Plain Concrete Highway Pavements in New South Wales Australia 1983 – 2015

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Abstract

Concrete highway pavements have been constructed in the State of New South Wales (NSW) Australia since the mid 1920s. Prior to 1983 the principal pavement type was jointed reinforced with a few sections of CRCP. In 1983 plain concrete was introduced for highway construction. Within an overall inventory of more than 850km of highway pavement completed and under construction since around 1980, approximately 650km are plain concrete, the remainder being CRCP. One feature of these plain concrete pavements is the inclusion of undoweled transverse contraction joints. This paper sets out the factors leading to the introduction of this type of pavement including undoweled joints, and the evolution of jointing, thickness, subbase, shoulders and overall carriageway cross sections since 1983. The importance of designing the pavement as a system, not just a series of components, and placing as much importance on what is under the slab, as for the slab itself, is emphasised. International source technologies from which NSW practice has drawn are referenced. Contemporary design traffic loads, thickness design procedures, model drawings and construction specifications, construction practice and an overview of maintenance experience are referenced.

Metric Units and Strength Measurement

This paper uses metric units in the SI system introduced in the mid-1970s. In this paper imperial units of measurement in referenced documents dated prior to that time have been converted to metric units.

Where mentioned in this paper, concrete compressive strengths are based on crushing cylinders or cores. To compare with crushing by cubes, the cylinder strengths in this paper would be increased by about 15-20%.

Introduction

The construction of concrete pavements in urban and rural highways in New South Wales (NSW) commenced in 1925 with the establishment of the Main Roads Board under the Main Roads Act of NSW. That agency later became the Department of Main Roads, the Roads and Traffic Authority NSW and currently Roads and Maritime Services NSW. By the early 1950s an inventory of about 345 km of concrete highway; 180km in the Sydney region (Figure1)and 165 km in rural locations had been constructed (1). These were pavements with their original

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concrete surface. Although not known accurately, an additional approximate 30% had been constructed but had received an asphalt resurfacing, primarily arising from surface wear and skid resistance issues.

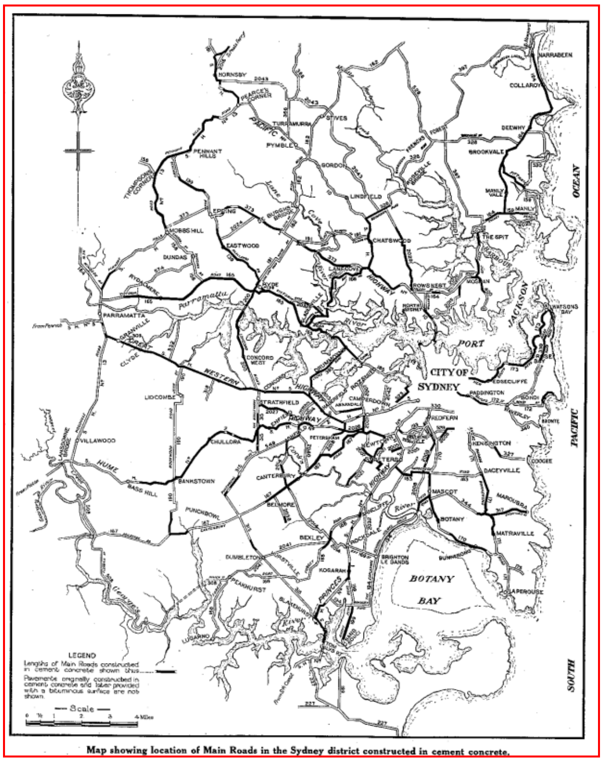

Figure 1 Sydney concrete roads in 1953 retaining original concrete surface

These pavements were of the jointed reinforced (JRCP) type. In 1960 standard details of construction were contained in a single drawing (2) Typical slab lengths were about 14m with doweled transverse expansion joints; 20mm dowels, 600mm long at 300mm centres. The thickness was standard at 200mm, but some earlier pavements were only 175mm thick. Steel reinforcement was in the form of welded wire fabric (“mesh”).
The subbase under the slab was a nominal thickness of granular unbound material.

With the exception of one urban expressway in Sydney (3,4) constructed in the mid-1960s, there was little concrete highway construction in NSW from the mid 1950s until the mid 1970s.

Triggered in part by the 1973 Middle East oil crisis, a renewed interest in NSW saw the introduction of a new era of concrete highway pavement construction. The initial three projects were CRCP. However, after evaluating practice in other countries and the growing use of mechanised construction including slipform paving, plain concrete pavement (PCP) was introduced in 1983. This was also assisted by a Federal Government funding program leading to 200 years of European settlement in Australia, the Australian Bicentennial Road Development fund. In 2015 terms this fund provided $8 billion (AUD) from which individual States submitted proposals. Two features of this program were that comparatively long lengths of construction were now possible and all work had to be constructed by the private sector. This provided incentive for the private sector to invest in concrete production and paving plant that was not previously possible.

Since 1983 approximately 850km of concrete highway pavement have been completed or are committed to construction in NSW. Of this approximately 85% has been in PCP, the remainder is CRCP. Many details are common such as subgrade design, shoulder design, and thickness design for both PCP and CRCP. This paper deals with the PCP inventory.

Pavement Nomenclature

The pavement layer nomenclature used in NSW (Figure 2) is used in this paper

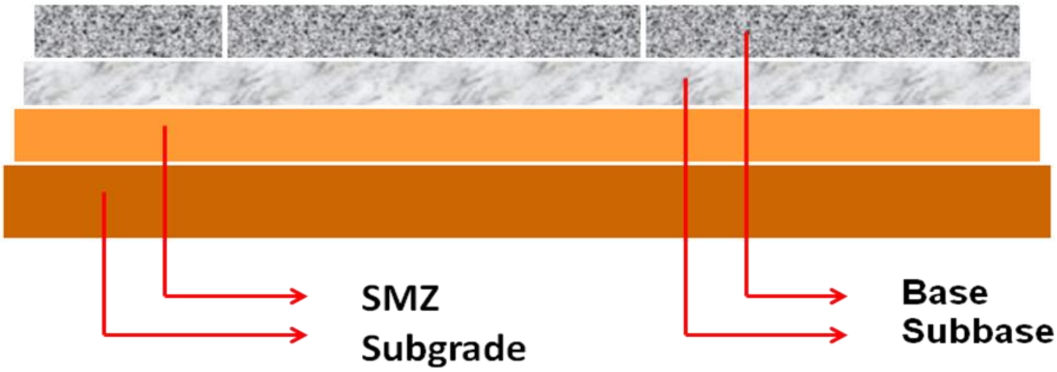

Figure 2 . NSW concrete pavement nomenclature

Factors Leading to PCP Construction

A number of factors combined leading to the introduction of PCP;

- Investigations of overseas practice showed that the large majority of concrete highway pavements internationally were plain concrete.
- Compared with JRCP, plain concrete was simpler and less expensive to build and simpler to replace slabs if and when required
- With modern high performance joint sealants, narrow joint grooves were needed which would overcome much of the criticism of ride quality associated with earlier wide grooves in expansion joints.
- It was still common for concrete pavements to be designed for 40-year traffic design lives compared with 20-year designs for flexible pavements with no or little consideration of future maintenance costs. If concrete was to be considered for extensive usage, construction costs had to be minimised.

