ABSTRACT: This paper describes phases of designing and construction of a new fifth runway at Amsterdam Airport Schiphol on very weak soils, 4.5 m below sea level. Schiphol Airport is a major European hub with 42.6 million annual passengers in 2003. The airport authorities had imposed very demanding requirements for the functioning of the new pavement, especially consolidation settlement leading to runway roughness and the runway should be maintenance free for a long time. The pavement had to withstand the loading of more than 6 million operations during the first 30 years after construction including all wide bodied and new large aircraft up to 600 tons take off weight. The structural design included effects of all wheels of all aircraft, using APSDS software and fatigue equations applicable to each pavement layer. The pavement materials used included well tried re-cycled cement treated base and SBS polymer modified asphalt mixes.

KEY WORDS: Airport pavements, pavement consolidation, new large aircraft, cement treated base, polymer modified mix, reflective cracking.
1. INTRODUCTION

The construction of Schiphol Airport near Amsterdam began in 1920, located in the bottom of a reclaimed lake, at an elevation of 4.5 m below the mean sea level. The annual traffic reached in 2003 was 42.6 million passengers and 423,000 aircraft movements; supported by four operational runways with the terminal area located in the centre (Figure 1). Schiphol occupies 4\textsuperscript{th} place among the European airports.

The four operational runways had a combined capacity of 460,000 aircraft movements based on the conditions prevailing at the airport. A master plan study (completed 1995) identified the need to construct a 5\textsuperscript{th} runway, which could increase the runway capacity to 528,000 annual movements.

Based on a number of studies, the fifth runway of 3800 x 60 m was located 2100 m west from the existing runway 18C-36C (see Figure 1). The detailed engineering and design was completed by NACO by the end of 1999 and the construction completed by the end of 2002 and the runway became fully operational with the name “Polderbaan” during February 2003. In the following pages design considerations and implementation of civil works including aircraft pavements of the new runway is presented.

Figure 1: Amsterdam Airport Schiphol - Layout
2. PROGRAM OF REQUIREMENTS

The management of Schiphol Airport had conducted their own internal study. The requirements of the new runway to satisfy local regulations and international standards were formalised during 1997-1998 in a document termed “Program of Requirements” (1). This document was detailed, covering all aspects of the 5th runway development and formed the basis of the design brief to the consultant. As the design progressed the requirements were updated and modified by the airport management. This system avoided the misinterpretation and ambiguity of requirements and enabled the development of design and reporting process without rancour.

The significant aspect of the requirements were:
- The loadings of then known new large aircraft, B-747-600 are to be considered in the design.
- The foundation layers of the pavement should be designed for at least 30 years traffic.
- The asphalt surface layers should be maintenance free for the first 15 years.

3. SITE CONDITIONS AND INVESTIGATIONS

The airport is built on the bottom of a former lake and is approximately 4.5 m below the sea level. The soil conditions consist of an upper layer of 0.50 m topsoil followed by 7.5 m of clay and peat over dense sand as shown in Figure 2.

Extensive soil investigations was carried out including Dutch cone penetration tests, borings and sampling of soils with laboratory analysis during 1995 – 1997 (2).

<table>
<thead>
<tr>
<th>MSL (m)</th>
<th>Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>-4.5</td>
<td>Topsoil</td>
</tr>
<tr>
<td>-5.0</td>
<td>Clay</td>
</tr>
<tr>
<td>-6.0</td>
<td>Sandy Clay</td>
</tr>
<tr>
<td>-9.5</td>
<td>Clay / Peat</td>
</tr>
<tr>
<td>-12</td>
<td>Peat</td>
</tr>
<tr>
<td>-12.5</td>
<td>Dense Sand</td>
</tr>
</tbody>
</table>

Figure 2: Stratification 5th Runway

The aim was to arrive at the following:
- Expected settlement due to pavement construction along the runway alignment.
- Subgrade bearing capacity for pavement analysis.

The computed long term consolidation settlement for the expected pavement loading varied between 0 – 35 cm and depended on the thickness of the compressible layers and presence of sand deposits. The computed magnitude of settlement and the actual values achieved are shown in Figure 3.
The Dutch cone penetration value varied significantly at locations and depth. The cone resistance varied considerably. Based on the analysis of this variation up to depth 4.0 m below pavement, a characteristic cone resistance (85 percentile) value of 0.60 MPa, was chosen which translated into a subgrade elastic modulus of 30 MPa.

4. CONSOLIDATION SETTLEMENT

At Schiphol Airport, the existing 4th runway 01L-19R, had experienced an uneven settlement, after 30 years of operation (1994), with a maximum local settlement of 200 mm. This situation occurred, because the pavement was placed on unconsolidated terrain. For the new fifth runway this risk was not acceptable, it was decided to initiate and complete the primary settlement prior to placing the pavement to avoid pavement roughness in the future. Therefore during the design stage, 8 consolidation methods were considered, ranging from pre-loading forced drainage to light weight pavements. In addition a structural runway supported on piles was also considered as an option.

