

DESIGN PROCEDURE FOR CRCP BASED ON THEORETICAL CONSIDERATIONS AND SERVICE BEHAVIOUR

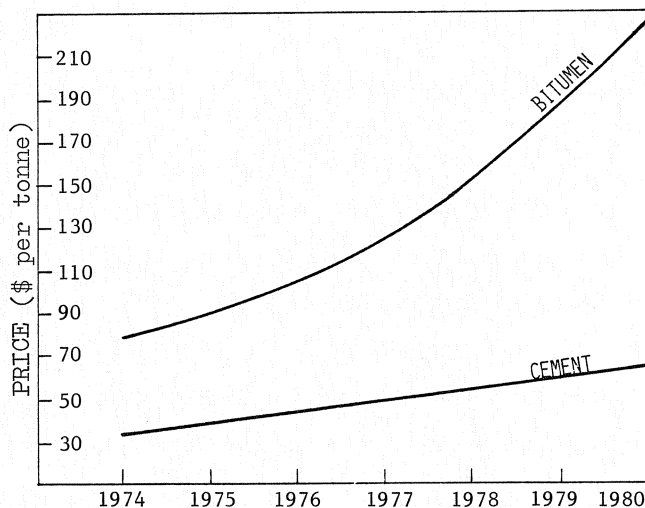
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This paper outlines the growth in the use of concrete pavements in Australia in recent years, with particular emphasis upon the most populous State of New South Wales. The development in the use of CRCP is traced and its behaviour recorded, culminating in the adoption of a simplified design procedure based upon the environmental cracking approach necessitating some assumptions pertaining to bond strength. Consideration is also given to the specification and control of CRCP construction, including the stipulation of an upper limit on concrete strength.

In the last five years, there has been an increase in the use of concrete pavements in Australia, particularly in New South Wales (NSW), the most populated state in the Commonwealth. (Population 5 million or 35% of Australia's 14 million.) Since 1975, it is estimated that rigid pavement construction in NSW has increased by 25% annually to a 1980 level of approximately 30 km (20 miles) of two-lane road annually. The major constructing Authority is the Department of Main Roads (DMR).

The reasons for this increase include the oil crisis and traffic growth. Australia in 1980 obtained approximately 60% of its petroleum needs from indigenous crude oil. However, Australian crudes are deficient in the heavy fractions necessary for bitumen production, and hence all bitumen is derived from imported crude. Figure 1 shows relative price increases of bitumen and portland cement from 1974 to 1980. The relatively lower increase in cost of cement plus its independence of overseas supply sources has enhanced concrete's cost competitive position. Again, increased traffic volumes and legal axle load limits have made rigid pavements more structurally efficient in accommodating traffic loads. Other factors have included desire for lower maintenance and the availability of modern construction methods (e.g. slip-form pavers).

Figure 1. Sydney prices for cement and bitumen as of June each year in \$Aust. (\$US1.12 - 1.20).



Historical Developments

NSW has had a long history of concrete pavements. Some Sydney streets had been paved with wooden blocks on a concrete base as long ago as 1880. The first concrete road was constructed in 1908, and in the 1930's many kilometres of highways, urban arterials and residential streets were paved. Most are still in use, particularly in Sydney urban arterials, although faulting has required asphalt overlays on many. Nevertheless, these concrete roads have required relatively low maintenance.

It is also clear that modern improvements, particularly in detailing rather than thickness design, has minimised many of the problems associated with these 50 year old concrete pavements, for example, the elimination of expansion joints, (where previously full expansion joints were placed at approximately 12 m (40 ft) centres), recognition of the need for non-pumping sub-bases (where previously some concrete roads had been placed directly on clay subgrades),

improvements in joint sealant properties, and indeed, the very elimination of joints such as in CRCP. Against this background, the implementation of modern concrete pavement practices has developed. Some significant milestones in this development will now be outlined.

