Comments and observations

on

AS Teede Tehnokeskuse Aruanne:

''Riigimaantee nr 1 Tallinn – Narva, Iru lõigu vana betoontee omaduste väljaselgitamine ja analüüs, 2014-10''

by

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1 Introduction

I started lobbying for the technical evaluation of the old concrete pavement on Narva ''maantee'' a few years ago as it was planned for replacement. I argued that we should make sure that we have the very valuable performance information available as input for future assessments for the options for concrete pavements in Estonia. It is through the efforts of Estonian Concrete Association and also the then Chief Executive of Maanteeamet, Mr Aivo Adamson that the need for investigation came to fruition and was carried out by the Teede Tehnokeskus (TT) under the Project Manager, Mr Marek Truu.

It is his Report 2014 -10, entitled ''Riigimaantee nr 1 Tallinn-Narva, Iru lõigu vana betoontee omaduste väljaselgitamine ja analüüs'', which I now use as my reference for the following assessment and observations.

Even though I have not been asked to comment, there is a lot of valuable information that has now been documented which actually refutes some of the commentaries of advice from our neighbours. Having done so much lobbying, I felt that it is now only proper that I also provide commentaries on the results.

2 Purposes

The purpose of my comments is to draw particular attention to aspects of the pavement as documented in the TT Report and compare these with the Estonian ‘’Maintenance Code’’ requirements, as well as Australian and the current concrete paving technology.

Where necessary, I also draw attention to the Ramboll 2012 Report: ‘’Eesti tingimustele vastava betoonkatendi projekteerimine ja selle tasuvus analüüs’’. In particular the current findings can now refute the so called expert advices by the Finns.

The TT Report and my commentaries will provide valuable guidelines for future designs of concrete pavements in Estonia, as well as prompt further investigations in still grey areas like shrinkage, heat-of-hydration and suitable compressive strength.

3 Executive summary

Some very valuable information has been gained from the quite detailed testing undertaken by Teede Tehnokeskus. This Report has classified the good, bad and not warranted information based on actual facts.

Design shortcomings:

1 Incorrect slab lengths and variable lengths up to 8 m.
2 The nominal 220 mm slab thickness is too thin based on current technology.
3 The nomination of 20 mm deep saw cut depths for inducement of transverse contraction joint cracking is virtually useless. Transverse construction joints should be cut D/4 mm or 73 mm in this case.

Construction shortcomings:

1. Incorrect saw cuts for transverse joints.
2. Saw cutting generally too late as unplanned shrinkage cracking has already occurred.
3. Unplanned cracking.
4. Varying crossfalls very variable and below standards.
5. Apparent haphazard placement of reinforcing mesh.
6. Longitudinal joints left untied.

Maintenance shortcomings:

1. Apparent non-existence of routine maintenance.
2. Regular joint sealer inspections and replacements appear non-existent.
3. Poor quality patching with asphalt.

Important findings:

1. The concrete is impervious.
2. Water penetration occurs through unmaintained joint seals.
3. No salt (water) penetration or damage.
4. Completely carbonation resistant.
5. Completely frost resistant. No signs of frost damage. Class F150.
6. Excellent skid resistance.
7. The original 20 MPa concrete now averages 60 MPa after 47 years.
8. There does not seem to be any reason to increase the design concrete strength above 35 MPa which is the current practice by the well informed.

Suggested future investigations and follow-up actions:

1. Evaluate shrinkage vs compressive strength.
2. Confirmation that the fatigue strength of 4.8 to 5 MPa is sufficient.
3. Physical checking of freezing depths with concrete pavements.
4. Formulation of proper concrete design standards based on successful (current) overseas practices.
5. On new work establish planned testing of rut formation.
6. Establishment of practical and timely Quality Assurance standards for concrete pavement construction.
7. If JRCP type pavements are used, evaluation of the sizes and grade of the reinforcing mesh is required.
8. Determine who/what should be done with information now available.

9 Task Maanteeamet or TT to appoint a specialist/engineer to keep up with the changes in technology and provide documentation for dynamic guidelines for design, construction and maintenance of concrete pavements.