Doweled or Undoweled Contraction Joints.

During the late 1960s and 1970s in older JRCP urban pavements that had been constructed with concrete in centre lanes with flexible pavements in kerb lanes, those kerb lanes were progressively reconstructed in concrete. It was logical to match the adjacent JRCP joint layout. This required doweled transverse contraction joints. With a lapse of major new construction for many years, field practice saw many occasions where dowel alignment was inaccurate and not of good quality. So the question arose as to what options might be available that did not require the use of dowels.

The use of dowels would require additional construction plant and related costs for side concrete discharge and subsequent placement as supply trucks could not back up directly in front of the paver.

The majority of world practice saw the inclusion of doweled joints in plain concrete pavements. Investigations into plain concrete construction in countries with climatic similarity to much of the eastern coastal region of NSW revealed two major sources, around 1980, where PCP was being successfully used without doweled joints. These were California in USA (5) and South Africa.

Although overnight winter temperatures in some areas may be a few degrees below freezing, NSW is free from frost penetration, frost heave or freeze thaw conditions.

An extensive study in South Africa of construction taking place and also major documentation produced by the late legendary Louis Marias of the PCI in South Africa (6) and South African Division of National Roads (7) where Californian practice, with modification, had been followed provided contemporary reassurance in the use of undoweled joints.

The Pavement System

In the introduction of undoweled PCP in NSW, the South African experience indicated that reliance on increasing slab thickness alone was insufficient. From the outset two key factors emerged;

- A pavement system, not just the main slab, had to be considered. This included very short slabs, a heavily bound subbase, structural concrete shoulders and high performance joint sealants
- As much attention had to be paid as to what is under the slab as to the slab itself. Merely increasing slab thickness may not compensate for deficiencies in the subgrade and subbase.

These two cornerstones of design philosophy have remained.

Non-Erodible Subbases

One of the key design criteria for joint load transfer in concrete pavements was presented at the 2nd International Conference on Concrete Pavements (8). The major on the issues of drainage, load transfer and subbase erodibility were presented. This reinforced the South African practice of a cement bound subbase in PCP without dowels and . One key recommendation was that in order to provide subbase erosion resistance, particularly where large volumes of heavy trucks were expected, a minimum cement content of 5% by weight was required in subbase material

One of the major causes of pavement distress in older NSW JRCP had been major joint faulting often accompanied by slab fracture arising from subgrade/subbase pumping where unbound subbases had been standard.

Around 1980 a number of non-erodible subbase solutions were considered. These included dense graded asphaltic concrete, cement stabilised material which on large projects would be plant mixed and paver laid, or a low early strength or “lean“ concrete. The asphalt option was not used oncost grounds. The lean concrete option was adopted and has remained standard since the introduction of PCP.

The lean concrete option has the following characteristics both from a design and construction perspective;

- It is non erodible
- The same concrete production and paving plant can be used for it and the overlying base. For a given project this effectively doubles the plant usage and provides economy of scale for plant establishment and usage.
- Lean concrete poured one day can be worked on the following day. Wet weather construction delays can be greatly minimised.

Lean Concrete (LMC) Properties.
A concrete having a cement content of only 5% or about 115kg per m3 would most likely be unworkable and unsuited to slipform paving. After early industry consultation, a mix containing a total cementitious content of 250kg per m3 with 90kg cement and 160kg of flash from black coal fired powered power stations was introduced.

A decision was taken by the road agency to place this material as mass concrete with no internal joints or reinforcement. Cracking will occur. The preferred crack pattern is a larger number of fine cracks rather than fewer wider cracks. The preferred crack spacing is in the approximate range 3-15m.

The road agency cracking controls are a combination of; low early strength in the range 5-7 MPa at 7 days during which most of the dryng shrinkage will occur in ten field, drying shrinkage after 21 days air drying in laboratory conditions of about 450 microstrain. The full agency details are contained in its Specification (9). The required 28-day compressive strength is 15 MPa. An indicative mix design for LMC is given in Appendix 1

With specified 7-day curing and the amount of flyash in the LMC an extended strength gain is yielded. At ages of about 8-9 months after placement at which time very little of the design service traffic will have used the pavement, cores taken from LMC have shown compressive strengths around 25MPa. In terms of thickness design discussed below, this is not taken into account and is accepted as a bonus. This strength also contributes, on an unquantified basis to support for undoweled joints.

Over the first few years of PCP construction, LMC thicknesses varied in the range 100-150mm. This was later standardised at 150mm. It is now considered that there is no structural or economic benefit in increasing this thickness with a view to decreasing base thickness.

With a 10m wide carriageway (2 lanes and a concrete shoulder) common industry practice on major projects is to slipform the LMC full width. On one early project where the LMC was constructed in partial widths and a plastic tape inserter was used for a longitudinal joint (this was discontinued in the mid-198s and replaced by sawing) but did not work properly a longitudinal crack occurred in the base over several kilometres arising from reflection from the LMC longitudinal construction joint. This crack was effectively repaired by cross-stitching. However, road agency drawings now only permit longitudinal construction joints in LMC very close to the line of intended longitudinal joints in the base (Figure 3)

