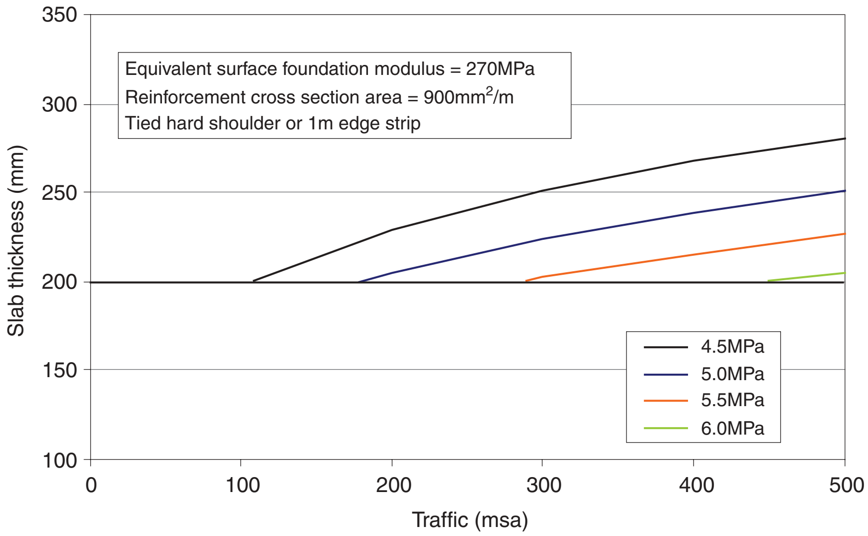

Figure 3.4 CRCP thickness design curves based on concrete flexural strength

compressive strength that can be used with confidence when assessing the cubes for quality control and the cores for compliance.

The designs will be specified by a characteristic 28-day concrete flexural strength. Further work is required to establish the standard deviation for determining the characteristic from the target or mean flexural strength.

Current UK designs are based on the compressive strength and used relationships from data for old concrete.

The structural performance of a concrete pavement is more related to the flexural strength.

Relationships between flexural and compressive strengths were established and validated to develop new CRCP design curves based on the flexural strength of concrete.

The aggregate type greatly influences the ratio of compressive to flexural strength of concrete.

The new CRCP designs should be more economic than the current designs through a reduction in the slab thickness.

4 Steel reinforcement

Continuous longitudinal reinforcement has the benefit of holding the transverse cracks tightly closed to ensure high load transfer across the cracks and improve the structural integrity of the pavement. The reinforcement adds to the initial cost but the superior long-term performance and thinner pavement thickness required make CRCP cost-effective. However, there is a need to balance the amount of the steel with the concrete strength, and to determine the most suitable location of the steel to ensure a satisfactory crack pattern and performance.

4.1 Percentage of reinforcement

There must be a correct balance between the properties of the concrete and the steel for the pavement to behave in a satisfactory manner. A high percentage of steel induces small crack spacings with narrow transverse cracks, and for the same percentage of reinforcement, a larger longitudinal bar diameter results in wider cracks (Jimenez *et al.,* 1992). A low percentage of steel is mainly associated with large crack spacings and wide crack openings, which can lead to the loss of load transfer, increased stresses in the concrete slab, spalling and steel rupture.

The quantity of longitudinal reinforcement specified in many countries is between 0.6 and 0.7 per cent of the concrete cross-sectional area. In the UK, the current requirement given in the DMRB 7.2.3 is 0.6 per cent and 0.4 per cent for use in CRCP and Continuously Reinforced Concrete Roadbase (CRCR), respectively. For a 220mm thick slab, this is approximately 1300mm^2/m for a CRCP and 900mm^2/m for a CRCR. The longitudinal reinforcement bars in CRCP are usually 16mm diameter deformed steel.

In developing the CRCP thickness designs, Garnham (1989) used the lower value of 900mm^2/m for the CRCP thickness design equation, erring on the conservative side. For a jointed reinforced concrete (JRC) pavement, the relationship between the cumulative traffic loading and the slab thickness for a Class C32/40 concrete on a foundation with an equivalent surface foundation modulus (ESFM) of 270MPa and a reinforcement content of 900mm^2/m or 1300mm^2/m is shown in Figure 4.1.

This shows that, using the RR87 equation for JRC, there is approximately a 10 per cent reduction in slab thickness when the amount of reinforcement is increased from 900mm^2/m to 1300mm^2/m. It should be noted that the reinforcement content in the RR87 equation ranged between 312mm^2/m and 920mm^2/m. Therefore, extrapolation beyond the data set for design purposes may give less reliable results. However, in the current design curve, HD26/01 (DMRB 7.2.3), there has been no

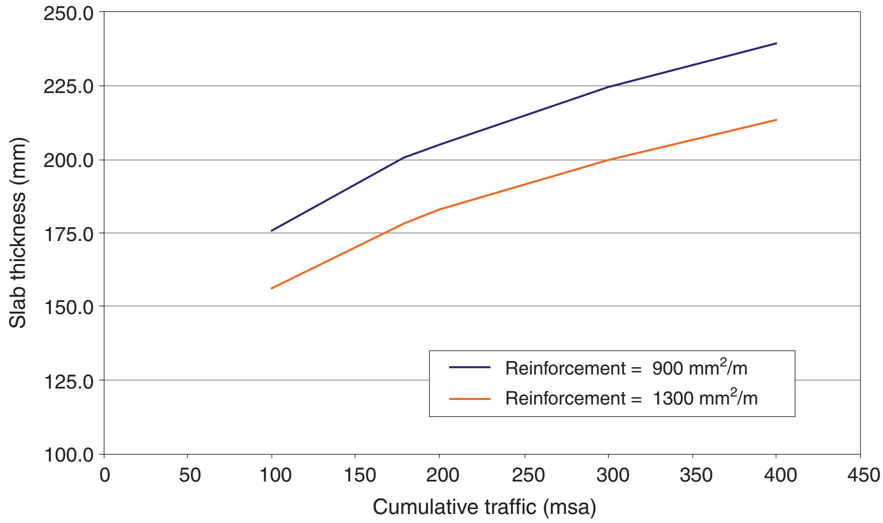

Figure 4.1 Relationship between amount of reinforcement and slab thickness for JRC

reduction in slab thickness to take into account the 0.6 per cent of steel reinforcement currently required for CRCP or for different concrete strength. In future designs, consideration should be given to adjust the amount of reinforcement depending upon the concrete strength.

Transverse reinforcement is specified in the UK as 12mm diameter deformed bars at 600mm spacing. The purpose of the reinforcement is to enable locating and fixing of the longitudinal reinforcement and to eliminate the formation of longitudinal cracks. It is also considered to contribute to the formation of the transverse cracks. However, transverse reinforcement has much less contribution to the structural performance of the CRCP than the longitudinal reinforcement. Where the transverse steel has been omitted the results from crack surveys have shown that a more random crack pattern has been induced.

4.2 Reinforcement corrosion

The cracks in a CRCP have the potential to allow the ingress of aggressive de-icing salt solution into the body of the slab. Experience from concrete bridges indicates the onset of reinforcement corrosion takes about 15 to 20 years (Vassie, 1987). This is dependent upon many variables such as the amount of de-icing salt used, the amount of traffic, the concrete quality, the cover thickness, the number of wetting and drying cycles and the maintenance history.

The amount of chloride ion contamination and the degree of corrosion of the steel were investigated in a variety of CRCP slabs and JRC pavements in the UK and the results are given in Table 4.1. Cores at cracked and uncracked locations were taken from the pavements and tested for chloride ion contents at different depths, in accordance with BS 1881: Part 124 (1988).

4.2.1 Uncracked concrete

Figure 4.2 shows the chloride concentration profile for the uncracked cores taken from Site 1, which is a JRC pavement. The concrete was 230mm thick laid on a polythene slip membrane layer. The subbase comprised 125mm of lean concrete on 100mm of Type 1 material. The reinforcement was laid to give a concrete cover of 50 to 65mm. The pavement was 36 years old, therefore the chloride concentrations can be regarded as typical of a pavement reaching the end of its forty year design life.

Table 4.1 Concrete sites investigated for chloride profile measurements

Site	*Pavement type*	*Concrete age (years)*	*Cracked/ uncracked cores*	*No. of cores*
Site 1	JRC	36	Both	6
Site 2	CRCP	7	Both	2
Site 3	CRCP	18	Uncracked	3
Site 4	CRCP	18	Both	31
Site 5	CRCP	12	Both	3
Site 6	CRCP	25	Both	6

The chloride concentrations generally decrease with increasing concrete depth until approximately mid-slab, and then increase to the bottom of the slab. The high chloride concentrations at the bottom of concrete could be attributed to the slip membrane layer preventing the passage of the de-icing salt solution to the underlying materials and allowing the salted water to pond. It is possible that large quantities of salt solution have passed through unsealed pavement joints and cracks to the bottom of the pavement quality concrete.

The chloride concentration at the depth of the reinforcement varied from 0.49 to 1.03 per cent, by weight of cement. However, no signs of corrosion were observed on the steel reinforcement.

For CRCP, Figure 4.3 shows the variation of the average chloride concentration with depth of uncracked cores taken from Sites 2, 5 and 6. The results show chloride concentrations of at least 2 per cent near the surface and a reduction in concentration with increasing depth. At the depth of the reinforcing steel, 100 to 120mm, the concentrations were 0.34, 0.05 and 0.21 per cent for Sites 2, 5 and 6, respectively. These values indicate a negligible probability of corrosion for Sites 5 and 6, and a low

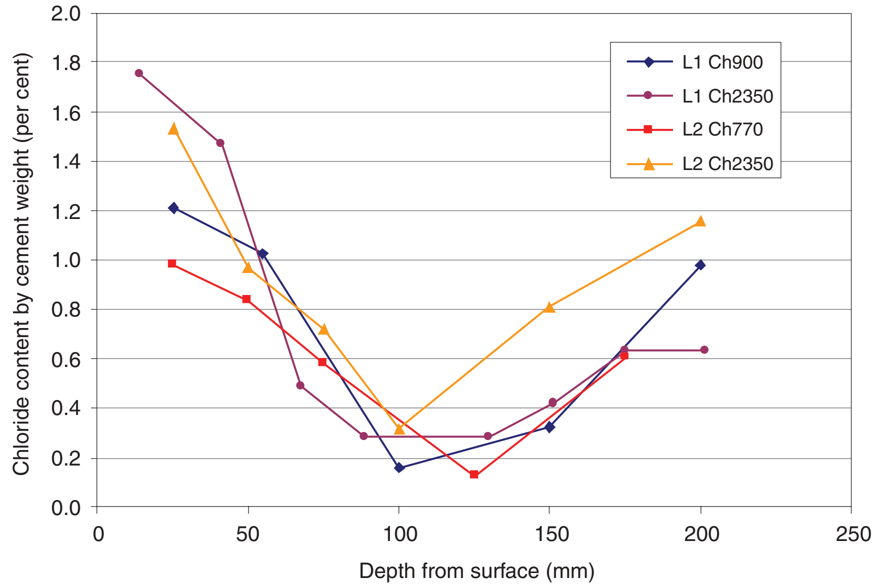

Figure 4.2 Chloride concentration profiles for uncracked JRC cores, Site 1

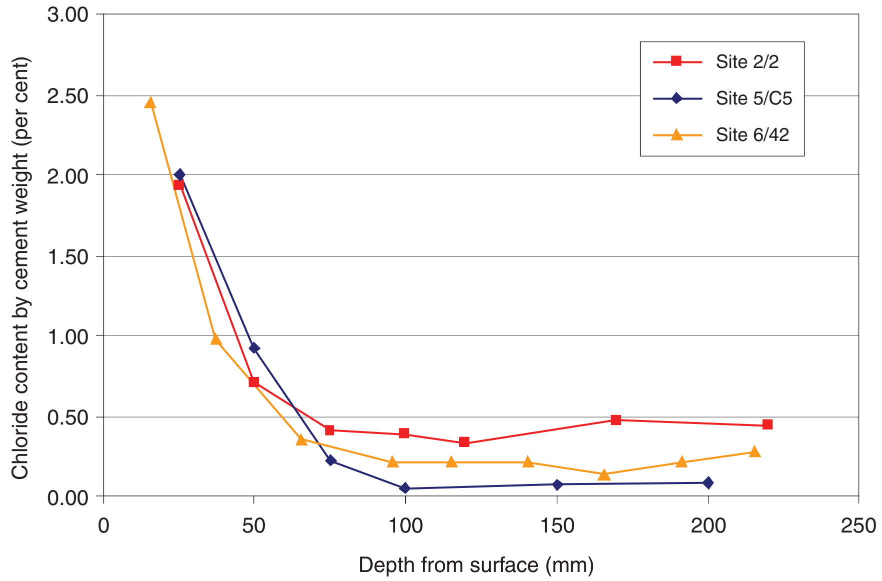

Figure 4.3 Chloride concentration profiles for uncracked CRCP cores

probability for Site 2. The concentrations did not increase towards the bottom of the slab as they did for the Site 1 cores, probably due to the elimination of the slip membrane layer under a CRCP.

Measurements of the chloride concentration for Site 3 were only carried out at the reinforcement level and the results showed values between 0.02 to 0.15 per cent by weight of cement. This range is in agreement with the results obtained from CRCP cores in other sites and confirms the low risk of corrosion to reinforcement in CRCP at a distance from the cracks.