4 Methodology

My comments may be read as a supplement to the generally excellent research and evaluation undertaken by TT and documented in detail. For the purpose of my commentary, I have followed the general sequence of the TT results (Chapters) and summarised each for easy reference.

5 Test Section

The investigation covered both carriageways, with adjacent test sections of 110 m and 140 m just west pf Pirita River bridges.

The pavement profile was:

22 cm - Reinforced concrete approximately equivalent to the current standard JRCP (Jointed Reinforced Concrete Pavement).
6 cm - Bitumen impregnated coarse sand. (It is assumed that the purpose of this was to provide smooth and accurate levels for the interlayer for the concrete base).
29 cm - Crushed rock.

This 57 cm thickness is regarded as the required thickness to cater for freezing threats (külmaohutu kiht) using a concrete pavement. Visual observations in the field appear to confirm this.

6 Slab (Panel) sizes

It is not known why slab lengths (formed by transverse joints) varied between 4 and 10.8 m, with the median length being 8 m or an average of 7.7 m.

In the 7.5 m pavement width there was a longitudinal joint.

The type of base construction is similar to Sydney's Warringah Expressway, which was JRCP and is now also 47 years old. (Visited by the Estonian Roads Cluster in October 2014).

In both cases the mesh reinforcement is continuous through the transverse contraction joints, however, the slab lengths on Warringah Expressway were designed to vary from 3.8 to 4.2 m. In the last 30 years we have adopted 4.2 m as the standard slab length for Plain Concrete Pavements (PCP) and JRCP.

Here it might be noted that the slab lengths are designed on the basis of allowable maximum shrinkage/contraction movement of 1 mm in the transverse contraction joint. The input parameters are:

- Shrinkage: Limestone aggregate – $6.8*10^{-6}$ mm/mm/°C
  Granite aggregate – $9.9*10^{-6}$ mm/mm/°C
  Quartz aggregate – $12.2*10^{-6}$ mm/mm/°C
  For design we adopt the average of $10*10^{-6}$ mm/mm/°C
- Every 50 kg/m^3 of cement in a m^3 of concrete creates 5 – 7 °C of heat-of-hydration.
- Note that an increase of the w/c ratio from 0.4 to 0.6 increases the heat-of-hydration by 11%.

7 Joints (vuugid)

Testing findings:

1 Transverse joint width varies from 10 mm to 20 mm.
2 Longitudinal joints 30 mm to 40 mm.
3 Odd slabs have moved vertically up to 10 mm at the joints.
4 Some adjacent slabs have shifted up to 7 mm compared to each other (See Photo 3).
5 The depth of the saw cuts to induce the cracking was apparently of the order of 2 cm.

Comments and observations:

1 To induce planned shrinkage cracking, joints apparently were cut, but often too late to stop unplanned shrinkage cracking first.
2 Long slabs will exhibit curling from ambient temperature. This can result in mid slab transverse cracking due to live loads and also permanent steps at joints if for some reason the void under the slab gets filled.

3 It appears obvious that the longitudinal joints were not tied, as is the normal requirement.
4 To induce planned shrinkage cracks the depth of the required saw cut for transverse joints is nowadays D/4. In this case it should have been 220/4 = 55 mm and for the longitudinal D/3 or 73 mm.
5 Mr O Raid, who worked on the project, has stated that most of the unplanned cracking occurred shortly after setting of the concrete as: ''…. vuuke ei jõutud piisavalt kiiresti sisse lõigata''. Hence, it is unlikely that any of the transverse joints were actually properly induced and the effects of poor construction were built in from day one.

6 Example:
 a The Westbound test section is 110 m long and has 14 slabs. Hence the average slab length is about 8 m.
 b Assume that the mix contained 350 kg of cement.
 c The heat-of-hydration is 5-7 ℃/50 kg of cement.
 d Hence, if the concrete was delivered at 20°C would heat up by 49°C, i.e. to 69°C.
 e Thus the expansion/contraction of the 8 mm joint would be 5.5 mm. This is about 5 times more than allowable!