The first CRCP on a public road in Australia was on the reconstruction of Brady Street, South Melbourne, Victoria, in 1970. Its use, rather than conventional flexible construction, was prompted by a major increase in truck traffic plus the presence of existing services whose costly relocation was avoided due to the shallow depth of rigid construction (1). About this time, Sydney's first major urban freeway, the Warringah Expressway was under construction, and a 200 mm (8 in) jointed reinforced pavement, without expansion joints, was selected to minimise maintenance. A thin asphalt wearing surface was placed on the concrete, and after ten years of traffic, no maintenance has been required. Again, in the period 1970-1980, many Sydney urban arterials, previously constructed two lanes wide in concrete up to 50 years ago, required widening to four lanes, and reinforced concrete pavements, jointed to match the existing central pavement, but with contraction rather than expansion joints, were found to be the better pavement solution. And finally, many lengths of existing flexible pavements on urban arterials, failing under very heavy traffic on clay subgrades, were reconstructed using concrete as the most economic first cost solution. However, these rigid pavements, apart from the CRCP at Brady Street, were generally surfaced with up to 50 mm (2 in) of asphalt, to ensure adequate rideability and preserve a uniform road appearance.

By 1975, the DMR had commenced construction of its first CRCP, on the Pacific Highway, which links Sydney and Brisbane, at Clybucca Flat, near Kempsey. The traffic alone on this rural highway did not demand a rigid pavement. However, at Clybucca Flat, the highway crosses a flood plain, where the CBR is less than 2%, and raising of the road level, required for flexible pavement reconstruction, is impossible, since it would exacerbate flooding in other areas. Frequent inundation by floodwaters had lead to very high maintenance costs on this section. A CRCP solution was adopted, since possible joint problems on the low strength subgrade would be eliminated. A low strength plain concrete sub-base was adopted, principally as a construction expedient (2). The success of this expedient was to have significant results in later years.

Meanwhile, the increase in use of concrete pavements had extended to local government (city and municipal) and other road authorities. In particular, the Housing Commission of NSW began construction of 100 mm (4 in) thick reinforced concrete pavement in residential housing developments. From 1975 to 1980, 20 km (13 miles) were constructed. In 1980, the largest recent municipal project, approximately 8 km (5 miles) of collector road, was commenced. This pavement is 200 mm (8 in) JRCPC, using for the first time in Australia a plastic crack inducer with "zip-off" top section, forming a sealant reservoir for contraction joints at 12 m (38 ft) centres.

The success of the CRCP at Clybucca lead to its use again in 1978 on the Foreshore Road adjacent to Botany Bay, near Sydney, and again in 1979 on the first two stages of pavement construction for the new F4 Western Freeway which links Sydney with Parramatta. CRCP was selected because of the need for low maintenance under very heavy traffic, and a competitive first cost. A further stage of the

Western Freeway has since been approved for construction in CRCP, bringing the total length of CRCP in NSW to 50 lane-km (30 lane-miles).

Construction Developments

Although rail-form mounted concrete paving equipment had been used on the Warringah Expressway and several major airport pavements, by the 1970's all concrete road construction in NSW used timber side forms and double beam vibrating screeds, and construction was one lane width at a time. Surface finish was not critical, since asphalt overlays were the normal practice.

However, for the Clybucca Flat project, the DMR purchased a 'Gomaco C450' self-propelled spreader finisher, which ran on steel rail forms. Although a simple machine, it represented a marked step up in concrete road equipment. The steel rail was used widely on subsequent projects.

For the Foreshore Road project, which formed part of the development of a second port for Sydney on Botany Bay, a British contractor, McGregor (Paving) Ltd imported a slip-form paver to Australia in late 1978 and this was hired by the DMR. Thus, the Foreshore Road was to be Australia's first slip-formed concrete road. A high degree of compaction was achieved, and the surface finish was proved to be very good, the roughness being an average 29 counts per kilometre compared with the usual specification for freeways of no more than 50. The slip-former was subsequently used in 1980 on the F4 Western Freeway. In late 1980, a second road contractor imported a slip-form paver from the USA.

One construction development which is a by-product from the Clybucca project is the growing use of "lean-mix" flyash concrete, with a specified minimum compressive strength of 5 MPa (700 lbf/in²), as both a sub-base "working platform" and not only for concrete, but particularly for flexible pavements. Pavement analysis by such means as elastic analysis had demonstrated the structural efficiency of a stiff sub-base in flexible pavement, but the construction advantages on weak clay subgrades was even more significant. By mid 1980, the annual use of lean concrete sub-bases in the Sydney area alone had grown to over 15 000 cubic metres (20 000 cubic yards).