4.2.2 Cracked concrete

The variations of chloride concentration with depth from the surface of concrete cores taken through cracks in CRCP are shown in Figure 4.4. The results show the general trend of lower chloride concentrations with increasing concrete depth from the surface, but less markedly than for the uncracked CRCP cores. At the reinforcement locations, 90 to 120mm, the chloride concentrations varied from 1.0 to 4.0 per cent, indicating a high probability of reinforcement corrosion. Chloride concentration measurements for Site 4 were only undertaken at the reinforcement level and the results indicated that in the vicinity of a vertical crack, the chloride concentration was between 1.11 and 1.67 per cent.

The variation of average crack width, measured from both sides of each core, with depth from the surface is given in Table 4.2. In general, the crack width decreases with increasing depth from the surface, although for the cores Site 2/1 and Site 6/17 the changes in crack width are small. For Site 5/D3 the crack width increases after the

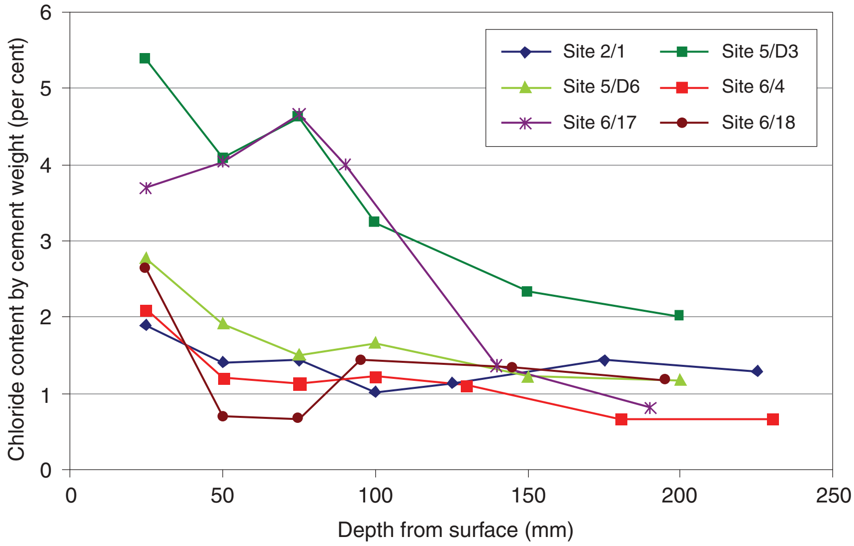

Figure 4.4 Chloride concentration profiles for cracked CRCP cores

Table 4.2 Average crack width at different depths from the surface of CRCP

	Average crack width (mm)					
Depth (mm)	*Site 2/1*	*Site 5/D3*	*Site 5/D6*	*Site 6/4*	*Site 6/17*	*Site 6/18*
0	0.25	0.20	0.45	0.80	0.20	0.60
50	0.20	0.10	0.30	0.55	0.15	0.35
100	0.15	0.15	0.15	0.35	0.15	0.25
150	0.10	0.20	0.15	0.30	0.15	0.25
200	0.10	0.60	0.15	0.20	0.10	0.15
225	–	0.65	0.10	–	–	–

crack passed the reinforcement so that the width at the bottom of the core was greater than at the surface.

The results from Table 4.2 and chloride concentrations in Table 4.3 suggest that there is no correlation between the crack width at the surface, at 0mm depth, and chloride concentration at the reinforcement locations. For example cores from Site 2/1, Site 5/D3 and Site 6/17 have narrow surface crack widths of 0.20 to 0.25mm. However, the chloride concentration at the reinforcement level was the lowest for Site 2/1, 1.1 per cent by the weight of cement, and very high for Site 5/D3 and Site 6/17, being 3.2 and 4.0 per cent, respectively. In contrast, the Site 6/4 core with the widest surface crack of 0.8mm also had the lowest chloride concentration of 1.1 per cent at the reinforcement level. This core also had the greatest cover over the reinforcment. Thus, while the cracked concrete has consistently higher chloride concentrations than the uncracked concrete, the width of the crack does not appear to have a consistent influence on the chloride concentration. This is unexpected and with current knowledge there is no plausible explanation. It is important to note that large quantities of chloride have entered the concrete even through narrow cracks.

4.2.3 Condition of the reinforcement

For the uncracked cores, the concrete cover depth ranged from 55 to 120mm and the chloride concentration was highest at the lowest depth of reinforcement. There was no

Table 4.3 Chloride concentration and reinforcement condition for cracked cores

Site/core	*Age (yr)*	*Concrete cover (mm)*	*Surface crack width (mm)*	*At the reinforcement: Chloride (per cent)*	*At the reinforcement: Crack width (mm)*	*Steel corrosion: Longitudinal*	*Steel corrosion: Transverse*
Site 2/1	7	125	0.25	1.1	0.10	None	–
Site 5/D3	12	100	0.20	3.2	0.10	Low	High
Site 5/D6		100	0.45	1.7	0.15	None	–
Site 6/4	25	130	0.80	1.1	0.35	Medium	–
Site 6/17		90	0.20	4.0	0.15	Low	High
Site 6/18		95	0.60	1.4	0.25	Low	–

evidence of significant corrosion in these uncracked cores where the chloride content at the reinforcement locations varied from 0.07 to 1.03 per cent by weight of cement.

For the cracked cores, Table 4.3 gives the concrete cover, the chloride concentration and the crack width at the reinforcement location, and the state of corrosion of the reinforcement. There was a trend for the chloride concentration to increase as the concrete cover decreased. For Site 2/1 at 7 years, the concrete cover was 125mm and the steel was not corroding at a relatively low chloride concentration of 1.1 per cent. However, this chloride concentration was sufficient to initiate corrosion in another core (Site 6/4) from an older pavement, with a wider crack width of 0.35mm. The Site 5 cores were 12 years old and the steel in one of the cores was corroding. However, the cracks were only classified as narrow, but the chloride concentrations were relatively high, 3.2 and 1.7 per cent. The Site 6 cores were 25 years old with crack widths ranging from 0.15 to 0.35mm and the chloride concentrations from 1.1 to 4.0 per cent; all three cores had steel that was corroding.

The highest chloride concentration for which there was no corrosion was 1.7 per cent of cement weight, Site 5/D6, whereas the lowest chloride content for which there was corrosion was 1.1 per cent, Site 6/4. There is no clear relationship between crack width and the occurrence of corrosion although the two cores where the steel was not corroding had relatively narrow crack widths at the depth of the reinforcement.

The steel reinforcement had corroded in four of the six cracked cores included in Table 4.3. The corrosion on the longitudinal steel was localised in the position of the transverse cracks. High corrosion levels were only observed in the transverse bars in cores Site 5/D3 and Site 6/17 and occurred over most of the bar length. The more extensive corrosion on the transverse bars is to be expected because they are usually coincident with the crack over significant distances, whereas the longitudinal steel is perpendicular to the transverse cracks. The transverse steel bars were situated below the longitudinal steel so the penetrating chloride ions would have reached the longitudinal steel before the transverse steel, however, the corrosion was much worse on the transverse steel as shown in Figure 4.5. The longitudinal bar (top) showed no significant loss in cross-section and the transverse bar (bottom) had lost the ribbing but there was no significant reduction in cross-section of the bar.

4.3 Corrosion risk and protection measures

The term 'risk' refers to the probability of reinforcement corrosion occurring and its consequences. The main cause of corrosion in CRCP is the penetration of the concrete cover by the chloride ions in the rock salt de-icing solution. This is normally a fairly slow process and it takes some time for the chloride ions to reach the reinforcement in sufficient quantities to initiate corrosion. It is difficult to determine the time to corrosion with much precision because of the uncertainty about the value of the threshold chloride concentration. However, what is plain from the results of the chloride analysis is that the time to initiate corrosion is definitely less than 40 years in the vicinity of cracked concrete.

Figure 4.5 Example of corrosion of longitudinal and transverse bars

When steel corrodes, iron atoms are converted to iron oxide molecules, which is the brown rust commonly seen on corroding steel. Thus, corrosion results in a reduction in the number of iron atoms and hence a reduction in the cross sectional area of the steel. The rust formed when a steel bar corrodes initially adheres to the bar and its volume is several times greater than the volume of the iron atoms from which it was formed. This generates an internal pressure in the concrete and can result in cracking; corrosion cracks. Figure 4.6 shows examples of corrosion cracking originating at the reinforcement and travelling either up towards the road surface (left photograph) or moving horizontally (right photograph), which could lead to an incipient delamination. In the cores examined in this report, there is no evidence of a corrosion crack reaching the running surface. This, and the fact that most corrosion occurs on the transverse reinforcement indicates that the consequences of reinforcement corrosion for the functioning of a CRCP are small. However, it should be noted that the consequences could be expected to increase if the service life of the CRCP was extended or the concrete cover to the reinforcement was reduced.

When corrosion occurred in CRCP, the corrosion level was much less and more localised on the longitudinal reinforcement than on the transverse bars, and coincided with the transverse cracks. As the longitudinal bars have greater influence on the structural performance of pavements than the transverse bars, good performance could still be achieved even with significant corrosion. On the basis of our assessment of the risk of corrosion it can be stated that:

- The probability of corrosion during the service life is high.
- The consequences of corrosion on the structural performance of CRCP are very low.
- Overall, the risk of corrosion affecting the service life of CRCP is low.

This initial study should be supplemented by further testing to provide a clearer understanding of the long-term corrosion of CRCP. Further examination of cores would give a clearer picture of the rate of chloride ingress, the time to corrosion, the chloride threshold concentration, and the predicted service life of CRCP.

Figure 4.6 Examples of corrosion cracking

Based on the above assessment, protective measures against reinforcement corrosion in CRCP are not justified in most circumstances, especially as the cost of protection is likely to be high. However, it is possible that corrosion protection measures would enable the life of CRCP to be extended, which would reduce whole life costs and improve sustainability. There are three main approaches to protecting steel reinforcement in concrete from corroding:

- Use a reinforcing material that is less vulnerable to corrosion than mild steel.
- Modify the concrete to make it more difficult for chloride ions to pass through.
- Use an overlay to seal the concrete surface.

Examples of the first approach are epoxy coated steel, galvanised steel, stainless steel or non-metallic reinforcement. Examples of the second are to increase the cover depth, produce less permeable concrete, use cast-in corrosion inhibitors such as calcium nitrite or apply a surface coating treatment. The third approach would involve using an impermeable asphalt overlay.

In Belgium, the longitudinal reinforcement bars are placed at third-depth on transverse bars of 12mm diameter at an angle of 60 degrees to prevent the transverse reinforcement coinciding with the transverse cracks in CRCP. These arrangements were reported to result in a network of fine cracks with almost no reinforcement corrosion, less than 5 per cent steel loss, after 10-20 years in service (FEBELCEM, 2003).

The results obtained in this investigation showed no clear relationship between crack width and chloride concentration, as de-icing solution can even penetrate through cracks of 0.2mm width causing significant corrosion. In the vicinity of cracks, increasing the concrete cover reduces the probability of corrosion. Skewing transverse reinforcement could be good practice to reduce the corrosion level of the transverse reinforcement, provided that transverse cracks do not follow the diagonal transverse reinforcement. However, results from crack surveys have indicate that the transverse reinforcement is beneficial in forming a regular transverse crack pattern in the slab, and that more random crack patterns have formed where the transverse steel has been omitted.

4.4 Fibre reinforced concrete

Steel reinforcement bars are commonly used in concrete to withstand the induced tensile stresses and protect the concrete from tensile failure. When multidirectional stresses are induced, the reinforcement detailing becomes more complicated and expensive. Therefore, the use of short, discontinuous fibres, uniformly mixed and dispersed throughout concrete, could be advantageous. Steel fibre reinforced concrete (SFRC) provides a means of arresting crack propagation by improving the post-crack properties of the concrete.

Steel fibres are produced in a variety of types and shapes, and can affect the properties of concrete based on their quantities, properties, and their bond with the concrete matrix. Steel fibres can be straight, but the majority are shaped in such a way as to improve their anchorage in the concrete (e.g. wavy, crimped end or enlarged end). The most useful parameters for describing fibres are:

- Aspect ratio (length/diameter ratio).
- Fibre tensile strength.
- Geometrical shape.

In general, increasing the aspect ratio improves the effectiveness of the fibres but impedes the consistence (workability) of the concrete. Fibres with a high aspect ratio give better post-crack toughness and residual strength. Their long length and efficient anchorage mechanisms make them ideally suited in applications where the anticipated mode of failure is flexure. The short fibres give a finer distribution of the reinforcement and may be more efficient at controlling the propagation of cracking. The third edition of TR34 (Concrete Society, 2003) shows that in ground floor slabs a variety of aspect ratios were used for steel fibres, ranging between 20 and 100, with the most commonly used length of fibre being 60mm. The Association of Concrete Industrial Flooring Contractors Introductory Guide (ACIFC, 1999) suggests that fibres should be 19 to 60mm in length, have an aspect ratio of 30 to 100, a tensile strength of 345 to 1700MPa, a modulus of elasticity of 205GPa and be able to bend through 180° without rupture. As a compromise between performance and dispersion, Maidl (1995) suggested an aspect ratio of 50 to 100, and indicated that the diameter of the fibre should be at least 0.5mm in order to avoid failure due to corrosion of the fibres spanning cracks.