7 The reason that in some areas slab joints are out of sync is the haphazard way joints cracked or did not actually crack. If one joint did not crack as planned then the next one could move 5.5*2 = 11 mm. (And so on).

8 Concrete mix

Test results:
1 It is stated that the maximum aggregate size of 32 mm was observed. This equates to the normal 40 mm aggregate mix.
2 The large aggregate was granite
3 Photo 2 illustrates the very large number of voids in the mortar.

Comments and observations:
1 Granite has a Coefficient Thermal Expansion of $9.6*10^{-6}$ mm/mm/°C.
2 As the voids appear spherical, it appears that some aerating additive was also used in the mix. Possible overdosing?

3 There is no doubt that these voids created some permeability, but it is interesting that there is no sign of frost damage. This would suggest that the pores were not interconnected.

9 Concrete strength

Test results:

1 Photo 5 shows an excellent quality core, except that coarse aggregate is not apparent. (Different mix?)

2 The average compressive strength of the concrete from ''good'' areas was 56.9 MPa (55.6 – 58.6MPa) and from ''bad'' areas 61.8 MPa (58.3 – 65.5 MPa)

Comments and observations:

1 The results are not logical because the concrete from the ''bad'' area had a higher strength and also higher densities. The definition of the ''bad'' area may have been incorrect.

3 The design strength of the concrete is not known, however, it has increased to approximately 60 MPa after 47 years.

4 Based on a similar investigation in Australia, the design strength was probably about 20 MPa. (Warringah Expressway in Australia, had a design strength of 17 MPa and a placement target strength of 20 MPa. After also of 47 years the average compressive strength of 44 cores was 55 MPa).

5 In the testing section the strength gain to approximately 60 MPa would be as normally expected. For any future comparisons it would be safe to assume 60 MPa as the average strength at the end of the 40 year design life.

10 Wheel Ruts (Roobaste mõõtmine)

Comments and observations:

1 According to measurements between 2005 and 2011, the rut depth increased from 6 mm to 23 mm (Graph 4). This would mean that in the first 30 years the rut depth was only 9 mm which does not support the premise.

2 Furthermore, the 2007 and 2011 measurements must be questioned as for Direction 2 the 2007 rut depth is mostly more than the 2011.

3 A very detailed analysis of the rut depth information is warranted to test the quite unreal advice from the Finns.

11 Smoothness (Tasasus)

Test findings:

1 The 2005, 2007 and 2011 measurements remained in the region IRI 4 – 6 m/km.

2 Close examination of the graph, Joonis 5, shows the quite illogical results as, e.g. at Station 2700 the IRI reading is 1040 in 2005 and 400 in 2011.

Comments and observations:

1 The results of 4 - 6 IRI are far too high for any highway surface, however, nothing seems to have been done about it.

2 I think this is primarily due to poor joints, poor quality asphalt patching and apparent nonexistence of routine maintenance as demonstrated on the attached photos of the TT Report.

3 The results as they are do not have any value worth considering.

4 If they were already so bad in 2005, surprisingly it took another 10 years for reconstruction.

5 A concrete overlay could have been considered.

12 Skid resistance (Haardetegur)

Test results:

1 The average of values in the Tallinn to Narva direction were 0.38 (0.32 – 0.45) and from Narva to Tallinn direction 0.36 (0.28 – 0.44).

Comments and observations:

1 Results >0.35 are regarded as acceptable for high speed main roads. This requirement increases for winding roads, junctions and intersections.

2 The 0.36/0.38 result must be regarded as excellent for a surface that is 47 years old.

3 Part of the reason could be the apparent weak mortar which will wear quicker and thus allows the quality aggregate to become more exposed. (See Photo 2).

4 It is not normal to measure transverse skid resistance as well. (What remedial action would this prompt?).

13 Texture depth (Tekstuuri sügavus)

Test results:

1. Nine locations were tested, using the sand patch method.
2. The texture depth varied from 0.41 to 1.01, with an average of 0.75 mm.