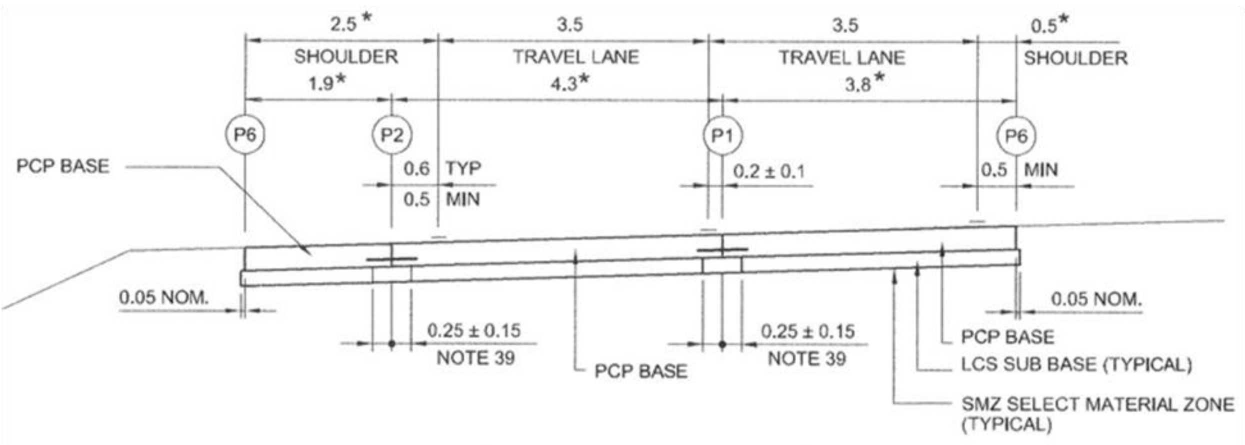

Figure 3. Locations of longitudinal joints in partial width LMC paving to avoid reflection cracking shown

A design intent, both for PCP and CRCP, is that the subbase is debonded from the base to allow base joints/cracks to operate without restraint. This is achieved by two means; the LMC is paved with a steel float finish only and no other texturing, a thin debonding layer comprising a wax emulsion curing membrane and a sprayed bitumen seal with 7mm aggregate (called a primer seal). There is lateral friction but no vertical debonding (Figures 4,5).

Figure 4. 10m wide slipform paving of LMC subbase with steel float finish

Figure 5. Wax emulsion curing and bitumen seal bond breaker between subbase and base

Slab Detailing for Undoweled Contraction Joints

Load transfer and Joint Opening .

During the 19606 and 1970s, extensive research into the effectiveness of load transfer by aggregate interlock (10,11,12) showed that under laboratory conditions a high level of load transfer effectiveness can be achieved for concrete slab thicknesses in the range 175-225mm on cement treated subbases under applied loads of 2.25t provide that joint openings did not exceed about 0.9mm. The standard single wheel load in NSW is 2.3 t. Monitoring of roads in 7 US States (10) showed that given levels of load transfer effectiveness could be achieved at somewhat greater openings than indicated by laboratory research. However the laboratory values were highlighted as the principal conclusions of this research. This led to a general criterion found in many documents that aggregate interlock can be effectively managed provided that joint openings do not exceed about 1mm.

In 1982/83 a newly constructed 3.5km, 2-lane concrete highway pavement in Queensland (13) near northern NSW became available for research to measure and analyse joint movements. The pavement was 200mm of concrete on a 10mm cement treated base. Doweled transverse contraction joint were at 5.0m intervals. The results of this research were later published at the 1988 Australian Road Research Board Conference (14).

Four sets of four consecutive joints were instrumented (Figure 6). At each joint instruments were located at the pavement edge, mid slab width and near the centre joint, providing a total of 48 instrument locations. Measurements were taken at both early morning and at the middle of the day in the first week of each month over a 12-month period yielding about 1150 data points.

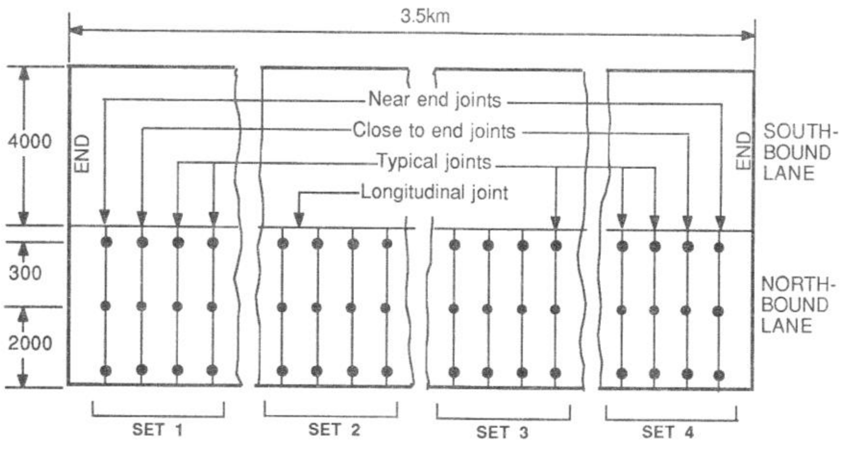

Figure 6. Locations of instrumented joints for joint opening measurement.

Changes in temperature within the concrete at a depth 50mm below the surface were made at each of the 16 joint locations. The temperature range over the full year cycle was 30C degrees between 10^0C and 40^0C

Joint openings were measured at the surface. Arising from friction at the slab/subbase interface, and some slab curling, joint opening measured at the surface is likely to be more than that at the subbase interface and the 'mean' over the slab depth. However, the surface measurements were taken as the basis for conclusions for conservatism.

As would be expected joint openings were greater in winter months than in summer months. Over the full 12 months the 95% characteristic maximum joint opening was 1.23mm. Leaving aside the main two winter months the maximum value for the remaining 10 months was 1.14mm. (Figure 7)

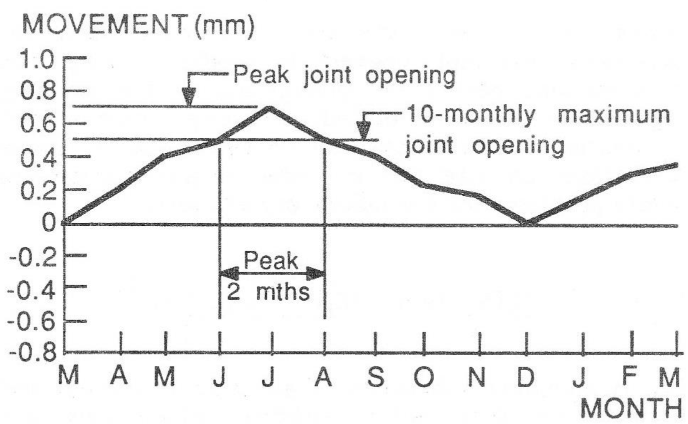

Figure 7. Peak and 10-monthly joint openings

Assuming that over a slab length range of approximately 4-5m, maximum joint opening could be linearly related to slab length. On this basis the maximum joint opening would be reduced to about 1mm for slab lengths in the range 4.1 to 4.4m.

Slab Lengths Adopted in NSW

In 1980 the standard slab length for undoweled joints in South Africa was 4.5m (7). In California at that time, a repeated sequence of 4.0-5.8-5.5-3.7m was used with undoweled joints (15), an average length of 4.8m.