Pavement mixtures with low water/cement ratios generally have low consistence and specific considerations should be made when using steel fibres. The high water content associated with the increased cement content used in SFRC could cause problems with curling and high shrinkage. Therefore, it is essential to use superplasticisers and to adjust the aggregate grading and maximum size. Attention should also be paid to the finished surface of a pavement, as a brushed, dragged or tined macrotexture could result in many exposed or loose fibres at the pavement surface.

Steel fibres have little effect on the compressive strength of concrete but more pronounced influence on the fatigue, impact resistance, shear strength, shrinkage cracking, thermal shock and flexural toughness (ACIFC, 1999). In pavements, SFRC can reduce the amount of longitudinal cracking or allow wider slabs to be constructed. Once cracks have formed, the fibres control the width of the crack, resulting in a better performance of the pavement. However, in the review undertaken there was no evidence of replacing the longitudinal or the transverse bar reinforcement in CRCP with steel fibres.

Economic benefits may be achieved with SFRC construction, compared to conventional bar reinforced concrete, by reducing the slab thickness or, by enhancing the service life with reduced maintenance requirements when the thickness is not reduced.

> The amount of reinforcement used is dependent on the concrete strength, and both parameters should be considered in the CRCP designs.
>
> The transverse reinforcement is beneficial in forming a regular transverse crack pattern in the slab, more random crack patterns have formed where the transverse steel has been omitted.
>
> The presence of a slip membrane increases the chloride concentration at the bottom of the slab.
>
> There was no evidence of corrosion at locations away from cracks or at chloride concentrations less than 1.1 per cent, by weight of cement.
>
> High chloride concentration and reinforcement corrosion occur in the vicinity of cracks, but no clear relationship was found between the chloride concentration and the surface crack width.
>
> The initiation of corrosion at cracks appears to be between 7 and 12 years in CRCP.
>
> At transverse cracks more severe corrosion occurs on the transverse reinforcement than the longitudinal reinforcement.
>
> There is no evidence of significant consequences of corrosion damage on the performance of CRCP in the UK.

5 Foundations

The main purpose of the foundation, is to distribute the applied traffic loads to the underlying subgrade without allowing distress in the foundation layers or in the overlying layers during the construction and the service life of the pavement. The current design method for foundations is given in HD25/95, (DMRB 7.2.2).

Rigid and rigid composite pavements have many benefits in respect of foundation materials and designs. The high stiffness of these pavements distributes the traffic load over a relatively large area of the subgrade. Therefore, the stresses on the foundation are reduced and minor variations in the subgrade strength have little influence upon the structural capacity of the pavement. In contrast, flexible pavements are inherently less stiff and do not spread loads as well as rigid pavements. CRCP has the potential advantages over jointed concrete of reducing the amount of water penetrating into the pavement, through poorly maintained joints, and the associated pumping of fine material under traffic loading, leading to enhanced foundation durability. Therefore, due to the relatively less stringent requirement for foundations under CRCP, a wide range of materials could be used including secondary and recycled materials.

5.1 Reclaimed materials

The use of secondary and recycled materials contributes to more economic and sustainable construction by reducing the amount of material sent to landfill and minimising the extraction of natural resources. Aggregate is consumed in large quantities in construction and there are some positive indications of the increased use of alternative materials. Statistical data on the annual production of primary aggregate indicate a significant reduction from over 300 million tonnes (Mt) in 1989 to 215Mt in 1996, followed by a fairly stable production rate of about 220Mt through to 2000 (British Geological Survey, 2001). Conversely, the amount of secondary aggregate used in construction has increased by approximately 40 per cent, from 32Mt in 1989 to 46Mt in 1999, indicating the acceptance of recycling and the use of alternative materials in the construction industry.

Road construction has a high demand for aggregates, and there is a wide range of secondary and recycled alternative materials available in large quantities in the UK, more than 150Mt/annum arisings and 1200Mt stockpiled (Hassan *et al.*, 2004). The use of alternative materials is dependent on their properties and availability to meet the demand. The site location relative to the source also greatly influences the decision of whether or not to use alternative aggregates. The initial costs of alternative materials are often lower than conventional materials, and could provide significant economic benefits if site-won or available locally, thereby reducing transport costs.

5.2 Subgrade and capping

Road Note 29 (DoE and RRL, 1970) considered different classes of subgrade based on the subgrade strength, determined by the California Bearing Ratio (CBR) value, which was used for the determination of pavement thickness. LR1132 (Powell *et al*., 1984) adopted a different approach based on the modulus of the subgrade and capping. The subgrade design modulus, in MPa, was established from the equilibrium in-service CBR, for values between 2 and 12 per cent, in the form:

$$\text{Modulus} = 17.6\ (\text{CBR})^{0.64} \tag{5.1}$$

The calculation of the stresses and strains within the capping layer is relatively difficult due to the expected non-linear behaviour of the material. The modulus for the capping layer was considered in LR1132 (Powell *et al*., 1984) to be in the range of 50 to 100MPa, and in developing the design equations in RR87 a value of 70MPa was used.

The current UK foundation designs for rigid and rigid composite pavements, HD25/94 (DMRB 7.2.2), require a capping layer when the CBR of the subgrade is less than 15 per cent. The capping layer is laid between the subbase and the subgrade to improve and protect weaker subgrades, with the ability to increase the modulus and strength of the formation before laying the subbase layer. A maximum capping thickness of 600mm is required for a weak subgrade of 2 per cent or less CBR, and the thickness is reduced as the subgrade strength increases. No capping layer is required for a subgrade CBR more than 15 per cent.

For weak soils, when cheap aggregate is not available locally, the use of imported materials for the capping layer will increase the costs of the pavement construction. A more economic construction could be obtained by *in situ* stabilisation of the existing soil, as specified in MCHW1 Clauses 614 and 615 (Highways Agency *et al*.).

There is a need to review the minimum CBR requirement of 15 per cent for the subbase only option, which is currently specified under rigid and rigid composite pavements. Practical experience in the UK has indicated that it is possible to lay and adequately compact a cemented subbase directly on a subgrade with a lower CBR value than 15 per cent. It has been reported by Griffiths (2003) that on the M6 Toll road a subgrade with a CBR of 3 per cent was strong enough to omit the capping layer and provide an adequate platform for compacting the CBM2A subbase. Also, on the A417/A419 Swindon to Gloucester road a satisfactory performance was obtained from a CBM1A with a minimum thickness of 250mm constructed on a subgrade with a minimum CBR of 3.5 per cent. Although this evidence suggests that a minimum value of 3 per cent CBR could be acceptable for the subbase only option, it is considered more prudent to use a higher minimum value of 5 per cent CBR.

5.3 Subbase

The subbase is a platform layer upon which the structural layers of pavements are constructed. Currently, HD25/94 (DMRB 7.2.2) specifies that only cemented subbases are currently permitted for use under rigid and rigid composite pavements in the UK.

5.3.1 Subbase requirements

Table 5.1 gives a summary of the cemented subbase requirements under concrete roads in different countries (PIARC, 1994). In general, there is no uniformity between the subbase requirements of strength, testing age and thickness for the eight European countries assessed.

The results in Table 5.1 clearly show that, even when taking into account the age of testing, the compressive strength requirement for cement bound subbases in the UK is significantly higher than that for the other countries. Therefore, a more economic construction could be achieved by using weaker but durable subbase materials under CRCP.

The thickness requirement of the cemented subbase is also shown to vary in Table 5.1. Most of the countries specify a subbase thickness of 150mm. Austria and Belgium also require an additional 50mm of asphalt regulating layer on the top of the cemented subbase. It has been reported that the use of an asphalt regulating layer improves the performance of concrete pavements by

Table 5.1 Cemented subbase requirements under concrete roads (PIARC, 1994)

Country	*Materials and thickness*	*Strength requirements*
Austria*	Granular or cement bound >200mm + 50mm asphalt regulating layer	7d compressive ≥3MPa
Belgium	Lean concrete: 180-200mm + 50mm asphalt regulating layer	90d compressive 7-10MPa
France	Related to design traffic, subgrade and pavement type. Vibrated lean concrete on Motorways 120-220mm	360d tensile ≥1.5MPa
Germany*	Cement bound 150mm Cement treated 150-200mm	28d compressive 9-12MPa 28d compressive 6MPa
Italy	Cement treated 150mm on granular material 150mm	7d compressive 4-7MPa
Spain	Lean concrete or cement treated 150mm	7d compressive 8MPa
Netherlands**	Lean concrete 150-200mm	7d compressive ≥3MPa
UK	Cement bound 150mm	7d compressive ≥10MPa

** Countries where unreinforced jointed concrete only is used.*

*** Country where the concrete pavement type is not given.*

overcoming cemented subbase problems associated with dimensional changes and susceptibility to frost erosion (Kraneis, 1990). The asphalt elasticity provides a cushion for the deformations of the concrete slab with improved foundation contact. Asphalt also acts as a sealing layer to prevent the absorption of mixing water from the fresh concrete and the penetration of surface water through cracks to the foundation.

5.3.2 Cracking of cemented subbases

Cemented subbases offer the advantages of high stiffness and less erosion, but are susceptible to dimensional changes due to shrinkage and temperature variations. Restraint to such movements contributes to natural cracking in the subbase that could influence the performance of the CRCP. Most and Vring (1990) indicated the benefit of stabilised dense subbases in minimising the risk of erosion but a disadvantage in increasing the risk of reflection cracking. They gave recommendations to induce ‘transverse cracks’ by pre-cracking the subbase layer to overcome this problem.

The crack pattern of cemented layers was investigated through a review of TRL data from various trials. Figure 5.1 shows the general relationship between the cemented 7-day strength and average crack spacing. This indicates that the higher the strength of the layer the larger the crack spacing becomes. There was also a tendency for wider cracks at larger crack spacings. Other factors influencing the cracking pattern of cemented subbases are the coefficient of thermal expansion of the constituent materials, mainly the aggregate, and the climatic conditions at the time of construction.

A primary function of the cemented subbase is to provide a durable uniform support under CRCP. Whilst the foundation stiffness has only a little influence upon the structural capacity of the pavement, variation in the uniformity of foundation support could influence the performance of the CRCP. Wide cracks in the subbase result in a loss of aggregate interlock and are considered the main cause for the discontinuity of foundation support.

Wide cracks in cement bound materials are likely to occur where there are large crack spacings. Problems associated with high strength cemented subbases, in terms of natural cracks and discontinuity of foundation support, are addressed in several countries by pre-cracking the subbase layer to induce closely spaced cracks. Clause 1035 of the MCHW1 (Highways Agency *et al.*), shows that inducing transverse cracks during construction is now required in the UK for cement bound materials with an average minimum 7-day cube strength of 10MPa or greater.

5.3.3 Subbase friction

A review of literature showed that the amount of the frictional force, expressed in terms of the amount of bond, shear and bearing, between the bottom of a CRCP slab and the top of the underlying subbase layer is an important parameter because it affects the widths and spacing of the induced transverse cracks in the CRCP slab. Generally, subbases with large frictional forces, such as cement bound and unbound materials, induce closer spacing of transverse cracks than subbases with low frictional forces such as asphalt bound materials (McCullough and Moody, 1993). However, this difference between materials is reduced when limestone coarse aggregate rather than siliceous gravel is used in the CRCP concrete, indicating that the performance is more influenced by the aggregate type in the CRCP concrete than the frictional force with the subbase. It was also shown that the average crack spacing for CRCP with limestone aggregate was approximately twice the value for CRCP with siliceous gravel aggregate (Wimsatt *et al.,* 1987), in agreement with the results presented in Table 2.1.

5.4 New foundation classes

Recent research at TRL given in TRL615 (Nunn, 2004), has reviewed the restriction of pavement design standards to conventional materials and developed a more versatile approach to pavement design to support the wider and more efficient use of alternative materials. The versatile design approach considers four foundation stiffness classes, defined in terms of the equivalent half-space stiffness of the composite foundations. The composite foundation modulus used for design is 50, 100, 200 and 400MPa for Foundation Classes 1, 2, 3 and 4, respectively. A Poisson’s ratio of 0.35 is proposed for all foundation classes.

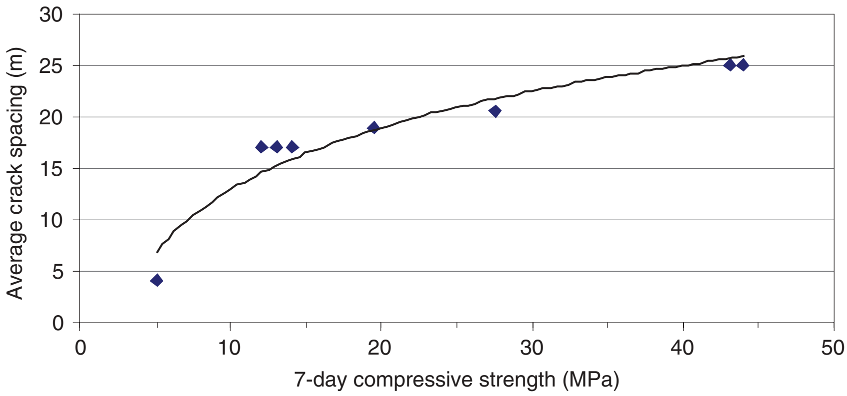

Figure 5.1 Relationship between strength and average crack spacing

The four proposed foundation classes have been developed for flexible and flexible composite pavements and are specified as follows:

- *Foundation Class 1:* is a capping only design that is permissible for the construction of the base of the pavement provided the capping material has adequate shear strength. The application of this Class for high traffic roads may need to be limited.
- *Foundation Class 2:* for all traffic categories, is a subbase only or subbase on capping design that is considered as equivalent to the current standard unbound granular foundations.
- *Foundation Classes 3 and 4:* are designs incorporating hydraulically bound materials that provide a range of foundations of superior quality to current standard unbound granular foundations. These classes could permit thinner overlying pavements than those in the DMRB 7.2.3.