Comments and observations:

1. Such texture depth measurements are not applicable to concrete pavements.
2. Depending on the design speed of the traffic , different methods of finishing the surface are used, e.g. brooming, hessian drag, transverse grooving and/or longitudinal grooving.
3. For asphalt, texture depths of greater than 0.5 mm are required. After a year or so of traffic, this is no longer achieved.

14 Water Penetration

Test results:

1. Using the standard water penetration test, it was found that the penetration was 35 – 108 mm with an average of 65 mm.

Comments and observations:

1. The Estonian Concrete Association recommends that penetration less than 100 mm (according to then above test) should be regarded as impervious.
2. This also suggests that the voids/pores in the mortar are actually not interconnected. (An overdose of aerating additive?).

15 Carbonisation and salt impregnation

Test results:

1. The tests showed that possible carbonisation did not penetrate further than 7 mm.
2. Separate tests also showed that salt (NaCl) penetration varied from 0.041 to 0.055% in all samples.

Comments and observations:

1 The concrete was completely carbonisation resistant.
2 Salt water has not penetrated into the concrete and hence there has been no deterioration of concrete.
3 These results confirm that the concrete will not be subject to any frost damage. (Any concrete used nowadays will be of far better quality and hence, will definitely not be subject to any frost damage.

16 Frost damage and resistance

Loss of strength:

1 Average core strength before 100 cycles of freeze/thaw was 47.4 MPa.
2 Average core strength after 100 cycles was 44.8 MPa
3 Hence, the change was 5.5%, against allowable 5%.
4 However, Core 1325-7 had a core strength of only 37.2 MPa compared to the average of 46.3 MPa of the other 5 cores. This odd low strength result should have been rejected.
5 After omitting the odd sample, the loss on freeze/thaw becomes 2.3%.
6 At 2.3% it raises the frost resistance to Class F150, which is the highest rating.

Loss of mass

1 Test results show that after 100 cycles there was a loss of mass of 0.223% which is well under the allowable 3%.

Comments and observations:

1 It is not clear why a 47.4 MPa sample was used as the average for all samples was 60 MPa.
2 The 47.4 MPa sample lost strength of 5.4%. This is slightly over the 5% allowed for Frost Class F100 requirements.
3 It satisfies Class F100 which is applicable for road pavements.
4 After 100 cycles, the loss of weight (by all samples) was less them 0.5%, compared to 5% allowable.
5 In summary, the concrete even after 47 years satisfies the frost resistance requirements.

17 Conditions of joints (Vuukide seisund ja hindamine)

Test results:

1 The original planned/designed joint widths are not known.
2 Photos 6 and 7 illustrate the condition of the ‘’bad’’ and ‘’good’’ joints.
3 Photo 8 shows an opened up joint that is full of sand and fine aggregate. It is obvious that there has been no required maintenance since construction.
4 Photo 10 shows the faces of a washed clean joint.
5 Photo 11 is a joint in bad condition and some 10++ mm in width. There is no sign of sealer material.

Comments and observations:

1 It appears that joints 6&7 have not moved at all and the original bituminous sealing is intact. This means that the movements that should have taken place have occurred elsewhere. Hence, the reason for the many very wide joints.
2 Photo 8 shows that the joint is full of incompressible material which would take up the space every time contraction/shrinkage occurs and hence any subsequent expansion would only widen the joint.
3 In Photo 10, the cleaned joint does not appear to have had a saw cut to induce start of cracking. The reason that no large aggregate is shown is not obvious.
4 It is not stated whether the reinforcement has rusted through or has been cut as part of the testing. In the case of the Warringah Expressway (WE), after 47 years it was found that maximum of 30% of the steel areas of the mesh bars at the crossing of joints had rusted. The WE had a pavement design life of 40 years.
5 In this example, my observation is that aggregate interlock was lost after a very small widening of the crack.
6 Photo 11 shows a very wide joint as mentioned above. Here it is noted that there is no sign of bituminous sealer ever having been in place and material also from the failing arises collects in the joint and stops it contracting.