Initially the Dept Main Roads in NSW adopted a repeated sequence of 3.7-4.3-4.6-4.0m, an average of 4.15m.This was prior to the above research results being available.

In 1988 in a major report prepared for FHWA (16) a survey of 22 US States that used or permitted undoweled joints showed the shortest, ie most conservative, length of 14 ft which converts to 4.2m. This adopted in NSW and has remained standard since the late 1980s. This length falls within the range suggested by the above field research for effective load transfer.

Skewed Joints.

Consistent with South African and Californian practice, transverse contraction joints in NSW were initially skewed at 1:6 with ten driver side wheel crossing the joint first. Some corner cracking occurred at the longitudinal joint between the slow lane and ten concrete shoulder.

Although not significant as a percentage of slabs, the frequency nonetheless warranted investigation.

An initial opinion was that the cracking may have been a result of late joint sawing as sawn joints were still in their infancy and experience was still being obtained. However, this was discounted on the basis that if sawing was consistently late, a higher frequency of cracks and numerous consecutive cracked slabs could be expected.

Finite element modelling later published in 1996 (17)demonstrated that curling stresses at a 1:6 acute angle at a slab corner could be significantly reduced if the skew was reduced to 1:10. This led to a reduction in the skew which to 1:10 in the mid-1980s and which has been standardised since. Except where occasional significantly late sawing was identified, the earlier corner cracking issue did not reappear.

In one urban 21km tollroad/motorway in Sydney, M2 Hills Motorway, constructed in 1996/97 joints were not skewed. This has been the only known exception to the 1:10 skew.

Concrete Shoulder and Widened Truck Lane

Since the introduction of PCP with undoweled joints in 1983, concrete shoulders have been standard. They have proven to be a very sound investment. Until recently when high output concrete mixing plant was introduced, typical construction of a 2-lane carriageway saw the two lanes being paved with a sawn and tied longitudinal joint and the shoulder added with a tied longitudinal construction joint.

The marked shoulder width is typically 2.5m wide. The shoulder is constructed to the same thickness and uses the same quality concrete as per the trafficked lanes. In addition to a fundamental decision to incorporate concrete shoulders commencing in 1983, later work by Zollinger (18) confirmed the amount of additional integral pavement widening required to reduce edge stresses to that of an interior load position (Figure 8)

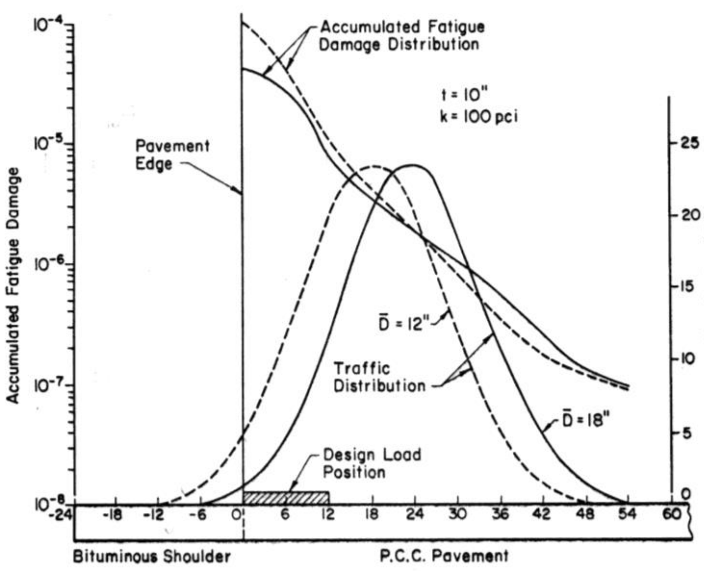

Figure 8. Fatigue damage and wheel load position (Zollinger)

The reasons for adopting concrete shoulders included;

- Edge load stresses are reduced by about 30%. If concrete shoulders were not included, base thickness would need to be increased by about 40mm.
- The concrete shoulder eliminates what would otherwise be a very long longitudinal joint between the concrete lanes and a flexible pavement shoulder. Water ingress and higher shoulder maintenance could be expected. This was proven from an earlier 1975 CRCP project (19) where a concrete shoulder was not used and regular maintenance has been required since that time in an approximate 0.5m strip in the shoulder next to the concrete lane (Figure 9).

Figure 9. Concrete pavement without concrete shoulder

- Shoulder maintenance has been effectively eliminated with the inclusion of the concrete shoulder.
- Adopting the same thickness as for the main pavement provides the benefit of a uniform concrete pavement thickness across the whole carriageway. Tapered thickness shoulders have not been used.
- With high efficiency twin shaft concrete mixers giving hourly production of about $200m^3$, the option of full carriageway width paving, ie 10m, has become available as shown later in this paper.

In addition to concrete shoulders, and following practice in Europe, particularly Germany and also some USA States, the widened truck lane with longitudinal joints outside the marked truck lane was introduced in the late 1980s to further remove longitudinal joints from typical truck wheel paths (Figure 10). This has proved to be a simple construction detail with no added cost to construction.

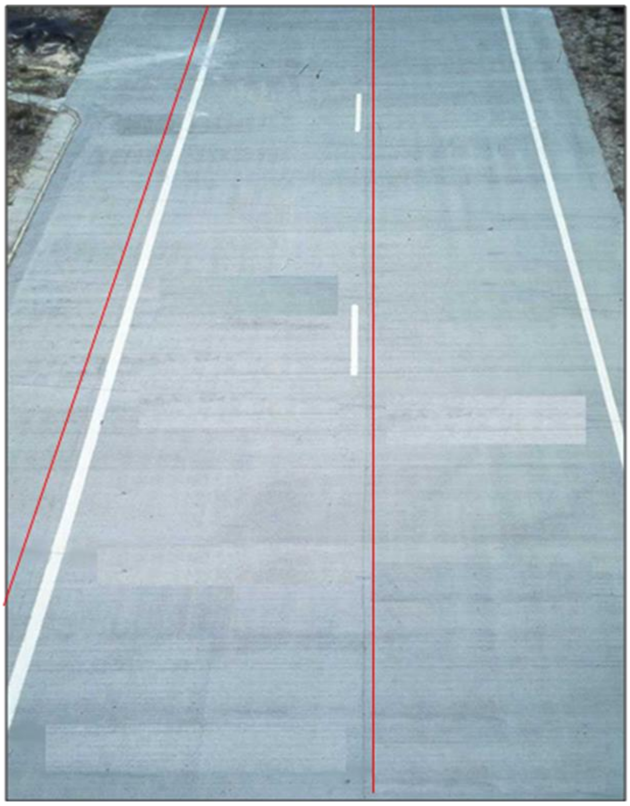

Figure 10 Two-lane carriageway with concrete shoulder and locations of longitudinal joints for widened truck lane shown.