General Design Developments

In 1980, the DMR issued a revised and upgraded Pavement Thickness Design Manual, which contains design methods for flexible pavements, overlays and rigid pavements. The thickness of the rigid pavement is based on an assessment of subgrade strength (CBR), concrete strength, and the predicted traffic volume and composition over the design period, in terms of ESA's. (One ESA = a single axle with dual wheels loaded to a total mass on 8.2 tonnes (18 000 lbs)). The same thickness is used for jointed unreinforced, jointed reinforced and continuously reinforced concrete pavement. Comprehensive details of reinforcement, joints, sub-bases etc. for these three pavement types are included in the Manual. Incidentally, for flexible pavement design, the structural value of low strength concrete sub-base is recognised by according it an "equivalence" of up to 2.5 when used more than 250 mm (10 in) below the road surface. ("Equivalence" refers to a thickness reduction factor when compared to a standard

Table 1. Details of CRCP in New South Wales.

Description	Details			
Location	Clybucca	Botany	Homebush	Mays Hill
Length (lane-km)	11.0	11.5	13.5	13.5
Date Constructed	1975-76	1978-79	1979-80	under construction
Design ESA ^a , 10 ⁶	8	27	45	31
Subgrade CBR	1-2	3-8	3-6	2-6
Sub-base Type	lean concrete	lime-treated crushed rock		lean concrete
Sub-base, mm	130	125	100	100
Thickness, mm	230	200	200	200
C16 (No.5) Bar Spacing, mm	140	150	150	150
Steel Ratio, %	0.60	0.67	0.67	0.67
Design 90-days Modulus of Rupture, MPa	4.4	4.8	4.9	4.9
Specified 28-days Compressive Strength, MPa	23-33	30-38	30-40	30-40

Note: 1 km = 0.62 mile; 1 mm = 0.04 in; 1 MPa = 145 lbf/in².

^aESA = equivalent 8.2 t standard axles over 40 years.

natural gravel or crushed rock.)

The Manual gives specific guidance for the design of CRCP, based on theoretical analysis and the performance of the CRCP's built between 1975 and 1980. These projects, together with an explanation of the design method, are detailed below.

Observation of Service Behaviour of CRCP

To date there are only three public roads in NSW of CRCP construction (3). A fourth is currently under construction. These are summarised in Table 1. The first two - one a rural highway and the other an urban export road, each totalling about 5.5 km (3.5 miles) in length of two-lane carriageway - were opened to traffic in 1976 and 1979 respectively. Both pavements are regularly monitored, not only for surface crack patterns, but also strain and temperature readings taken from vibrating wire and electrical resistance strain gauges and thermistors buried within the pavement. The bulk of the instrumentation has been undertaken by the Australian Road Research Board and a detailed analysis of results is recorded elsewhere (2, 4). The instrumentation serves as a check on the CRCP design based upon overseas practice (5, 6).

The third completed road comprised of CRCP construction is an urban freeway, yet to be opened to traffic other than construction traffic. Two stages totalling 6.75 km (4.2 miles) in length of two-lane carriageway - one hand placed and the other slip formed - were completed in November 1979 and May 1980 respectively. Of this length, about 1.6 km (1 mile) of the hand-placed CRCP contained fly ash pozzolan (25% by mass of cement) on an experimental basis for comparison with the adjoining CRCP comprised solely of cement concrete. Table 2 summarises the transverse crack patterns measured three months after casting in the spring. Though the fly ash concrete tended to exhibit the anticipated favourable trans-

verse crack pattern of narrower cracks at broader spacing, its further use is in abeyance on account of its greater propensity for plastic shrinkage which can result in adverse longitudinal cracking.

Table 2. CRCP surface crack patterns effect of inclusion of fly ash in concrete.

Crack Parameter	Type of Concrete	
	With Fly Ash	Without Fly Ash
Mean Crack Width, mm	.25	.30
Mean Crack Spacing, m	1.9	1.7
Cumulative Crack Width (millimetres per metre)	.13	.18

Note: 1 mm = 0.04 in; 1 m = 3.28 ft.

A summary of some typical transverse crack patterns achieved on the two earlier projects is shown in Table 3. The tabulation depicts both the crack development with time and the effect of seasonal variation in readings. The two patterns shown are considered to be significantly different, this being attributable not only to the strength difference indicated but also possibly to the presence of a heavily textured concrete sub-base underlying the lower concrete strength CRCP.

The season of casting is also deemed significant. Though both patterns shown in Table 3 were achieved following the casting of concrete in

Table 3. CRCP surface crack patterns variation with time and season of measurement.