It is important to highlight that only Foundation Classes 2, 3 and 4 with bound subbases have been considered in this report for use under CRCP.

The new foundation classes have been developed mainly for flexible and flexible composite pavements with a conservative approach to foundation deterioration from traffic loading and environmental conditions. When used under CRCP, it is necessary to consider the high load spreading ability and protection provided to foundations. On comparing the foundation classes to the current foundation designs for CRCP, the following observations were made:

- The Poisson's ratio assumed for the new foundation classes is 0.35, compared to 0.45 used in RR87.
- The modulus values for the new foundation classes are developed for design purposes based on the long life foundation stiffness, and values have not yet been established for early age construction. In RR87, a value of 270MPa is considered at an early age.
- The concept of subbase only, without capping, has been considered in the new foundation classes for subgrades with a CBR less than 15 per cent.
- Cemented subbases of CBM3 and wet lean concrete C12/15, currently specified for use under rigid and rigid composite construction, are not assigned to any of the new foundation classes.
- The amount of deterioration from traffic induced stresses in a foundation under a well constructed CRCP is likely to be much less than that under a flexible or a flexible composite pavement.

5.5 Equivalent surface foundation modulus

The foundation design parameter used in RR87 was expressed as an equivalent surface foundation modulus (ESFM). This was defined as the modulus of a uniform elastic foundation that would give the same deflection, d_0, under the same wheel load, as that of the actual road structure. The calculation of the ESFM was made using a simplified method devised by Ullidtz and Peattie (1980) that transforms a multilayered elastic structure into an equivalent semi-infinite space, by assuming a single value of Poisson's ratio for all the layers:

$$ESFM = \frac{2\left(1-\nu^2\right)\sigma_0 a}{d_0} \tag{5.1}$$

Where *ESFM* is the equivalent surface foundation modulus (MPa)

σ_0 is the applied stress (MPa)

ν is a common Poisson ratio for all the layers

a is the radius of loaded area (mm)

d_0 is the total deflection of the surface of a structure (mm)

Values for ESFM given in RR87 used a standard wheel load with a contact pressure of 0.558MPa over a contact area of 151mm^2 and a Poisson's ratio of 0.45 for all foundation layers. Typical ESFM values in RR87 ranged from 50 to 1700MPa. The lowest values between 50 and 100MPa were associated with unbound layers. For cement bound subbases, the ESFM varied between 250 and 700MPa when a single layer was used, and between 700 and 1300MPa for two layers. The use of pavement quality concrete as a subbase, which has not been fractured by techniques such as 'crack and seat' or 'saw-cut, crack and seat', provided the highest ESFM value of 1700MPa. A value of 270MPa for the ESFM was deemed the most appropriate to use in the designs of concrete pavements derived in RR87, and was implemented in the current designs for CRCP, DMRB 7.2.2.

5.6 CRCP designs for different foundations

The relationships between the required CRCP thickness with a tied shoulder or a 1m edge strip and the cumulative traffic loading for the new foundation classes represented by the ESFM values of 100, 200 and 400MP, and of the 270MPa used in the current thickness designs, are shown in Figure 5.2. The figure shows the minimum thickness of 200mm currently specified for CRCP and assumes a mean concrete flexural strength of 5.0MPa and a reinforcement cross sectional area of 900mm^2/m.

For a cumulative traffic loading of 400msa, the slab thickness, rounded up to the nearest 10mm, for an ESFM of 100, 200 or 400MPa, representing Foundations Classes 2, 3 and 4, is 260, 250 or 240mm, respectively. Clearly, the reduction in slab thickness for an increase in ESFM is small. Doubling the ESFM will only have a small effect, 10mm, on the thickness design of CRCP.

When comparing the current and new designs at 400msa traffic loading, the difference in slab thickness between the current foundation with an ESFM of 270MPa and the new foundation classes is approximately 20mm more for Foundation Class 2 and 10mm less for a Foundation Class 4.

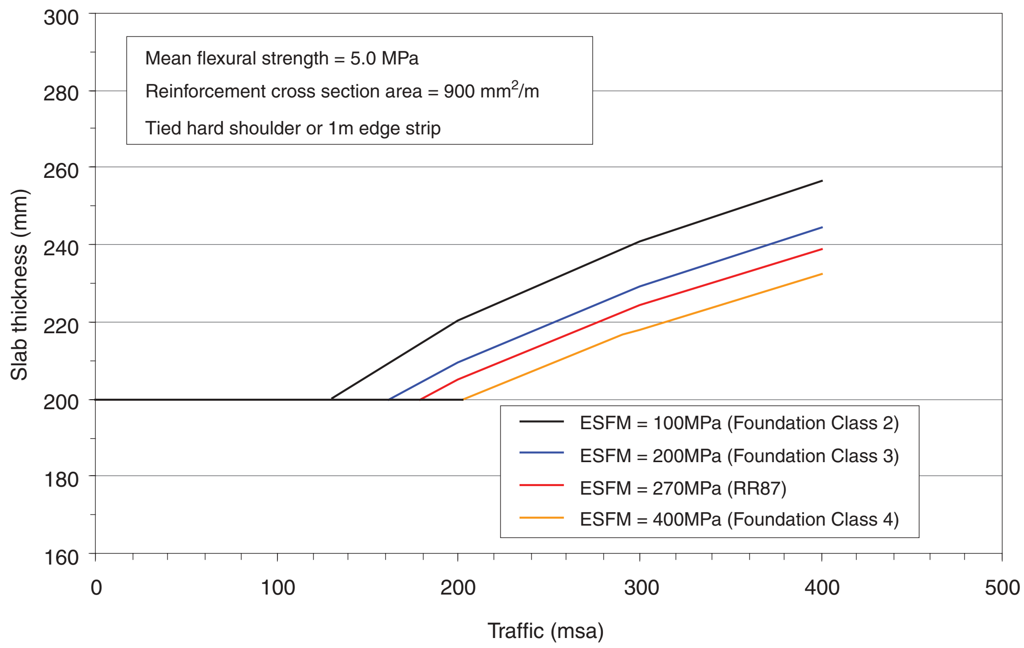

Figure 5.2 CRCP designs for different ESFM values

The subbase requirement for CRCP in the UK is significantly higher than that for other countries. A more economic construction could be achieved by using weaker but durable subbase materials.

The higher the strength of the cemented subbase the larger the crack spacing within it, giving a tendency for wider cracks and resulting in a discontinuity of foundation support.

The current UK specification requires transverse cracks to be induced (pre-cracking) for cement bound materials with an average minimum 7-day cube strength of 10MPa or greater.

There is potential to lower the minimum strength requirement of the subgrade from 15 to 5 per cent CBR for the subbase only option.

The new foundation design given in TRL615 incorporates a wider range of bound subbases with and without capping than currently specified.

Foundation Class 4 will reduce the current thickness design of the CRCP.

6 Shoulders and edge strips

Hard shoulders and edge strips to a CRCP provide many benefits including the option to construct thinner pavements and the provision of a safety zone at the side of the road. Properly designed and constructed hard shoulders have the capacity to act as an emergency traffic route, an additional traffic lane during contra-flow traffic management schemes and a traffic lane in road widening schemes. The types of construction of the edges adjacent to a CRCP are:

- A monolithic edge strip created by widening the pavement slab beyond the nearside and off side traffic lane edges.
- A concrete shoulder, tied into the main pavement slab.
- An asphalt shoulder.

Edge strips created by widening the CRCP slab can provide similar edge support as having a tied shoulder. Field studies by Colley *et al*. (1978) have reported that nearside widening of 0.4m to be as structurally effective as a hard shoulder. Sehr, (1989) considered that paving at least 0.45m wider than the lane width should keep heavy vehicle tyres away from the slab edge. This will greatly reduce the induced slab stresses in concrete pavements and having no longitudinal joint is likely to reduce the chances of punchouts.

Sawan and Darter (1978) reported that to obtain the optimum benefit from tied-shoulders they should be at least 1m wide. The results of an analytical study by Tayabji *et al.* (1984) indicated that the damaging effect of a single-axle load applied on a pavement with a tied shoulder is approximately half of that for the same axle load applied on a pavement without a tied shoulder. For application to the AASHTO thickness design procedure (AASHTO, 1998), concrete pavement depths between 193mm and 274mm without a tied shoulder can be reduced by 25mm for concrete roads with a tied shoulder.

Ceran and Newman (1992) recommended the use of full-width shoulders with the same thickness of concrete as for the main slab in urban areas. It was also recommended that transverse joints in a tied shoulder should match the transverse joints in the adjacent traffic lane to prevent induced transverse cracking from the joint. On this premise, the use of a CRCP shoulder against a jointed pavement should not be considered because of potential problems for induced shoulder cracking from the transverse joints in the main slab.

Keller (1988) indicated that asphalt shoulders are cheaper to construct than concrete shoulders, but they offer little structural support to the main pavement and tend to deteriorate within 150mm to 300mm of the edge of the concrete. As a consequence, the concrete suffers more distress, requiring more maintenance.

The current thickness design curve for CRCP given in the HD26/01 (DMRB 7.2.3) assumes the presence of a 1m edge strip or a tied hard shoulder adjacent to the most heavily trafficked lane. In the UK, the performance of CRCP with a 1m wide edge strip is considered to be satisfactory. The current design specifies that where there is no hard shoulder or 1m edge strip, the thickness of the CRCP for all the trafficked lanes is increased by between 20mm and 35mm, depending on the cumulative traffic loading. This could be considered uneconomic as only the near side traffic lane, Lane 1, carries the majority of the heavy goods vehicles. However, it may have long-term advantages in accommodating any increase in the maximum axle loading implemented in the future. Figure 6.1 shows the proposed UK design for a CRCP with a mean flexural strength of 4.5MPa, and the new Foundation Class 4. It can be seen that when a tied shoulder is used the slab may be reduced by between 24mm and 30mm for traffic loadings of 32msa and 400msa, respectively.

The design of the CRCP shoulder, including the foundation, should match that of the main CRCP slab.

It is recommended that jointed and asphalt shoulders should not be constructed alongside a CRCP.

The use of a CRCP shoulder against a jointed pavement is not recommended.

Overseas experience has indicated the potential of reducing the edge strip width from 1m to 0.4m.

7 Traffic loading

The current thickness design curve for CRCP given in the HD26/01 (DMRB 7.2.3) assumes that the pavement is designed to carry traffic for 40 years. During this period the maximum cumulative traffic is predicted as being up to 400 million standard axles (msa), where a standard axle is defined as an axle exerting or applying a force of 80kN, equivalent to an 8.16 tonne axle load.

A review of the amount of traffic currently carried by some of the most heavily trafficked Trunk Roads and Motorways in England has been carried out. Six sites were selected, which are not necessarily concrete pavements, and their traffic data are given in Table 7.1.

Table 7.1 Estimated cumulative traffic

Site	*AADF*	*Traffic count/carriageway* *Number of cv/d*	*Percentage of HGVs*	*Cumulative traffic loading over 40 years (msa)*
M5: Junctions 26 to 25	23,932	3,218	13	113
M6: Junction 12 to 13	49,724	9,070	18	262
M25: Junctions 12 to 11	55,747	8,871	16	248
M60: Junctions 25 to 26	47,178	3,858	8	117
A1: A168/Moor Lane	36,614	8,634	24	264
A12: A138 to B1389	39,143	4,422	11	83
Average	42,056	6,345	15	181

The annual average daily flow (AADF) for each site was taken from data collected in 2002 by the Department for Transport. The number of heavy goods vehicles (HGVs), expressed as commercial vehicles per day (cv/d) ranged from 3,218 to 9,070, with an average value of 6,345cv/d. The average percentage of HGVs was 15 per cent and ranged between 8 per cent on the M60 and 24 per cent on the A1.