Subgrade and Subbase Support

The principal parameter used for support under the base is the California Bearing Ratio (CBR). As most NSW subgrade soils have low support a soaked CBR is typically used. In thickness design an increased CBR value is assigned to the subgrade CBR with the inclusion of the LMC subbase. This is then called the effective design CBR. Until around 1990 the LMC was placed directly on the subgrade. Since 1990 and for all candidate pavements both flexible and concrete on heavy duty roads in NSW defined as those designed to carry more than 10^7 commercial vehicles, which effectively covers all major roads, an improved subgrade was introduced. The NSW term for this is a Select Material Zone (SMZ). This is 300mm thick extending the full carriageway width and constructed in two separate 150mm compacted layers. There is some variation in practice but typically the lower layer is required to have a CBR of 15% and the upper layer CBR 30%. (Figure 11)

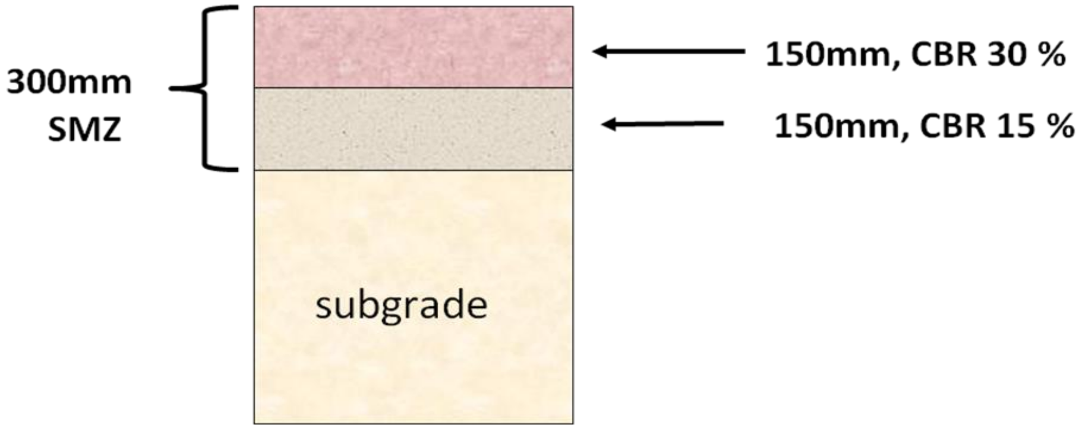

Figure 11. Typical 300mm thick Select Material Zone (SMZ)

Depending on material available from earthworks the existing material may be suitable or may need to be stabilised. In some cases imported material is required. The intent of the SMZ is not to reduce pavement thickness but to provide an improved platform for pavement construction and to enhance long term performance. As a means of limiting water entry into the SMZ prior to pavement construction, a full carriageway sprayed bitumen seal is commonly applied, similar to that used in the subbase/base debonding layer (Figure 12).

Figure 12. Trucks hauling subbase LMC on sealed SMZ

For thickness design there are differing professional opinions as to whether any improvement is assigned to foundation CBR with the SMZ. Austroads (20) provides a model that may be used to determine equivalent subgrade design strength in a multi layered subgrade including a SMZ. An underlying subgrade CBR of 2% could be increased to about 5% with the SMZ.

Drainage

Pavement drainage is provided in two forms;

- Edge drains to drain the subbase/base interface even though a non erodible LMC subabse is used
- Subsurface drainage alongside the pavment to restrict upwards moisture movement into the SMZ/pavement region

Until recently they were constructed as separate drains. This has now been amended to a combined drain. This includes a vertical 'trench' containing no-fines concrete surrounded by a geofabric. An example is shown in (Fig 13). Full details are available in model drawings (21)

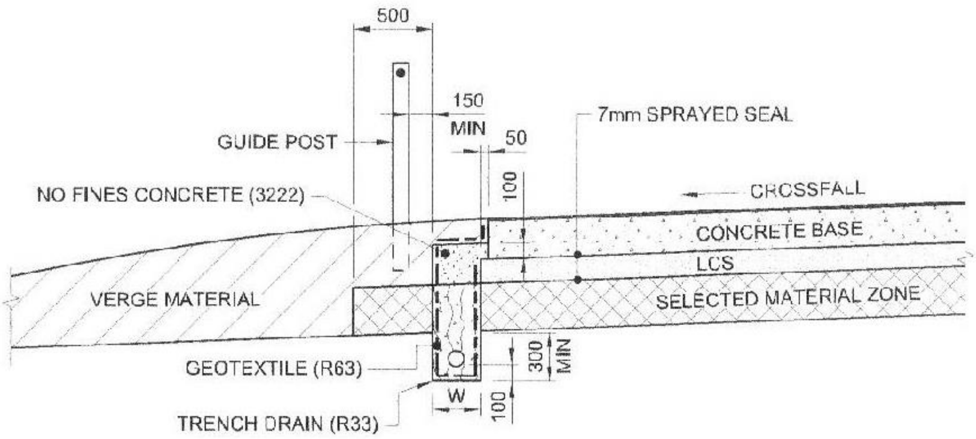

Figure 13. Example of combined subbase/base edge drain and subsoil drain. Perforated subsoil pipe in trench filled with no-fines concrete and trench 'wrapped' in geofabric

Joint Sealants

From 1983 until 1985 transverse contraction joints were sealed using preformed neoprene compression seals adopted from South African practice. In the main they worked well. However, in some cases poor installation methods that included stretching saw early deterioration in the neoprene material.

Following investigations into experience in Georgia (22) and an extended visit by the author to Georgia DOT (Mr Wouter Gulden) in 1985, the neoprene was replaced by field moulded silicone. From that time longitudinal joints as well as transverse joints have been sealed.

Material suppliers installation requirements have been followed and the typical life of sealants when properly installed is exceeding 20 years.

Concrete Strength, Shrinkage and Compaction

For thickness design, flexural strength is used. For construction, both flexural and compressive strengths are tested. A typical 28-day flexural strength 4.5MPa is used for thickness design.