Description of Parameter	Details of CRCP Behaviour									
Season of Casting	Summer (S)									
Characteristic Compressive Strength	23 MPa					30 MPa				
Age of Concrete, months	1	5	12	20	25	3	6	9	18	
Season of Measurement	A	W	S	Sp	A	A	W	Sp	W	
Mean Crack Width, mm	.20	.20	.13	.17	.19	.31	.29	.26	.30	
Mean Crack Spacing, m	1.8	1.1	1.0	1.0	1.0	2.7	1.7	1.6	1.4	
Cumulative Crack Width, mm/m	.11	.17	.13	.17	.20	.11	.17	.16	.21	

Note: 1 MPa = 145 lbf/in²; 1 mm = 0.04 in; 1 m = 3.28 ft.

summer, a carriageway adjoining the section shown first in the Table was cast in winter. This resulted in a particularly uniform closely-spaced pattern of fine cracks, with both crack widths and spacings approaching half the values recorded in Table 3 for the adjoining carriageway. Furthermore, the incipient crack pattern appeared to develop overnight, immediately following casting, indicating cracking as a result of the thermal shock produced by the rapid temperature drop occurring during a cold winter's night. The mechanism of cracking in this instance is probably flexure rather than direct tension (7).

The effect of seasonal variation in readings depicted in the tabulation is paralleled by the strain gauge readings recorded at cracks. The strain variation produced by diurnal temperature variations proved to be of the same order as that produced by seasonal temperature variations.

At Clybucca the daily fluctuation in steel strain at a crack throughout the winter period was usually between 600 and 1000 microstrain and in summer between 300 and 600 microstrain. Assuming a modulus of 200 GPa (29×10^6 lbf/in²) for steel, the daily stress fluctuations at a crack in winter would be between 120 MPa (17 400 lbf/in²) and 200 MPa (29 000 lbf/in²) and in summer between 50 MPa (7250 lbf/in²) and 120 MPa (17 400 lbf/in²).

The behaviour at all instrumented CRCP sites shows the same general pattern, with temperature being the most striking factor affecting the strain particularly at a crack. The concrete segments between the cracks expand and contract with fluctuations in temperature. During life, other cracks open up and lower the tension. The system is a dynamic one with strains constantly changing (4).

Theoretical Considerations in Design of CRCP

A study of the available literature on restrained cracking of concrete and CRCP theory consistently highlights the significance of the bond strength between the steel and concrete in determining the ultimate crack patterns (6, 7, 8, 9, 10). The influence of bond strength on the ultimate crack pattern is clearly manifested by virtue of the fact that the complete potential crack pattern comprises cracks at spacings between once and twice the "bond slip zone" length, L_s , with a mean of $1.5 L_s$ (7, 8, 9).

For the complete potential crack pattern to develop, it is necessary for the combined thermal

and shrinkage strain in the concrete to be sufficiently large to produce cracks at all positions which are theoretically possible. Under many conditions, the potential shrinkage and temperature strain may be less than the strain required to develop the full potential crack pattern, and correspondingly fewer cracks would be formed (8).

This normal type of stunted crack development can alternatively be described in terms of a slab's behaviour becoming progressively "softer" as each crack forms and the amount of shrinkage required to produce each new crack increases. At the same time, the rate of shrinkage or contraction will be progressively reducing and the tensile strength of the concrete may be increasing. Therefore it is unlikely that the cracks will form the complete potential pattern. However, it can be shown that a constant maximum crack width is generally achieved early in the crack development process (7), while the crack interval or spacing gradually reduces with time as shown in Table 3.

Leaving aside variations resulting from diurnal and seasonal temperature effects, the crack width remains constant and its value is determined by reference to the ultimate potential crack pattern configuration based upon spacings between once and twice the "slip zone length", L_s (8, 9).

The formation of the full potential crack pattern is finally achieved when the bond force between the steel and concrete is equal to the tensile strength of the concrete, that is,

$$f_b L_s \Sigma \pi d = F'_t A_c \quad (1)$$

where F'_t is concrete tensile strength;

f_b is average bond strength between concrete and reinforcement;

d is diameter of reinforcing bar.

Also

$$r = \frac{\Sigma \pi d^2}{4 A_c} \quad (2)$$

where r is ratio of the steel area to the area of the concrete, A_c .