The cumulative traffic loading, over a forty year period from the year of the traffic count, 2002, was determined using relationships for traffic growth and percentages of HGV in Lane 1 proposed for future use in the DMRB 7.2.1 and the factors for wear given in the current DMRB 7.2.1

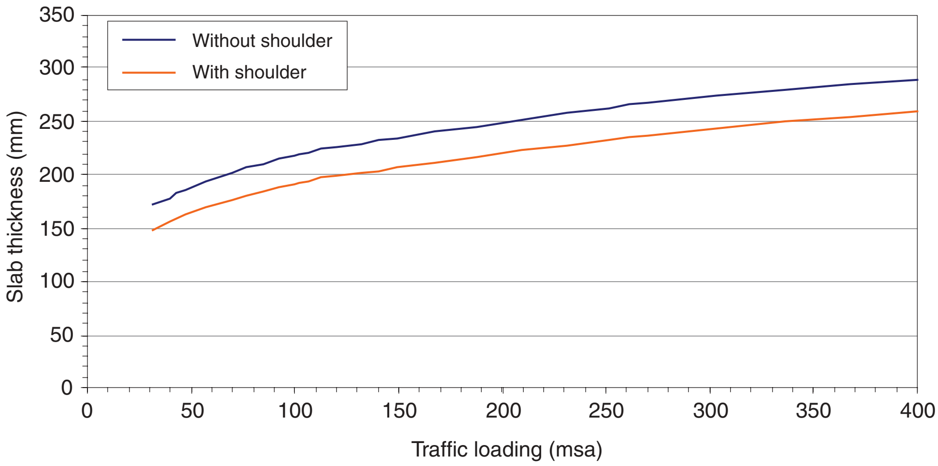

Figure 6.1 Relationship between traffic loading and pavement thickness

(Highways Agency *et al.*). Table 7.1 shows that the forecasted cumulative traffic loading varied between 83msa for the A12 and 264msa for the A1, which are considerably less than the maximum 400msa allowed for in the thickness design curves in the DMRB 7.2.3. As these sites represent the highest traffic flows in the UK, a maximum value of 300msa may be more realistic for design purposes. However, there is a case for keeping the maximum cumulative traffic for design at 400msa to take into account the increase in pavement wear resulting from envisaged increases in the maximum HGV axle load. The current designs in the HD24/96 (DMRB 7.2.1, Highways Agency *et al.*) are based on a maximum axle load of 10.5tonnes but this may increase to 11.5tonnes, which is currently the maximum permissible within the European Community. Assuming a 4th power load equivalence, a 11.5tonne axle is approximately 44 per cent more damaging than a 10.5tonne axle.

A survey of standards and practices for designing and constructing concrete roads in various countries has been published by PIARC (1994). The parameters relating to the maximum axle load, standard design axle load, the design period and the design traffic loading have been extracted from the Synoptic Table and are given in Table 7.2. The majority of the European countries design concrete roads for a twenty year life. France designs for a thirty year life whilst a forty year life is used in Germany and the UK. The maximum axle load ranges between 10.5tonnes and 13.0tonnes. The range of the standard axles used in design is from 80kN to 130kN, with the highest value being used in France and Spain.

Table 7.2 National traffic loading specifications (PIARC, 1994)

Country	*Maximum axle load (tonnes)*	*Standard axle (kN)*	*Design period (years)*	*Design traffic (cv/d)*
Austria	11.5	100	20	2,055
Belgium	13.0	80	20	4,500
France	13.0	130	30	2,000
Germany	11.5	100	40	3,200
Italy	10.5	80	20	6,164
Netherlands	10.5	100	20	2,000
Norway	10.5	100	20	1,370
Spain	10.5	130	20	2,000
Sweden	11.5	100	20	2,603
Switzerland	12.0	80	20	1,918
UK	10.5	80	40	No data[1]

[1] *Average 6,345 cv/d indicated in Table 7.1 from 2002 data.*

A comparison of the maximum equivalent standard axles that a country specifies for concrete roads is not easy since the cumulative traffic loading is based on different design lives, different standard axles and different ways of expressing design traffic in terms of cv/d.

A simplified approach has been made by comparing only the design traffic flow in cv/d. However, this takes no account of the period in the life of a road to which that figure refers. As no value for the UK is given in the Synoptic Table, the average value of 6,345 cv/d from Table 7.1 has been taken for comparison. Using this figure for the UK, Table 7.2 shows that the UK value is higher than in the other countries which ranged from 1,370cv/d in Norway to 6,164cv/d in Italy, but the UK figure is based on 2002 data compared to the 1994 data in Table 7.2.

A mechanistic-empirical concrete pavement design was developed in New York State based on performance and construction data for a selection of pavements (Bendaña *et al.,* 1994). When compared to the AASHTO design, the New York design predicts a greater number of axle loads to failure than the AASHTO design for slabs thinner than 250mm. New York State performance data from a sample of 35 concrete pavements have shown that these roads had lasted longer than would have been predicted by the AASHTO design. The traffic design parameters implemented for New York State rigid pavements were for a 50 year design life and a maximum traffic loading of 500 million equivalent 80kN single axle loads to failure.

The maximum estimated cumulative traffic loading of the most heavily trafficked roads in the UK was in the range of 80msa to 260msa for a forty year period.

There is a case for keeping the maximum traffic loading as 400msa to take into account the potential damaging effect from increased HGV axle loads.

The highest maximum traffic flow found in Europe is 6,164cv/d in Italy from 1994 data.

New York State designs are for a maximum traffic loading of 500msa over 50 years.

8 Terminations

Longitudinal movement of CRCP takes place at the end of the slab as the central part is more restrained and induced stresses are relieved by the transverse cracks. The amount of movement at CRCP ends can be significant, and if not accounted for, could cause damage to adjacent pavements or structures. Two systems, ground beam anchorage (GBA) and wide-flange steel beam (WFB), are commonly used. In both cases expansion joints between transition bays are used to accommodate any residual or unforeseen movements of the slab end. Four transition bays are currently specified in the MCHW Volume 3 (MCHW3) (Highways Agency *et al.*), constructed as 5m long jointed reinforced concrete with a separation membrane between the bottom of the concrete and the underlying material.

The time of CRCP construction also influences the movements at terminations. For CRCP constructed in winter, an initial seasonal summer expansion at the end of the slab will occur. In contrast, those constructed in summer will exhibit an initial contraction in the first winter.

For satisfactory pavement performance the joints accommodating these movements must remained sealed. Sealants are classified according to their ability to perform satisfactorily with the amount of thermal joint movement that a sealant is required to accommodate. BS 6213 (2000) defines this movement as the movement accommodation factor (MAF). Many manufacturers of joint sealants for concrete pavements recommend a MAF class 25, that is, the maximum thermal movement of a joint should not be more than 25 per cent of the minimum width of the joint groove. Specifications for expansion joints in concrete pavements given in the MCHW1 require that the minimum groove width is 30mm for hot and cold applied sealants. Therefore, in order to comply with the recommendation of the joint sealant manufacturers, the movement of an expansion joint should not exceed 7.5mm.

The thermal movements of CRCP slab ends and across expansion joints between transition bays were measured twice a year, once in the winter when the ambient temperature was low and in summer when the ambient temperature was high. The maximum and minimum temperature values of the CRCP concrete recorded during the period of monitoring the sites are given in Table 8.1 and ranged between 26.3 and 34.0°C for Sites E and A, respectively. Based on these results, a temperature range of 30°C is used in the report to express the seasonal thermal movements in the slab and across joints. The seasonal movement per degree C across the joints is the difference in opening between two consecutive seasons divided by the temperature difference. This has been expressed over the life of the pavement as the average seasonal movement coefficient for a 30°C temperature range.

Table 8.1 Maximum and minimum monitored slab temperatures

	Seasonal CRCP temperature		
Site	*Maximum (°C)*	*Minimum (°C)*	*Range (°C)*
Site A	30.8	-3.2	34.0
Site C	32.1	0.6	31.5
Site D	32.4	2.2	30.2
Site E	25.2	-1.1	26.3
Site F	31.8	-1.8	33.6

8.1 Ground beam anchorage

This system consists of a series of vertical ground anchors in the form of transverse concrete beams cast into the subbase and connected to the CRCP slab with steel reinforcement. The ground anchors are designed with a series of one or more transition bays, separated by expansion joints. The advantage of this system is that it requires no maintenance. However, its capability to restrict end movement is mainly dependent on the ability of the foundation, into which the ground anchors are installed, to resist the longitudinal movements at the end of the CRCP slab. This system is considered less effective on weak foundations, such as imported materials on high embankments. The current design for the ground anchorage system in the UK uses four ground anchors, spaced 6m apart, and four transition bays.

Site inspections have indicated that the performance of the GBA terminations is satisfactory in terms of requiring no maintenance. Generally, the crack patterns in the CRCP over the ground anchor beams were all similar with the formation of narrow, transverse cracks along the taper to the anchor beams and longitudinal cracks between the beams. The Netherlands (CROW, 1997) uses two layers of reinforcement in the CRCP slab at terminations, which may be beneficial in reducing cracks at the terminations.

The average seasonal movement coefficients for a 30°C temperature range are given in Table 8.2. This shows the average seasonal movement coefficient across any expansion joint varied between 0.63 and 8.07mm/30°C. The largest movement coefficients generally occurred at the expansion joint between the CRCP end and the first transition bay, Bay 1. An exception was termination B13 on Site D, which gave a higher movement value between Bay 1 and Bay 2.

The effect of varying the number of transition bays on the CRCP end movements can be seen from Site A. Out of the four sections constructed with only one transition bay, three Sections, T10, T11 and T12, had average seasonal movement coefficients exceeding the 7.5mm working range of the joint sealants. Where four transition bays were constructed, Sections T5 to T8, the maximum average value was 5.83mm. Therefore, it would be prudent to continue using the four transition bays currently specified in the MCHW3, especially when adjacent to concrete structures.

Transition bays are constructed on a slip membrane, which provides little resistance to thermal movement. On Sites C and D, at least one transition bay was longer than the specified 5m and the seasonal movements of the expansion joints were still within the working range of the sealant.

On Site B, one end of the CRCP slab had four beams, Section GN, whilst the other end, Section GS, had six beams. The average seasonal movement coefficient at the CRCP end with four beams was 3.38mm/30°C and more than double, 7.49mm/30°C, for the end with six beams. In this case, installing an additional two ground beam anchors to the four specified has not reduced the amount of seasonal CRCP end movement.

8.2 Wide-flange steel beam

In this system, the bottom flange of the universal beam is fixed into a concrete block cast in the subbase, as shown in Figure 8.1. A compressible material is placed between the vertical web of the beam and the CRCP to allow the slab end to move freely within the flange. As with the GBA system, a series of transition bays, separated by expansion joints, is placed immediately after the steel beam. The current UK design for the WFB termination is given in the MCHW3 and the design includes new details to increase the bonded area of the joint sealant at the top flange of the steel beam to overcome the problem which has been noted on a number of sites where the joint sealant had debonded from the WFB next to the CRCP ends.

Results of the average seasonal thermal movement coefficients (for a 30°C temperature range) at five CRCP sites with WFB terminations are given in Table 8.3.

In general, the slab end movements showed different patterns for the various CRCP sections. The results show a wide variation in the average seasonal movement coefficient values measured between the middle of the steel beam and the end of the CRCP slab. These values ranged between 0.16mm/30°C in Section L4N on Site E and 13.18mm/30°C in Section T4 on Site F.

Table 8.2 Average seasonal movement coefficients for the GBA terminations

			CRCP	*Transition bay*			
			CRCP end	*Bay 1*	*Bay 2*	*Bay 3*	*Bay 4*
Site	*Section reference*	*Period**	*Joint 1 (mm/30°C)*	*Joint 2 (mm/30°C)*	*Joint 3 (mm/30°C)*	*Joint 4 (mm/30°C)*	
Site A	T5	W93/94	5.23	1.52	1.01	2.05	
	T6	to S03	5.22	1.10	0.63	1.84	
	T7		5.83	2.88	1.01	1.13	
	T8		5.07	1.35	3.65	2.46	
	T9		5.96	–	–	–	
	T10		8.07	–	–	–	
	T11		8.06	–	–	–	
	T12		7.88	–	–	–	
Site B**	GN (4 beams)	S99 to	3.38	–	–	–	
	GS (6 beams)	W02/03	7.49	–	–	–	
Site C	S2	S87 to	4.24	3.79	1.61	–	
	S3	W99/00	2.94	2.18	3.09	–	
	S6		4.59	2.54	2.80	–	
	S7		5.09	4.69	2.91	–	
Site D	B8	S86 to	6.40	5.08	–	–	
	B10	S99	4.88	4.59	–	–	
	B12		6.00	4.88	–	–	
	B13		2.40	5.42	3.06	–	
	B14		6.64	4.70	1.82	–	

** S = summer   W = winter*
*** Site B has no transition bays.*

Figure 8.1 The wide-flange beam system

On Site E, Sections L4S and L4N, the average seasonal movement coefficients between the steel beam and the CRCP end were lower than the values between the steel beam and the adjacent transition bay. The anomaly in these results indicates that there may be a problem with the construction and/or performance of the WFB termination.

With the exception of Site G, which had no transition bays, Table 8.3 shows that the average seasonal movement coefficient across any expansion joint (joints 2 to 4) varied between 0.05 and 6.69mm/30°C. The range of values shows that no transition bays had an average seasonal movement coefficient exceeding the value of 7.5mm. In this type of termination, the CRCP end movement is generally taken up by the joint between the CRCP end and the WFB. On this basis, there is potential to reduce the number of transition bays from the four currently specified in the MCHW3. Some transition bays were constructed as trials and did not comply with the number and lengths given in the MCHW3. On Site F only two transition bays were installed instead of the four specified. On Sites C, D and F, some bays exceeded the recommended length of 5m. However, the results given in Table 8.2 indicate that these changes do not appear to have detracted from the performance of the expansion joints. There is a need for a concrete transition bay adjacent to the wide-flange beam to provide an additional support to the top flange, and it is recommended that as a minimum, two transition bays are used for a CRCP termination adjacent to a flexible construction. However, to reduce the damage risk where a termination is adjacent to a concrete structure, for example a bridge, it would be prudent to continue using four 5m transition bays currently specified in the MCHW3.