For construction which is now carried under Quality Assurance management, concrete strength is controlled as follows;

- Laboratory trials are conducted for both flexural and compressive strength
- When satisfactory results are obtained the mixes are registered as “nominated” mixes
- With other inclusions in the contractor’s Quality Plan eg construction pant and work methods), a short trial pavement is constructed in which the contractor demonstrates and provides assurance to the client that the pavement meets the Specification requirements.
- When the trial pavement is successful, production paving commences.
- For slipform paving concrete is tested every 50 m^3 and where fixed form paving is required every 30 m^3

Most production concrete testing is by compressive strength (cylinders) with companion flexural strength testing. In outline the laboratory trials must achieve at least 40MPa, production paving must achieve 36MPa. Payment deductions apply below 36MPa leading to rejection below 32 MPa. An indicative mix design for base concrete is in Appendix 1

Drying shrinkage is specified and has to be met at laboratory trial stage. There is some variation depending on cementitious material used, but typically using standard concrete bars a 21-day air dried shrinkage limit of 450 microstrain is required.

Since the mid 1990s concrete compaction has been rigorously specified and tested. This has been major initiative and program developed by Mr Geoff Ayton Honorary Member ISCP (now retired from NSW RMS). Special reference cylinders are prepared using a test method (Ref 23) designed to simulate a slipform paver operation. A core is taken from pavement sub-lots and the density of the core compared with the reference cylinder. Relative density/compaction of at least 98% is required for acceptance. Below 97% the whole sub-lot has to be removed and replaced at the contractor’s cost. For slipform paving compaction is tested every 50m^3 of concrete placed.

The full details of the concrete Specification including curing are available in (24)

Thickness

From the 1970s until 1992, the thickness design method in use was the 1966 PCA Method (25). Using standard axle loads for various axle types and Load Safety Factors, sometimes with addition of construction tolerance allowances, design thickness were in the range 200-230mm. They were not always constructed to this thickness.

In 1989 Austroads formed a working group under its Pavement Research Group (APRG) including the author to evaluate existing international design procedures with a view to updating the Australian national pavement design guide. After evaluating design procedures in eight countries, Austroads selected the 1984 PCA procedure (26). This was incorporated into the 1992 Austroads pavement design guide (27). One feature of this procedure is that it allows for specific consideration of the inclusion/deletion of concrete shoulders and doweled /undoweled joints. The Austroads guide has been updated on a number of occasion since 1992, the current edition being dated 2012. RMS has developed a supplement to the Austroads Guide (28) However the core design methodology has remained unchanged. The 1989 evaluation was later presented in a paper by the author at a Conference of the Australian Concrete Institutue (29). A commentary and implications for designers on the use of the 1992 procedure was presented at an APRG Workshop at the 1992 Australian Research Board Conference (30).

AASHO/AASHTO/NCHRPdesign procedures have not been used in Australia for flexible or concrete pavements. Many of the design inputs are not used in Australia and the pavement types used in the AASHO Road Test differ from pavements built in Australia.

Of relevance to concrete pavement thickness design, the view generally held in Australia is that concrete pavement thickness design cannot be directly related to concepts of axle load equivalency. It is necessary to have a good estimate of actual expected axle loads. Design traffic loads are expressed as a cumulative number of heavy vehicle axle groups (HVAG) (31) combined with a traffic load distribution.

With the emergence of weigh-in-motion measuring of axle loads and repetitions dating from the mid 1980s, traffic load distributions have allowed the development of software that can be directly input into the 1984PCA/Austroads procedures. An example of a representative TLD is in Appendix 2.

For major highways contemporary 40-year design traffic loads are in the range 1-2 x 10^8 HVAG. The author's view, supported by Austroads (20, 31) is that pavements are not designed for 40 years per se, but the traffic that is estimated over that period. The design is only as good as the estimate. There have been many instances where traffic growth has exceeded initial estimates and after about 30 years the pavement has been subject to at least double the original estimate.

For PCP with undoweled joints on low strength subgrades with SMZ, LMC subbase and structural concrete shoulders, calculated thicknesses are of the order of 240-250mm. The road agency usually adds a further 10mm for construction tolerances.

As an initiative and based on extensive experience in California (32), Geoff Ayton introduced a design direction to add a further 10mm for future diamond grinding (33). On the basis that the constructed riding quality meets Specification requirements, with a conforming IRI value of 1.56m/km (24) and if retexturing is necessary at approximate 17-20 year intervals to rehabilitate worn surface texture, and on each occasion approximately 5mm is removed by diamond grinding, then no new surfacing material will be required for at least 50 years after which time the original structural is retained. This is considered be a major long term sustainability measure. This additional 10m at construction is arguably the lowest cost concrete an agency is likely to get. Current industry experience is that constructed IRI values in the range 0.80-1.18 m/km are being consistently produced. As a result of the above provision for grinding, constructed bases are now 260-280mm thick, approximately 60mm thicker than prior to 1990.

Carriageway Configuration

Consideration of the pavement as a system, not a discrete slab on some support is outlined earlier in this paper. This extends to the whole carriageway. A typical 2-lane carriageway is shown in Figure 14. Drainage is not shown. In addition to short slabs, concrete shoulders, widened truck lane, SMZ, heavily bound ie LMC subbase properly sealed joints, and whether in cut or fill formations layers in the foundation and the pavement are carried across the whole carriageway at uniform thickness.

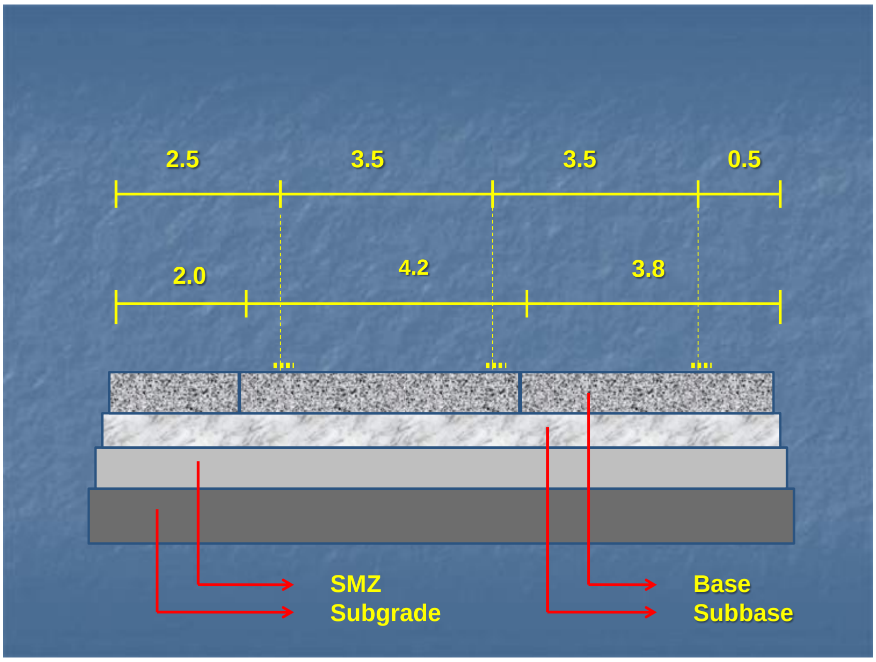

Figure 14 Typical two-lane carriageway cross section showing concrete shoulder, lanemarking and longitudinal joint locations. Drainage not shown

Darter (34) drew attention to the design shortcomings of pavements built in “bath tubs”, with erodible subbases and no or inadequate drainage. Whatever shortcomings may have occurred, particularly in re-1990 pavements in NSW, these design shortcomings have not been used in NSW.