Substituting "r" in the above equation, results in an expression for L_s as follows -

$$L_s = \frac{F'_t}{4f_b} \cdot \frac{d}{r} \quad (3)$$

Thus the theoretical maximum crack spacing becomes -

$$2L_s = \frac{F'_t}{f_b} \cdot \frac{d}{2r} \quad (4)$$

From the theoretical maximum crack spacing, the maximum crack width (w) is derived; and rearrangement of the expression results in the suggested basic design criterion for CRCP in a readily recognisable form as follows:-

$$r = \frac{F'_t}{f_b} \cdot \frac{d}{2w} (\epsilon_s + \epsilon_T) \quad (5)$$

where w is the maximum allowable calculated crack width for which a value of 0.30 mm (0.012 in) may be adopted in normal conditions and 0.20 mm (0.008 in) in severe exposure conditions;

ϵ_s is the estimated shrinkage strain for which a value of 200 microstrain may be assumed under normal conditions and 300 microstrain in dry conditions, provided the concrete mix exhibits a laboratory shrinkage (in accordance with Australian Standard 1012 Part 13) not exceeding 450 microstrain after three weeks air drying;

ϵ_T is the estimated maximum thermal strain from the peak hydration temperature to the lowest likely seasonal temperature. This may be calculated from the expression

$$\epsilon_T = \frac{\alpha}{2} (T_1 + T_2) \quad (6)$$

where α is the coefficient of thermal expansion of mature concrete;

T_1 is the estimated fall in the temperature from the hydration peak to ambient, with 30°C (54°F) the suggested value under normal conditions;

T_2 is the estimated fall in temperature from ambient at the time of concreting to the seasonal minimum, with 30°C (54°F) the suggested value under average Australian conditions.

The term F'_t/f_b represents the ratio of the direct tensile strength of the immature concrete to the average bond strength between the concrete and steel, which may be taken as 1.0 for plain bars and 0.5 for deformed bars (8).

The figure of 0.5 suggested for deformed bars

can be expressed as equivalent to a L_s value of 300 mm (12 in), assuming a 200 mm (8 in) thick slab with 16 mm (5/8 in) deformed bars at 150 mm (6 in) centres - the most common CRCP configuration currently used in Australia. Such an L_s value is of a similar order to that suggested in an identical example adopted by NCHRP in its simplified analysis of environmental cracking of CRCP (10).

Nevertheless, elsewhere, somewhat larger values for L_s are implied. The "rule of thumb" of 40d (9) must L_s in the absence of more detailed information be necessarily conservative. Other suggested values expressed in terms of "no bond length" (assumed to be about half of L_s) are 250 mm (10 in) for 16 mm (5/8 in) deformed bar, 150 mm (6 in) for 12 mm (1/2 in) deformed bar and one pitch, usually 200 mm (8 in), for plain wire fabric (7). However, the latter figures appear to have been adopted in conjunction with a high modular ratio, implying applicability to early-age concrete with its attendant lower rate of development of bond strength relative to tensile strength.

Hence in the adoption of the environmental cracking approach to the design of CRCP, there is a ready recognition of the dependence of crack widths and spacing upon the bond characteristics of the concrete/steel interface. Provided that the critical reinforcement ratio (determined by balancing the critical steel and concrete forces) is exceeded, an increase in strain will lead to the formation of further cracks until the stage is reached that no more cracks can form because of there being insufficient available anchorage, i.e. complete potential crack pattern. (Further strain must then be relieved by bond slip and the widening of existing cracks, leading to the ultimate failure of the CRCP.)

The order of crack widths achieved in practice (recorded as surface measurements in the preceding section) is in accord with the suggested limiting value of 0.3 mm (0.012 in), bearing in mind that the "calculated" crack width (i.e. at steel level) is somewhat less if provision is made for the "tapering" of the crack away from the surface.

Specification and Control of Concrete in CRCP

As discussed in the preceding section, the propagation of the design crack pattern only occurs if the critical steel ratio is exceeded. Part of the design procedure is to check that the actual steel ratio determined in the CRCP design exceeds the critical steel ratio applicable to the design concrete strength adopted for the purposes of ascertaining the pavement thickness. If the critical steel ratio is not exceeded, then large cracks are likely to open up in the CRCP, leading to the inevitable yielding of the steel.

To ensure that the critical steel ratio is actually exceeded in practice, an upper limit on concrete strength is specified in addition to the usual lower limit or characteristic strength. This practice was adopted in NSW following early Australian experiences in CRCP construction in which yielding of the steel accompanied the achievement of excessively high concrete strengths (1).

The application of an upper limit on the strength of concrete for acceptance in CRCP construction is now an accepted practice in NSW. The authors have no knowledge of the use of this practice outside Australia, even though its use has been suggested elsewhere in the literature (6).