The seasonal end movements across the WFB joints on Site F are shown in Figure 8.2. Twelve years after construction, in Summer 1998, the ends of the slab, Section T1 and T4, have progressively extended by about 15mm. This CRCP site is an overlay to a flexible pavement, which provides a low subbase friction to the thermal movements. The seasonal end movements of the transition bay adjacent

Table 8.3 Average seasonal movement coefficients for the WFB terminations

			CRCP	*Transition bay*			
				WFB	*Bay 1*	*Bay 2*	*Bay 3* / *Bay 4*
Site	*Section reference*	*Period**	*CRCP-WFB (mm/30°C)*	*WFB-Bay 1 (mm/30°C)*	*Joint 2 (mm/30°C)*	*Joint 3 (mm/30°C)*	*Joint 4 (mm/30°C)*
Site C	S1	S87 to	8.84	0.10	2.31	2.11	1.91
	S4	W99/00	8.15	0.33	3.53	2.21	2.27
	S5		9.34	0.90	1.81	2.69	2.25
	S8		8.22	0.08	2.02	2.20	1.86
Site D	B9	S86 to	7.74	1.44	3.57	4.42	3.89
	B11	S99	7.27	1.30	5.12	3.86	4.44
Site E	L1 S	S98 to	0.92	0.71	6.29	0.96	2.00
	L1 N	S03	6.16	0.68	3.73	1.45	1.93
	L2 S		8.98	0.45	2.44	1.33	0.70
	L2 N		6.16	0.68	1.33	1.12	3.05
	L3 S	W99/00	2.64	1.55	6.47	0.97	0.90
	L4 S	to S03	1.81	3.84	6.69	1.61	0.98
	L4 N		0.16	7.82	0.05	1.49	1.17
Site F	T1	S86 to	11.02	0.17	5.48	–	–
	T4	S03	13.18	0.27	5.24	–	–
Site G	C1	S01 to	10.01	3.82	–	–	–
	C2	S03	9.27	1.45	–	–	–

** S = summer; W = winter.*

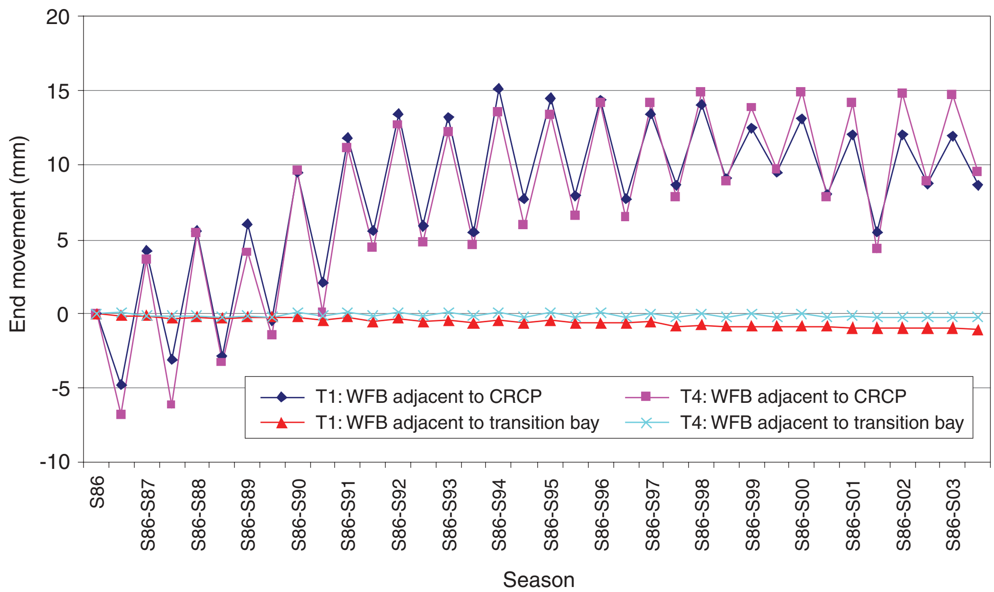

Figure 8.2 Site F seasonal end movements relative to the WFB

to the WFB are very small, approximately 0.5mm. The results indicate that the slab ends have progressively extended and may well have reached the limit of movement allowed within the flange of the beam. New transverse cracks have formed near the joints to relieve the thermal stresses. Also, a gradual loss of movement may be the result of the ingress of grit or incompressible detritus into the joint between the WFB and the end of the CRCP slab as a consequence of the poor condition of the joint sealant. Over a further period of time, thermally induced compressive forces could become sufficiently large to cause the beam to distort or break from the concrete sleeper slab.

The seasonal CRCP end movements on Site D are shown in Figure 8.3. For Section B9, the end of the CRCP had initially contracted then gradually extended to reach a length approximately 2mm greater in Summer 1996, after which it has again started contracting. By the same season, Summer 1996, the end of the CRCP in Section B11 had

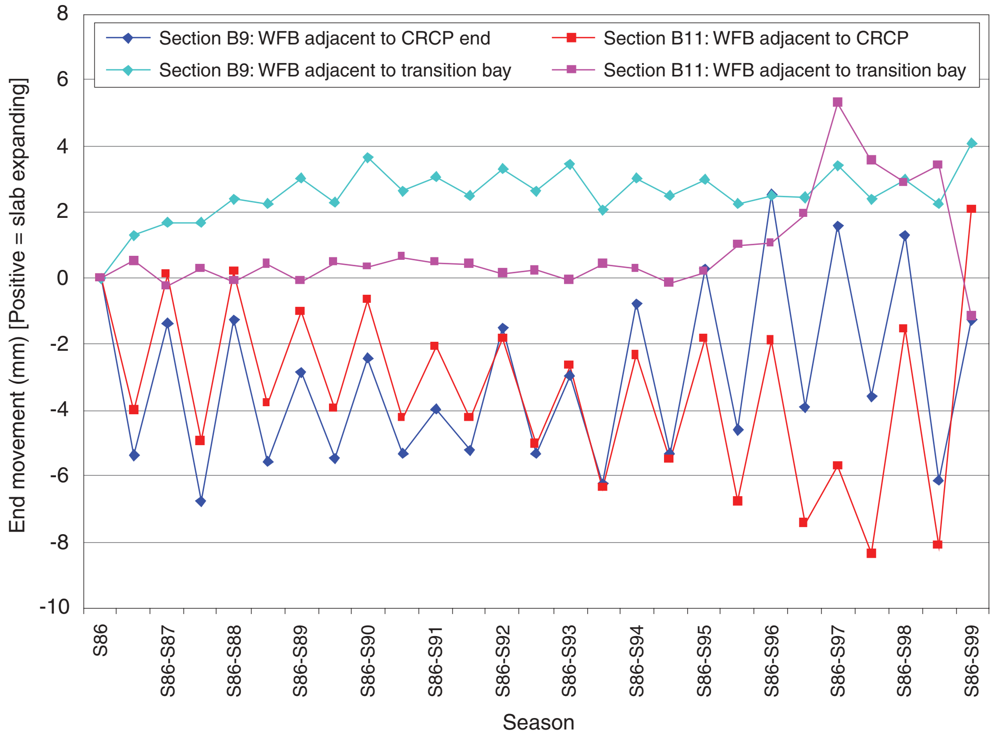

Figure 8.3 Site D seasonal end movements at the WFB

contracted by approximately 2mm but since then the movement between summer and winter has been getting gradually larger. It was observed in February 1998 that traffic passing over the WFB in Section B9 was causing the top flange of the beam to move and 'clatter' against the top of the concrete. A similar problem was observed for the WFB in Section B11, where the end movement of the transition bay adjacent to the WFB had progressively increased to 5mm in Summer 1997 and then subsequently contracted by 6mm in Summer 1999.

Fatigue of the web is the main performance problem with WFB terminations. Large bending moments from traffic loading generate high stresses at the top of the flange, causing a fatigue failure at the vertical web/top flange intersection. Solutions to this problem are to incorporate a metal plate at the ends of the beam to stiffen the top flange and/or to reduce the width of the top flange. The MCHW3 specifies a width of the top flange of either 305mm or 356mm, depending on the CRCP thickness. However, there is a potential to reduce the top flange width to only accommodate the concrete thermal movements and the width of sealing grooves, thus minimising the crucial bending moment at the vertical web/top flange intersection.

Another problem with the satisfactory long term performance of the WFB is the poor condition of the joint sealant, which has been noted on a number of sites. The joint sealant had debonded from the WFB next to the CRCP ends on Sites C, D and F, as shown in Figure 8.4. Debonding allows detritus between the web of the steel beam and the CRCP end. At some terminations the width of the joint between the slab end and the WFB had increased in summers, indicating the presence of detritus and incompressible material. Large thermal stresses induced in the concrete between winter and summer could eventually cause the beam to distort or fracture. This effect may be occurring at the two beams being monitored on Site F.

A solution to remedy the debonding problem has been proposed in the MCHW3 by altering the aspect ratio of the seal groove through doubling the depth of the sealing groove section. This detail has been incorporated in the WFB designs on Sites E and G.

Figure 8.4 Debonding of the joint sealant at the WFB

8.3 Factors affecting CRCP end movements

8.3.1 Temperature and surfacing materials

Site E had some sections with different thin asphalt surfacing systems on the CRCP, Sections L1S and L2S have an exposed concrete surface across the full carriageway width and Sections L1N and L2N, have an exposed concrete surface in the hard shoulder and a thin 22mm UL-M surfacing in the three adjacent traffic lanes. Three terminations, Sections L3S, L4N and L4S, have a 40mm thin SMA surfacing on the full width of carriageway. The average slab temperatures for the three different surfacing regimes at the time that the movement measurements were made are shown in Figure 8.5.

This figure shows no clear effect of surfacing materials on the CRCP slab temperature. The maximum temperature difference obtained at any seasonal measurements for the different surfaces was 3.5°C. This difference is small and was also found at other sites with the same surfacing, within the period of taking the measurements at different locations. This indicates that the various surfacing systems on CRCP have only a little effect on the slab temperature.

At this site, slab temperatures at a range of depths have been recorded at 30-minute intervals, over a period of a year. The difference between the maximum and minimum concrete temperatures was found to be 35.6, 33.8 and 33.9°C for the exposed concrete, UL-M and SMA surfacing, respectively. These results show that the various surfacing systems on CRCP have only influenced the annual range of slab temperature by 1.7°C.

8.3.2 Aggregate type

Table 8.4 gives the average end movement of the CRCP measured between the end of the CRCP slab and the WFB for sections with the same CRCP aggregate type and the same CRCP support. For the cement bound granular base (CBGM), the average end movement of the siliceous

Table 8.4 Influence of aggregate and CRCP support on end-movements

	CRCP end movement (mm/30°C)	
CRCP support	*Siliceous gravel*	*Limestone*
CBGM	8.64	7.51
Asphalt regulating layer	12.10	9.64

gravel is 8.64mm/30°C, compared to 7.51mm/30°C for the limestone aggregate. A similar trend could be seen from the asphalt regulating layer, where the siliceous gravel gave an average value of 12.1mm/30°C, which is about 25 per cent higher than that of the limestone of 9.64mm/30°C.

Coarse aggregate has the greatest volume of the concrete constituents, and therefore greatly influences the thermal properties of the concrete. In the USA, some states limit the coefficient of thermal expansion of concrete to a maximum value of 6×10^{-6} per °F, which equates to 10.8×10^{-6} per °C. In the UK, there is no requirement to control the thermal expansion of concrete pavements, however, typical values for siliceous gravel concrete are 11 to 13×10^{-6} per °C and for limestone concrete are 5.9 to 7.4×10^{-6} per °C (Neville, 1995). It is recommended that the CRCP end movement design should take into account the coefficient of thermal expansion of the concrete.

8.3.3 Subbase support

The frictional forces between the CRCP slab and the underlying material influence the thermal expansion and contraction of the concrete. Different underlying materials offer various degrees of restraint to CRCP ends. Wu and McCullough (1992) reported that the frictional force significantly affects the CRCP end movements. Field data showed an increase in the movement coefficient by about 50 per cent when an asphalt stabilised subbase was used in place of a cement stabilised subbase.