Performance

Since 1983 within an inventory of about 600km of completed PCP over a transport corridor of about 1,400km, varying terrain, and with changing design details, performance has been varied. The following can only be considered as an overview.

Three general aspects apply;

- After about 30 years the earlier mid 1980s pavements, by general consensus, have received at least double the original estimated 40-year traffic loading.
- The 20 year observed condition of pavements constructed after 1990, with design improvements mentioned above, is generally visibly better than similar aged pavements constructed in 1983-1985.
- Since 1983 there has been a maturing of experience within the construction industry. The companies that are currently active now have more than 25 years experience. RMS Specifications and model drawings have been updated and strengthened over this period.

The most obvious question in PCP is joint faulting. Within the completed inventory, and for dual carriageway pavements there is a completed inventory of about 280,000 undoweled transverse contraction joints. There has been some limited joint stepping not exceeding about 5mm. This has occurred primarily in pavements constructed prior to 1990 without a SMZ and also where there have been some long term subgrade issues (Figure 15). Many sections of pavement have no joint faulting. An accurate survey of the number of faulted joints is not available. However, the total number of joints showing small faulting is believed to be of the order of no more than a “few hundred”. On average this is about one per thousand or no more than one per 2.5km of carriageway with almost all occurring in pre-1990 construction. So whereas they exist, they are not assessed as a network performance problem.

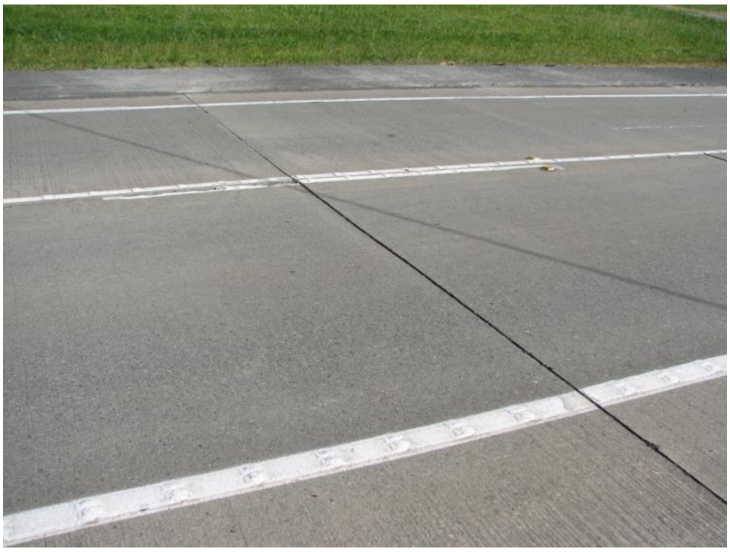

Figure 15. Joint faulting less than 5mm, 25 year old pavement constructed without SMZ on a flood plain with water table at subgrade level

In the earlier pavements now 30 years old , without SMZ and constructed at thicknesses as low as 185mm, slabs are currently being replaced in some regions in the approximate range 1-2% of slabs per year. The reasons are not the same in each region but generally arise from one or more of a combination of lower concrete quality than in contemporary construction, thickness which in retrospect was insufficient, problems with subgrade preparation, excessive joint skew, slabs in the fast lane in some cases being too large for PCP at the earlier constructed thicknesses, and inadequate induction of weakened plane longitudinal joints in multiple lane paving using plastic tape inserts..

Based on Californian experience over 20 years (35) slab replacement technology has been in use for about 6 years with approximately 200 slabs having being removed and replaced with re-opening to traffic within one standard night time shift.

However on highways with some sections in concrete and others in flexible pavements, maintenance costs for concrete are lower than for flexible pavements. Over the past approximate 20 years where project delivery has, in many instances, been design/construct/maintain (for periods up to 15 years after construction), competitive tendering has seen concrete selected by the private sector construction industry in almost all projects.

A principal cause of early life distress and subsequent repair, particularly prior to the introduction of compaction specification and testing specifications, was localised concrete rupture arising from inadequate concrete compaction particularly at and near end-of-day and start-of–day paving, not design issues. This has been substantially reduced over the past approximate 15 years.

With the evolution of design over more than 30 years, designs are considered to have reached a mature and effective position. The main contemporary emphasis for future good performance is now on consistent construction quality. As a contribution to enhancing construction quality a joint industry/road agency training course specifically aimed at construction paving crews has been conducted over the past 10 years (Ref). The course is now conducted by ASCP under a memorandum of understanding with RMS. Some 120 courses with over 2,700 participants.

Current Developments

In the past few years one significant development has been the introduction of full carriageway width paving ie 10m on a couple of projects. At a typical paving speed of about 1m/min and a slab thickness of about 270-280mm, consistent hourly concrete production of at least 170 m^3 is needed to maintain continuous paving without stop/start. Earlier split-drum and tilt-drum mixers were not capable of this requiring a mixing time of 90 seconds for a $6m^3$ batch. With the introduction of European twin-shaft mixers with improved mixing efficiency, mixing times have been reduced to 60 seconds, allowing concrete production of about 200m3 per hour. This has opened the option of full width paving.

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Based on work by Okamoto and FHWA (36, 37) on timing of contraction joint sawing and analysing internal restraint stresses across pavement widths, maximum stress locations that could lead to early longitudinal cracking can be calculated (Figure 16) leading to preferred induced joint locations (38)

Based on these calculations one section of highway has been constructed full width with sawn weakened plane longitudinal joints in locations shown in Figure 17 Confirmation of crack induction and initial pavement movement in these locations was reported by Egan (39) in two ways;

- Specialised micro photography vertically into the sawcuts showed cracks induced at the base of the sawcuts
- The installation of strain gauges at the base /subbase interface beneath saw cut locations showed some lateral movement, similar gauges at locations between sawcuts showed no lateral movement.