Any attempt to adopt the concrete flexural

Table 4. Control of quality of concrete in CRCP.

Description of Strength Parameter	Clybucca				Homebush		
	No.	mean MPa	s.d. MPa	t.v. ^a %	no.	mean MPa	s.d. MPa
<u>Compressive Strength</u>							
specified 28 days range		23 to 33				30 to 40	
target	-	28	3.0	-	-	35	3.0
actual 7 days	1006	18.27	2.09	2.71	132	26.02	2.58
actual 28 days	1034	28.24	2.32	2.07	827	34.72	3.00
actual 90 days	34	32.78	1.75	1.34			
<u>Flexural Strength</u>							
specified 90 days	-	4.4	-	-	-	4.9	-
actual 28 days	45	4.01	0.35	6.70			
actual 90 days	47	4.43	0.26	5.88			

Note: 1 MPa = 145 lbf/in²

^a"within-test coefficient of variation" shown as a measure of test variability.

strength or modulus of rupture as a routine quality control tool has been discontinued, on account of the insensitivity of the test to variations in the strength of the concrete placed in the pavement. This test can be regarded only as an indicative test, accurate to 0.5 MPa (70 lbf/in²) or about 15% - insufficient for assuring that the concrete strength is controlled within the narrow bandwidth advocated for CRCP construction. Nowadays, the modulus of rupture (or flexural strength at 90 days) is employed solely as a parameter in rigid pavement design. This test, together with others such as the shrinkage test, are conducted as part of the acceptance criteria for a proposed concrete mix to check its compliance with design requirements.

Generally the routine quality control of the concrete on delivery to the site is effected by means of checking the slump, the air content and, after 28 days, the compressive strength of standard cured cylinders - each 150 mm (6 in) by 300 mm (12 in). The specified concrete compressive strength at 28 days is equivalent to the design modulus of rupture at 90 days, as determined for the particular combination of mix ingredients in the concrete. A tolerance of ± 5 MPa (700 lbf/in²), usually about 15%, is permissible.

The specified bandwidth of 10 MPa (1450 lbf/in²) for the concrete compressive strength has been found to be achievable in practice, based upon a 90% probability and a permissible standard deviation of 3 MPa (450 lbf/in²) applicable over the duration of any particular project. A summary of the results achieved in two completed projects with a specified 10 MPa (1450 lbf/in²) bandwidth - Clybucca and Homebush Stage I (without fly ash) - is shown in Table 4.

The other characteristic requiring care in specification and control of the mix is the drying shrinkage of the concrete. Although Australia is not particularly blessed with low shrinkage cements, the DMR imposes a limitation upon the laboratory shrinkage of the concrete mix after three weeks air drying of 450 microstrain. This figure is commensurate with the values stated previously for design purposes, the latter values being derived as follows:

Maximum measured laboratory shrinkage (ϵ) after three weeks air drying at 50% RH^m and 23°C (73°F) 450 $\mu\epsilon$

Shape factor for laboratory specimen with dimensions 75 mm (3 in) x 75 mm (3 in) x 285 (11 $\frac{1}{4}$ in) 1.4

Age factor for laboratory specimen tested after three weeks air drying 0.6

Basic shrinkage (ϵ_b) = $\frac{\epsilon_m}{1.4 \times 0.6} = \frac{450}{1.4 \times 0.6} = 535 \mu\epsilon$

"Theoretical thickness" of slab with cross-section of 200 mm (8 in) x 3.7 m (12 ft) = $\frac{2 \times 3700 \times 200}{200 + 3700 + 200} = 360$ mm (14 in)

Correction factor for "theoretical thickness" 0.6

Restraint factor for 0.6% longitudinal reinforcement 0.9

Field shrinkage (ϵ_f) in dry air (= $\epsilon_b \times 0.6 \times 0.9$) 300 $\mu\epsilon$

Shrinkage "generally in open air" (= $\frac{2}{3} \epsilon_f$) 200 $\mu\epsilon$

All the foregoing factors are in accordance with Australian Standards (11).

Concluding Remarks

Tight but realistic specification requirements, coupled with strict quality control and supervision of the concrete mix production are regarded as the essence of successful CRCP construction and an integral consideration in its design and planning.

The simplified design procedure for CRCP based upon the environmental cracking approach is now

being applied successfully in Australia. Observation of the behaviour of CRCP in service confirms the adequacy of this design procedure and the assumptions made therein.

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