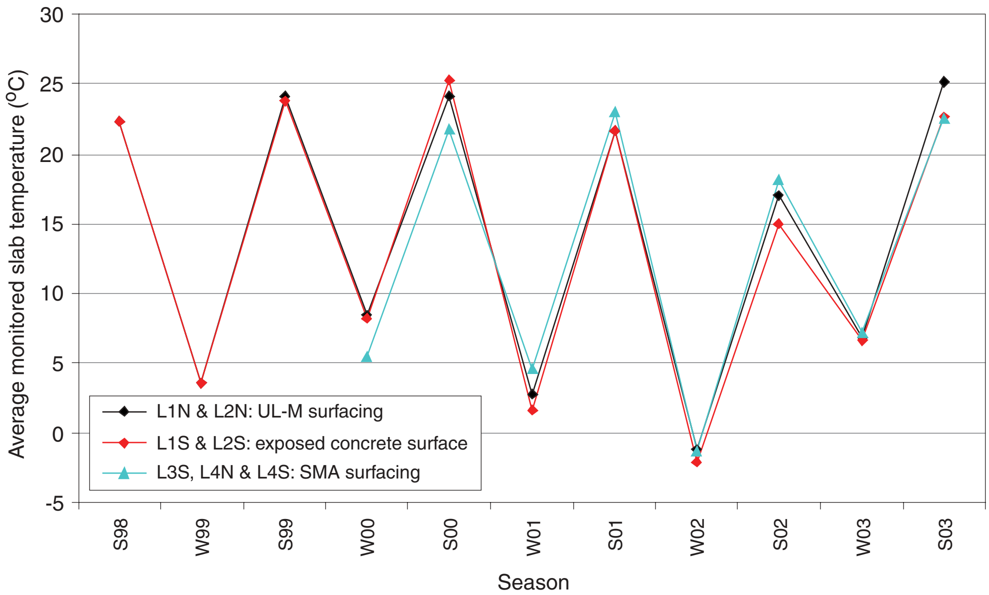

Figure 8.5 Site E concrete slab temperatures for sections with different surfacing

When comparing the cement bound subbase to the asphalt regulating layer, Table 8.4 shows that for the same aggregate type of siliceous gravel, the average CRCP end movement increased from 8.64mm/30°C on a CBGM to 12.1mm/30°C on an asphalt regulating layer; an increase of about 40 per cent for the asphalt regulating layer. For the limestone CRCP, there was a corresponding increase of about 28 per cent for the asphalt regulating layer.

The effect of frictional forces from unbound, Type 1 granular material and planed asphalt with reference to CBGM subbase is given in Table 8.5 for GBA terminatons. The average CRCP end movements for concrete containing siliceous gravel on the CBGM and the unbound Type 1 subbases are 3.92 and 5.09mm/30°C, respectively. This is an increase of about 30 per cent for the unbound, Type 1 subbase. However on sites with planed asphalt and CBGM subbases, the average CRCP end movements are similar, indicating similar frictional forces.

Table 8.5 Effect of subbase type on end movements

Subbase type	*CRCP end movement (mm/30°C)*	
Sites with CBGM and unbound	CBGM 3.92	Unbound, Type 1 5.09
Sites with CBGM and planed asphalt	CBGM 5.26	Planed asphalt 5.34

In summary, a planed asphalt provides similar frictional forces to CRCP ends as a CBGM subbase, whereas a Type 1 unbound subbase increases the CRCP end movements by about 30 per cent, and an asphalt regulating layer by up to 40 per cent.

It is recommended to increase the subbase friction near terminations over a minimum of 50m, by roughening the subbase surface to reduce the CRCP end movements.

8.3.4 Termination type

Table 8.6 gives the average joint movements for Site A with GBA and Site E with WFB terminations, both constructed in accordance with the current MCHW3. On both sites the CRCP had an exposed concrete surface and was made with limestone aggregate. The underlying material was planed asphalt for Site E and CBGM for Site A, which were shown above to provide similar frictional forces to the CRCP ends.

The average total thermal movement across expansion joints for the GBA terminations was 10.5mm and represents the combined thermal movement of the CRCP end and the expansion joints between the transition bays. However, the average thermal movement across the expansion joints for the WFB terminations was less, being 6.3mm, and represents the total thermal movement from the transition bays only. Therefore, it can be deduced that the movement taken by the ground anchor beams for a 30°C range is (10.5mm – 6.3mm), i.e. 4.2mm. By comparing this value with the 7.1mm for the unrestrained CRCP end, i.e. the WFB termination, the ratio of movement for the restrained end to the unrestrained end is 0.6. Therefore, this indicates that the ground anchor beams only restrain approximately 40 per cent of the total unrestrained CRCP end movement. This is in close agreement with Teng (1970), who showed that concrete ground anchor beams restrained only about 50 per cent of the end movement of a CRCP termination; the remaining movement was accommodated by the expansion joints in the transition bays.

Table 8.6 Average joint movements (mm/30°C) for different termination systems

Average joint movements (mm/30°C)		
Ground beam anchorage	*Wide-flange beam*	
Expansion joints between transition bays	*CRCP end*	*Expansion joints between transition bays*
10.5	7.1	6.3

8.4 Proposed new termination design

The two termination systems currently specified in the UK are relatively expensive to construct. In addition, the GBA system does not take all the CRCP end movement with uncertainties about the number of transition bays needed, and the WFB system can suffer from fatigue failures at the web of the steel beam.

PIARC (1994) has indicated that six countries use transverse expansion joints, similar to those used for bridges. Aunis (1990) indicated the use of bridge-type expansion joints, with a movement range of 50mm, as CRCP terminations in France, but highlighted potential problems associated with a lack of regular maintenance. French practice involves generating a greater restraint between the slab and the subbase by roughening the surface, for example by milling.

To overcome the high cost and defects of the current UK termination designs, it is proposed that a termination design should be developed based upon bridge-type joints. The design should be especially suitable where a thin low noise surfacing is required over a CRCP slab, and should also be used as a maintenance technique where the conventional terminations require repair.

Further research will be needed to investigate the potential use of the bridge-type joints for use in the UK with respect to the optimum dimensions for the system and the number of transition bays required

GBA terminations designed to the MCHW3 showed satisfactory long-term performance, but only restrain 40 per cent of the CRCP end movement.

Increasing the number of anchor beams from four to six has not reduced the amount of end movement. It would be prudent to continue using four anchor beams.

In WFB terminations there is the potential to reduce the number of transition bays from four to two where the termination is adjacent to a flexible pavement.

Debonding of the joint seal and fatigue of the beam are the main problems influencing the long-term performance of WFB terminations.

Siliceous gravel in CRCP increases the end movement by about 25 per cent compared to limestone.

When compared to a CBGM subbase, a planed asphalt is similar whilst an unbound Type 1 subbase increases the CRCP end movement by about 30 per cent, and an asphalt regulating layer by up to 40 per cent.

9 Summary of the new CRCP designs

A summary of the differences between the current and the new CRCP designs, and examples of the required thicknesses for a range of mean flexural strengths and foundations classes are given below.

9.1 Concrete strength

Current UK designs for CRCP given in HD26/01 (DMRB 7.2.3) are based on equations given in RR87 for old concrete data, constructed before 1975, which used the compressive strength as the design criterion. A review of international standards and practices in twelve countries showed that nine countries use the flexural strength to specify concrete as it is more related to the structural performance of concrete pavements. Laboratory and site data have been used to establish reliable relationships between flexural and compressive strengths.

The new CRCP designs, based on the flexural strength of concrete, are independent of the aggregate type and give consideration to the enhanced properties of modern concrete, as opposed to old concrete, due to the refinement in cement manufacture and improved characteristics at the cement/aggregate interface. The flexural strength of a Class C32/40 concrete, on which the current design curve is based, would now be expected to achieve a flexural strength of at least 5.0MPa.

Rather than designing for a single strength of concrete, the new design curves for CRCP are given for a range of flexural strengths with the benefit of a reduced slab thickness for higher strength concrete. A reduction in CRCP slab thickness of up to 60mm could be achieved by increasing the flexural strength of concrete to 6.0MPa.

9.1.1 Current designs

The current thickness designs for CRCP in HD26/01 (DMRB 7.2.3) are based on:

- A Class C32/40 concrete for pavement quality concrete, approximately equivalent to a mean compressive strength of 50MPa at 28 days.

9.1.2 New designs

The new CRCP designs are based on:

- A family of CRCP thickness design curves based on the flexural strength of the concrete (from 4.5MPa to 6.0MPa).

9.2 Foundations

The current UK requirements, given in HD25/94 (DMRB 7.2.2), assume a foundation with an equivalent surface foundation modulus (ESFM) of 270MPa and require the use of a capping layer for a subgrade with a CBR of 15 per cent and only permit cement bound material subbases of CBM3 or Grade C15 (now Class C12/15) concrete. The higher the strength of the cemented subbase the increased tendency there is of discontinuity of foundation support. Therefore, transverse cracks are to be induced when the average minimum subbase strength at 7-day is ≥10MPa.

Practical experiences in the UK have indicated the potential to lower the minimum strength requirement of the formation without capping to below 15 per cent CBR. It has been found that a minimum CBR of 5 per cent is adequate to compact a cement bound subbase layer without a capping layer. A review of international specifications has indicated the significantly higher strength requirement for cemented subbases under CRCP in the UK compared to other countries. Recommendations are made to achieve a more economic construction by specifying lower strength, bound and durable subbase materials and the use of the new foundation classes with a wider range of foundation materials and modulus values.

9.2.1 Current designs

The current designs for rigid and rigid composite (DMRB 7.2.2) require:

- A capping layer when the subgrade CBR is less than 15 per cent.
- A cemented subbase of CBM3 or wet lean concrete of Class C12/15. If the cumulative traffic loading is less than 12msa, a CBM2 or wet lean concrete of Class C8/10 is permitted.
- A minimum assumed equivalent surface foundation modulus (ESFM) of 270MPa.

9.2.2 New designs

The new designs include:

- Subbase only for CBR ≥ 5 per cent.
- Bound subbases in Foundation Classes 2, 3 and 4 with a design ESFM of 100MPa, 200MPa and 400MPa, respectively.
- Use of a wider range of cement bound and other hydraulically bound mixtures to BS EN 14227 (2004) for subbases.

9.3 Tied shoulders and edge strips

Tied shoulders and edge strips have the benefit of reducing the stresses along the edge of a CRCP slab. The current designs given in HD26/01 (DMRB 7.2.3) require that when a tied shoulder or 1m edge strip is not included in the construction of a CRCP the slab thickness is increased. No changes are made in CRCP thickness between the current and new designs. However, it is recommended that:

- Asphalt shoulders are not used alongside a CRCP.
- CRCP shoulders are not used alongside jointed concrete pavements.

9.4 Reinforcement

The amount of longitudinal steel in the CRCP determines the crack pattern and hence performance of the pavement. The transverse reinforcement is beneficial in forming a regular transverse crack pattern in the slab; more random crack patterns have formed where the transverse steel has been omitted. It is proposed that:

- The transverse reinforcement is retained to aid the formation of a satisfactory crack pattern.
- Consideration should be given to placing the longitudinal steel at third-depth when siliceous gravel aggregate is used in CRCP.
- Adjustments should be made to the amount of longitudinal reinforcement to take into account the different strengths of concrete proposed in the new CRCP designs.
- Until further research is undertaken a conservative value, of 900mm^2/m width of pavement, of reinforcement is employed in the new designs.

9.5 Traffic loading

No changes are proposed in the maximum cumulative traffic loading between the current and new CRCP design curves. It has been shown that:

- The predicted maximum 40 year cumulative traffic loading of the heaviest trafficked pavements in the UK is 260msa.
- The current maximum cumulative traffic loading of 400msa should be retained to allow for the possible effect of increasing the maximum permitted axle load.

9.6 CRCP terminations

9.6.1 Current designs

The designs given in the MCHW3 for terminations to a CRCP show:

- Two termination types; a ground beam anchorage and a wide-flange steel beam.
- Four transition bays, each 5m long constructed as jointed reinforced concrete and separated by expansion joints.

9.6.2 Proposals for the new designs

Proposals for amendments to the current designs for CRCP terminations given in the MCHW3 are:

- Reducing the movement in the end 50m of the CRCP by increasing the subbase friction and/or limiting the coefficient of thermal expansion of the concrete.
- Reducing the number of transition bays from four to two at wide-flange steel beam terminations when constructed adjacent to a flexible or flexible composite pavement.
- Developing a more economical termination system using bridge-type joints.

9.7 CRCP design equations

9.7.1 Current designs

The current CRCP design curve is derived from the equation for jointed reinforced concrete given in RR87:

$$\mathrm{Ln}(H_1) = \{\mathrm{Ln}(T) - 3.17\,\mathrm{Ln}(f_c) - 0.33\,\mathrm{Ln}(M) - 1.42\,\mathrm{Ln}(R) + 45.15\}/4.79 \qquad (9.1)$$

Where

H_1 = CRCP slab thickness for an untied shoulder (mm).
T = Cumulative traffic loading (msa).
f_c = Mean concrete compressive strength at 28 days (MPa).
M = Equivalent surface foundation modulus (MPa).
R = Reinforcement content, cross-sectional area of steel per metre width of slab (mm^2/m).

For CRCP with a tied shoulder or 1m edge strip, the required CRCP thickness (H_2) is given by:

$$H_2 = 0.934\,H_1 - 12.5 \qquad (9.2)$$

9.7.2 New designs

The thickness design equation for a CRCP slab with either an untied shoulder or an edge strip less than 1m, in relation to traffic, concrete flexural strength and foundation at the reinforcement content used in current designs is given by:

$$\mathrm{Ln}(H_1) = \{\mathrm{Ln}(T) - 3.17\,\mathrm{Ln}(f_f)^{1.55} - 0.33\,\mathrm{Ln}(M) + 30.47\}/4.79 \qquad (9.3)$$

Where

H_1 = CRCP slab thickness for an untied shoulder (mm)
T = Cumulative traffic loading (msa)
f_f = Mean concrete flexural strength at 28 days (MPa)
M = Equivalent surface foundation modulus (MPa)

Equation 9.2 is still applied when a tied shoulder or 1m edge strip is included in the design.