18 Checks on reinforcement (Armatuuri astme ja seisundi määramine)

Test results:

1. The results of the measurements of the reinforcement are quite variable and inconclusive.
2. In the end it has been assumed that it is a 150 x 150 mesh reinforcement with 6 mm bars.
3. The position of the reinforcement varied between 45 mm and 140 mm from the surface.
4. There appears to be plenty of evidence of rusting of the reinforcement, but no findings that there has been a complete loss of continuity through rusting. Use of salt (water) during winter would certainly accelerate the rusting in unsealed joints.

Comments and conclusions:

1. The positioning of the reinforcement is quite variable and suggests poor control during construction.
2. In Jointed Reinforced Concrete Pavements (JRCP), as this was, the reinforcement is there to hold cracks together and has no structural significance. It is normally placed in the middle third of the slab, i.e. in the space +73 mm and +147 mm measured from the surface. Hence, other than poor construction control, theoretically the location of the reinforcement is generally OK.
3. Due to the many very wide unsealed contraction joints, it must be assumed that the yield strength of all the bars crossing the joint has been exceeded and that the bars have been torn, even before they would have rusted through. (This is also confirmed in the geological investigation section on p19.).

19 Particular comments from the Test Reports

Reference Annexure 3 (Lisa3)

19.1 Wheel ruts (Roopad)

Narva-Tallinn – Average from 4 wheel paths: 14.5 mm
Tallinn-Narva – Average from 4 wheel paths: 13 mm

Comments and observations:

1. It is normal to start planning remedial measures when ruts become deeper than 20 mm.
2. The graph (Joonis 4 p9) of the measurements provided by Maanteeamet registers cannot be regarded as definitive as I have explained in my comments under Heading 8.2 on page 6.

19.2 Crossfall (Kalle)

Tallinn-Narva – 1.4%, min 0.9%

Comments and observations:

1. The recommended crossfall/camber for concrete pavements is 2-3%.
2. As the front suspension and camber in most cars is set at 3%, generally the crossfall of the pavements is also set at 3%.
3. The minimum 0.9% encountered appears a construction error and would lead to unplanned weaving in the travel lanes.

19.3 Water penetration (EVS-EN12390-8:2009)

1. Using the above test method the water penetration varied from 35 mm to 108 mm.
2. Penetration less than 100 mm is regarded as impermeable.

19.4 Salt water penetration – See comments as above.

20 Slab thickness

1. Under heading 5 above, I have quoted the pavement profile from that TT Report. It states that the slab thickness was 220 mm.
2. In the Test Report 1595/14 it shows that the thickness averaged 207 mm with the minimum being 203 mm
3. In Test Report 1596/4 the average slab thickness is 195 mm with the minimum being 183 mm. These are inbuilt construction deficiencies that will also skew other strength related results.
4. A loss of 17% of the design thickness of slabs results in significant loss of the bearing capacity of the base and hence, the design life of the pavement.

21 Terminology

1 In the Geological Report under ''Puurtulp'' it refers to the concrete base as ''Raudbetoon''. This description is incorrect. The reinforcements used in Jointed Reinforced Concrete Pavement (JRCP) or Continuously Reinforced Concrete Pavements (CRCP) are there for crack control and holding unplanned cracks together. ''Raudbetoon'' refers to structural strength concrete.

22 Suggested future investigations and actions

1 Evaluate shrinkage vs compressive strength.
2 Confirmation that the fatigue strength of 4.8 to 5 MPa is sufficient.
3 Physical checking of freezing depths with concrete pavements.
4 Formulation of proper concrete design standards based on successful (current) overseas practices.
5 On new work establish planned testing of rut formation.
6 Establishment of practical and timely Quality Assurance standards for concrete pavement construction.
7 If JRCP type pavements are used, evaluation of the sizes and grade of the reinforcing mesh is required.
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9 Task Maanteeamet or TT to appoint a specialist/engineer to keep up with the changes in technology and provide documentation for dynamic guidelines for design, construction and maintenance of concrete pavements.