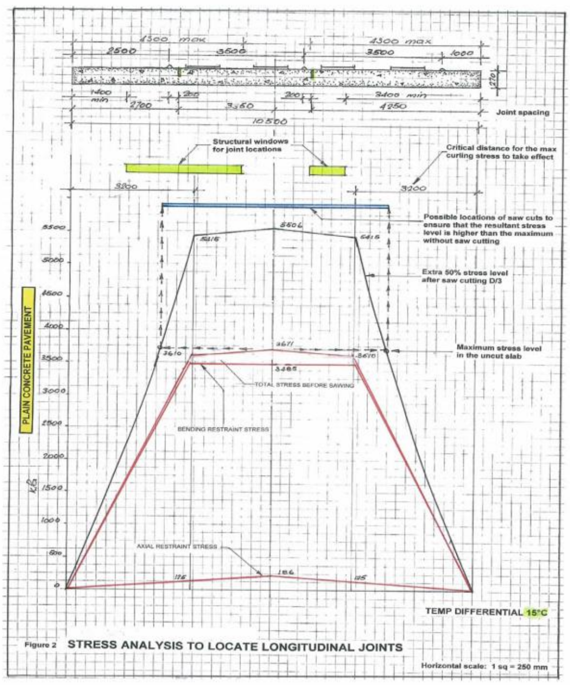

Figure 16. Stress analysis to locate longitudinal joints – full width paving (Tinni Ref 37)

Figure 17 . Full width highway PCP construction with sawn longitudinal joints (see also Fig15) shown. Prior to lane marking.

Conclusions

1. This paper has traced a 32 year development in PCP pavements in NSW against a background of no prior experience before 1983. Initial designs were based on relevant international experience at the end of the 1970s. They have been progressively updated and improved in line with both local experience and international practice.

2. A key feature of NSW PCP has been the use of undoweled transverse contraction joints. This paper has emphasised that this PCP needs to be designed as a system, not simply discrete elements. If any part of the system was to be omitted, performance is likely to be compromised.

3. The system includes two major components;

- Very short slabs, heavily bound non-erodible subbases, structural concrete shoulders, properly sealed joints, an improved subgrade in the form of a select material zone, edge and sub-surface drainage.

- All layers from the subgrade to the surface are constructed with uniform thickness and materials across the full carriageway width.

4. Some minor incidence of small joint faulting has occurred in the older pavements constructed prior to 1990 with superseded design details and where initial estimated traffic loads have been substantially exceeded, but when assessed against the overall inventory it is insignificant and does not represent a network problem suggesting a fundamental load transfer issue with the undoweled joints.

5. Design standards have reached a mature stage of development. The major industry emphasis is now on a consistently high quality of construction.

6. Construction costs are directly competitive with and often less than asphalt pavements designed for the same traffic loads and long term maintenance has been less than asphalt highway pavements.

7. Where slab replacement is required current practice sees removal, replacement and re-opening to traffic within one night time working shift.

Dedication

This paper is dedicated to the memory of the late Louis Marais, Honorary Member ISCP. Louis was the main tutor and mentor of the author in the pioneering days of the late 1970s and early 1980s in Australia. This continued through the 1980s and early 1990s. Without his wise counsel, guidance and friendship PCP in NSW may not have commenced or progressed to its current status. This paper would not have eventuated. As Bryan Perrie said in his eulogy to Louis "he was a giant in the field of concrete pavements - a legend in his lifetime."

Appendixes

1. **Indicative concrete mixes for NSW lean concrete subbase and concrete base construction**

Table 1 Lean Mix Concrete Subbase

20mm aggregate mixes, admixtures such as air entrainment not shown

Constituent	Mass (kg per m^3)
20mm aggregate	710 - 720
10mm aggregate	350 - 360
Coarse sand	280 - 300
Fine sand	510 - 540
Cement	90 -110
Flyash	140 - 160
Water	100-120 (litres) (*)

(*) varies according to slump requirement

Table 2 Concrete Base

20mm aggregate mixes, admixtures not shown

Constituent	Mass (kg/m^3)
20mm aggregate	700 - 720
10mm aggregate	330 - 350
Coarse sand	270 - 290
Fine sand	500 - 530
Cement	290 - 300
Flyash	50 - 60
Water	100 -130 (litres) (*)

(*) varies according to slump requirement

2. Presumptive Austroads Traffic Load Distribution (TLD) (Ref 20)

Presumptive Austroads TLD

Axle group load (kN)	**Axle group type**					
	SAST %	**SADT %**	**TAST %**	**TADT %**	**TRDT %**	**QADT %**
10	0.5034	3.031	0.0082	0.3815	0.0127	0
20	6.738	4.968	0.0844	0.5656	0.1246	0
30	9.237	6.858	1.285	2.364	1.878	0
40	15.30	8.484	1.986	4.009	3.859	0
50	28.74	10.23	5.115	4.255	5.213	0
60	32.19	14.16	11.48	5.173	4.359	0
70	5.653	13.58	15.24	4.866	4.078	0
80	1.211	13.29	16.19	4.846	3.843	0
90	0.2496	11.17	14.59	5.297	3.79	0
100	0.0768	7.837	15.37	6.244	3.842	0
110	0.0561	4.13	11.13	9.314	4.116	0
120	0.0312	2.002	6.396	9.173	3.574	0
130	0.0139	0.1787	0.6323	10.55	6.69	0
140	0	0.0701	0.2245	12.92	7.893	0
150	0	0.0117	0.1028	10.35	8.357	0
160	0	0	0.0911	3.767	8.126	0
170	0	0	0.0533	2.566	7.379	0
180	0	0	0.0214	1.598	6.076	0
190	0	0	0	0.8662	4.981	0
200	0	0	0	0.4341	4.424	0
210	0	0	0	0.2348	2.830	0
220	0	0	0	0.1346	1.928	0
230	0	0	0	0.0613	1.138	0
240	0	0	0	0.0279	0.6311	0
250	0	0	0	0	0.3486	0
260	0	0	0	0	0.2209	0
270	0	0	0	0	0.1293	0
280	0	0	0	0	0.0815	0
290	0	0	0	0	0.0441	0
300	0	0	0	0	0.0253	0
310	0	0	0	0	0.0079	0
320	0	0	0	0	0	0
330	0	0	0	0	0	0
340	0	0	0	0	0	0
350	0	0	0	0	0	0
360	0	0	0	0	0	0
370	0	0	0	0	0	0
380	0	0	0	0	0	0
390	0	0	0	0	0	0
400	0	0	0	0	0	0
Total	100.00	100.00	100.00	100.00	100.00	0.00

Group proportions	0.344	0.098	0.007	0.320	0.231	0.000

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