The new thickness design curves for CRCP are shown in Figures 9.1, 9.2 and 9.3 for a range of mean 28-day flexural strengths on Foundation Classes 2, 3 and 4, respectively, and a tied hard shoulder or 1m edge strip.

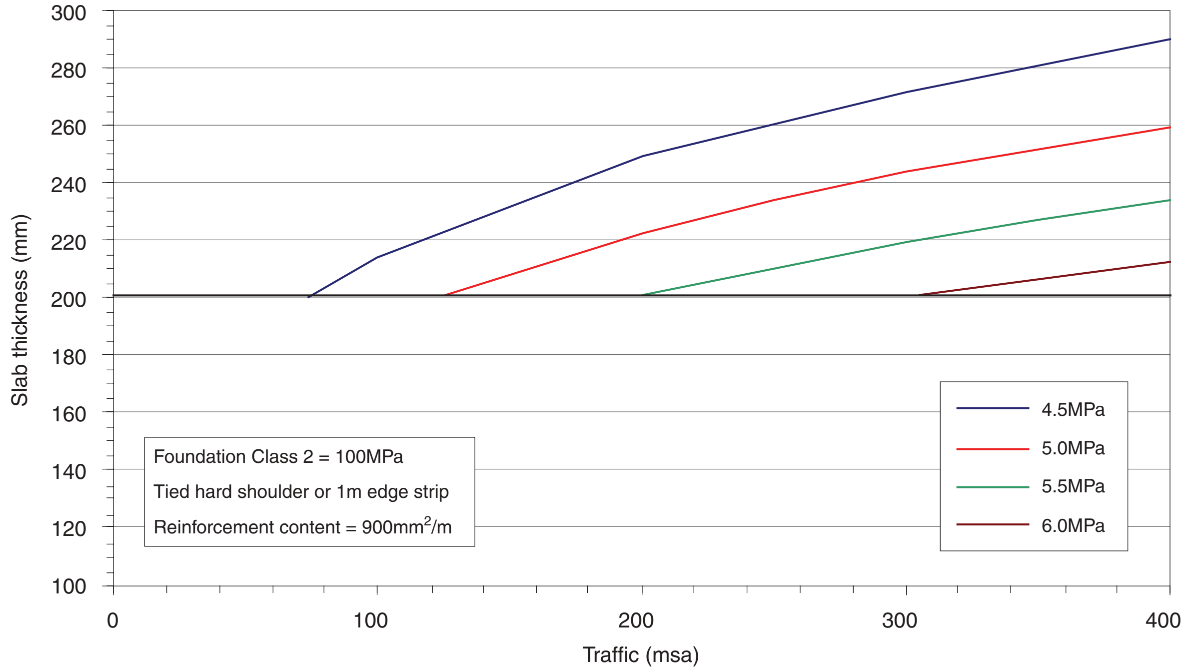

Figure 9.1 CRCP thickness design curves for Foundation Class 2, bound subbases

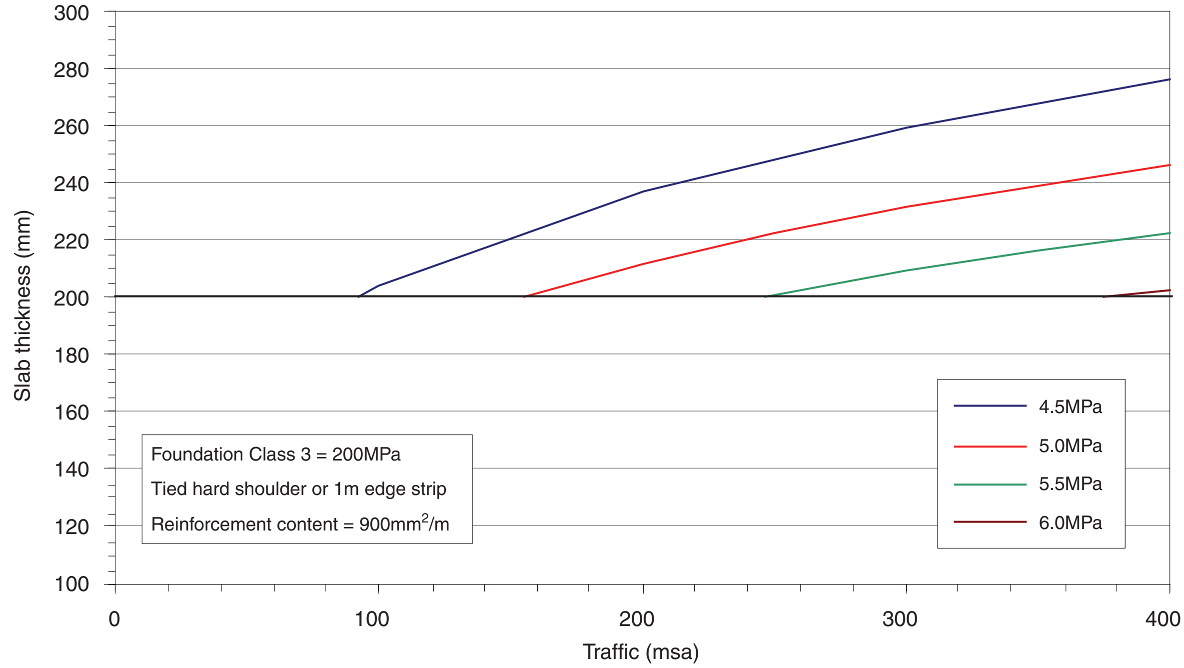

Figure 9.2 CRCP thickness design curves for Foundation Class 3

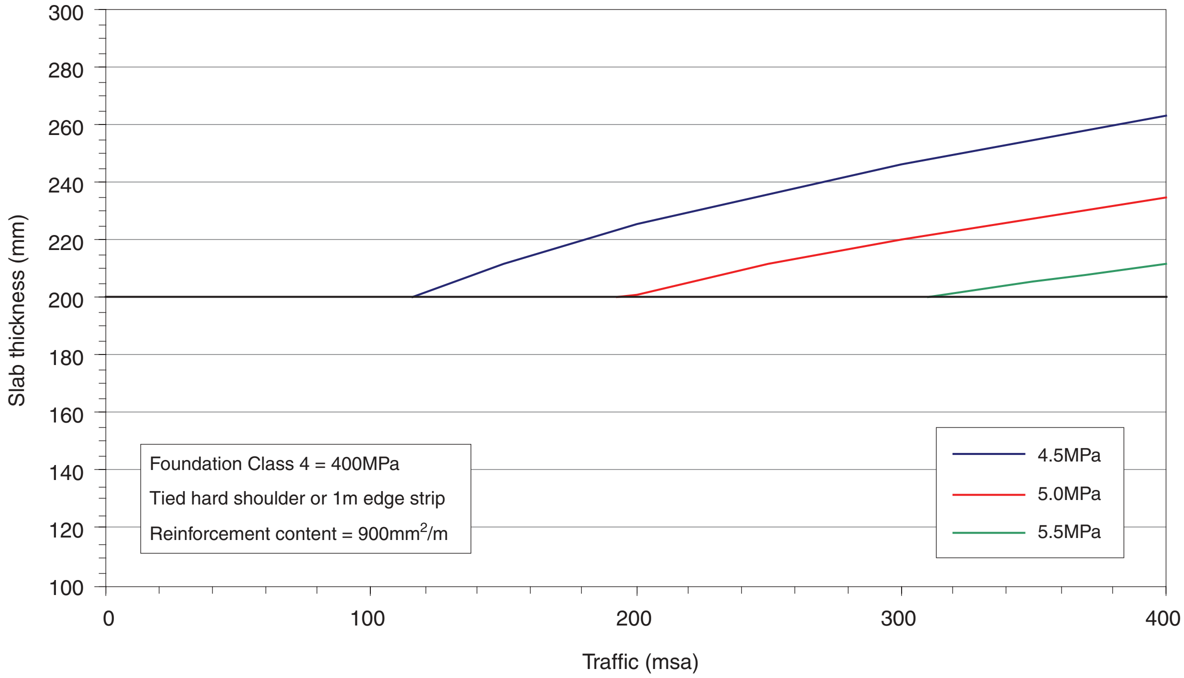

Figure 9.3 CRCP thickness design curves for Foundation Class 4

10 Conclusions and recommendations

The following conclusions have been drawn from the work detailed in this report:

- Aggregate type has more influence on the cracking pattern of CRCP than the subbase type. Higher percentages of medium and wide cracks and other defects occurred with siliceous gravel than limestone aggregate. However, locating the reinforcement at third-depth significantly improves the crack pattern of siliceous gravel CRCP.
- New CRCP designs have been developed based on the flexural strength of concrete, which is more related to the structural performance of CRCP. The new designs are independent of the aggregate type and utilise the benefit of a higher flexural strength.
- The threshold chloride concentration to initiate corrosion in CRCP is relatively high. Corrosion initiates within the pavement life, and occurs mainly in the transverse reinforcement coinciding with the transverse cracks. However, the corrosion damage has no significant effect on performance.
- The current UK requirement for subbase strength is significantly higher than in other countries. The new CRCP designs consider the use of subbase only for CBR ≥5 per cent and incorporate a wider range of cement bound subbases and other hydraulically bound mixtures, with significant economic and environmental benefits.
- Bound subbases from Foundation Class 2, and Foundation Classes 3 and 4 with a minimum equivalent surface foundation modulus of 100, 200 and 400MPa, respectively, allow the use of a wider range of secondary and recycled materials. The foundation stiffness has little effect on the thickness design of CRCP.
- The use of hard shoulders and hard strips reduces edge stresses and CRCP slab thickness. Recommendations are given against using an asphalt shoulder alongside a CRCP or a CRCP shoulder alongside a jointed concrete pavement.
- The predicted maximum 40 year cumulative traffic loading of the heaviest trafficked pavements in the UK is 260msa. However, it is recommended to retain the current maximum cumulative traffic loading of 400msa in the design curves to allow for the possibility of an increase in the maximum permitted axle load.
- Ground anchorage beam terminations restrain only 40 per cent of the CRCP end movement but require no maintenance. Properly designed and constructed wide-flange beam terminations can accommodate the full CRCP end movement, and therefore the number of transition bays could be reduced to 2 when adjacent to flexible pavements.
- There is the potential to develop a more economic termination system using bridge-type joints.
- Designs based on flexural strength and the new foundation classes will result in a reduced slab thickness requirement in the new CRCP design curves when a 28-day mean flexural strength of 5.0MPa is achieved.

11 Acknowledgements

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Abstract

The aim of this project was to assist the Highways Agency in re-assessing current designs and specifications for continuously reinforced concrete pavement (CRCP) in the light of performance data from the UK and other countries. The findings of this work have been used to develop more economic designs for sustainable long-life roads, which would give increased value for money and support the Government aims for sustainable construction.

The performance and design parameters investigated were crack patterns, concrete strength, steel reinforcement, foundations, hard shoulders and edge strips, traffic loading and terminations. The effects of these parameters on the structural integrity and durability of CRCP were assessed and the results were used to develop new design curves.

The performance of CRCP is mainly determined by the condition of the surface cracking and defects, with the greatest influence arising from medium, wide and bifurcated cracks, and localised punchouts. It was found that the aggregate type in CRCP has more influence on the cracking pattern than the subbase type. Locating the reinforcement at the third-depth of the slab significantly improves the crack pattern of CRCP made with siliceous gravel.

A review of international standards and practices has shown the widespread use of flexural strength rather than compressive strength for design purposes, which is more related to the structural performance of pavements. Reliable relationships between flexural and compressive strength were established and used to develop new CRCP designs with thinner slabs for higher strength concrete.

Cracks in CRCP provide the route for chlorides, from de-icing salts, to penetrate the slab and initiate reinforcement corrosion. Corrosion mainly occurs in the transverse reinforcement, which tends to be coincident with the transverse cracks, with no evidence of significant corrosion in the longitudinal reinforcement. No significant consequences of corrosion damage on the performance of CRCP have been found in the UK.

The currently specified cemented subbase under CRCP in the UK has significantly higher strength than that used in other countries. This high strength can increase the risk of discontinuity of the foundation support. The new designs consider lowering the subgrade strength requirement for a subbase only, without capping, to 5 per cent CBR. Also, the use of bound materials in Foundation Classes 2, 3 and 4, could result in significant economic benefits, as they may incorporate alternative materials.

Recommendations are given against the use of asphalt shoulders alongside CRCP and to retain the maximum cumulative traffic loading given in the current designs to allow for the possibility of an increase in the maximum permitted axle load.

Thermal movements at CRCP terminations indicated that the ground beam anchorage system restrains only 40 per cent of the CRCP end movement but requires little maintenance. For the wide-flange beam system, the CRCP end movement is mainly accommodated by the joint between the CRCP and the steel beam, and therefore there is a potential to reduce the number of transition bays. Recommendations are given to reduce the amount of thermal movement at terminations by locally increasing the subbase friction and/or reducing the coefficient of thermal expansion of the concrete. A proposal is made to develop a more economic termination system based on bridge-type joints

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