A detailed analysis of these options considering, costs, time and environmental aspects, a Dutch patented system IFCO (3) was chosen. This system is a combination of a horizontal drainage and a forced lowering of the ground water. The system consists of mechanically installing a perforated drain at the bottom of a vertical trench 0.25 m wide and 5.0 m deep. The trench is backfilled with sand. The perforated drain is connected to a ventury pump. A layer of 1.15 m of sand representing the weight of future pavement was placed on top of the soil (after removing the organic top soil) with PVC sheet separation, which also acted as an air tight cover.

When the pumps are working, they remove a mixture of groundwater and air. This results in an atmospheric pressure on top of the construction, adding extra weight for consolidation. Consolidation using this system shortens the time for settlement by more than 50 percent compared to traditional pre-loading. It also saves considerably logistical problems transporting and removing sand for pre-loading.

In order to validate and demonstrate the effectiveness of this system, a pilot project was carried out at the north end of the new runway, of approximately 14,000 m² in 1997. The lessons learnt, allowed modifications to the system and was used full scale on the new runway and part of the parallel taxiway system. The consolidation settlement was completed between March 2001 and April 2002 and was fully monitored.

The accelerated consolidation settlement was at various locations reasonably in agreement with the computed values. The consolidation process (extraction of water and air) was stopped when the time settlement curve was flat for at least 30 days. More details of the system used can be had at reference-3.
5. TRAFFIC DETAILS AND LOADS

The forecasted air traffic on the new runway was defined by the programme of requirements. It was separated into two phases as shown in Table 1. During this stage of design (1997), the unknown factor was the eventual undercarriage system of the new large aircraft (NLA), contemplated both by Boeing and Airbus industry. The then available undercarriage configuration of B747-600 at 600 tons is as shown at Figure 4, which has a grouping of 4, 4, 6, 6 wheels for the main undercarriage. The spacing between 6-6 wheels is greater compared to 4-4 wheels.

Table 1 Traffic Details

<table>
<thead>
<tr>
<th>Aircraft</th>
<th>MTOW (t)</th>
<th>Phase-1 2001 - 2015</th>
<th>Phase-2 2015 - 2030</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Departure</td>
<td>Arrival</td>
</tr>
<tr>
<td>Saab 340</td>
<td>14</td>
<td>37,975</td>
<td>49,000</td>
</tr>
<tr>
<td>Fokker 50</td>
<td>20</td>
<td>164,920</td>
<td>212,800</td>
</tr>
<tr>
<td>Fokker 100</td>
<td>45</td>
<td>338,520</td>
<td>436,800</td>
</tr>
<tr>
<td>Airbus A320</td>
<td>74</td>
<td>340,690</td>
<td>439,600</td>
</tr>
<tr>
<td>Boeing 767-300</td>
<td>165</td>
<td>187,705</td>
<td>242,200</td>
</tr>
<tr>
<td>Boeing 747-400</td>
<td>396</td>
<td>90,055</td>
<td>116,200</td>
</tr>
<tr>
<td>MD-11</td>
<td>275</td>
<td>49,910</td>
<td>64,400</td>
</tr>
<tr>
<td>Boeing 777-300</td>
<td>299</td>
<td>29,295</td>
<td>37,800</td>
</tr>
<tr>
<td>NLA (747-600)</td>
<td>600</td>
<td>9,765</td>
<td>12,600</td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td>1,258,835</td>
<td>1,611,400</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>2,860,235</td>
<td></td>
</tr>
<tr>
<td>Grand. Total</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The designers were aware of the industry discussions pertaining to the under carriage configuration of future large aircraft. Hence configuration changes were anticipated. Therefore the following options were included while making structural pavement computations applicable to new large aircraft, which were expected to use the new runway after 2006.

To include the effect all wheels of the main under carriage instead of one selected main gear as currently followed by FAA.

Bring the spacing of the 6-6 wheels closer by 40-50 percent to account for possible changes in configuration.
This approach, which was considered too conservative during 1997, was justified later when Airbus Industry froze the undercarriage configuration of A-380 during January 2002, as shown in Figure 5.

6. MATERIALS AND DESIGN PARAMETERS

6.1. Subgrade

It is a common practice at Schiphol Airport to place a 50 cm layer of sand over the insitu soil after removing the topsoil. This helps trafficking of construction vehicles and also act as a drainage layer. The arrangement is shown below.

<table>
<thead>
<tr>
<th></th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandfill</td>
<td>100 MPa</td>
</tr>
<tr>
<td>Insitu subgrade</td>
<td>30 MPa</td>
</tr>
<tr>
<td>Combined modulus</td>
<td>45 MPa</td>
</tr>
</tbody>
</table>

The subgrade fatigue equation used is based on FAA, as per equation 1:

\[ N = \left(3.3 \times 10^{3}/ e_v\right)^{8.36} \]  

where: \( e_v \) = vertical strain, \( N \) = coverage to 25 mm rutting.

Dutch Cone Penetration tests are routinely carried out in the Netherlands to assess the load bearing capacity of soil strata both for pavements and buildings. Based on 150 DCP tests on the new pavements, a design cone penetration resistance of 0.60 MPa was taken as representing the subgrade.

6.2. Cement Treated Base (CTB)

At Schiphol Airport almost right from the beginning cement bound bases have been used with great success, locally called “walsbeton”, is close to lean concrete. Due to high ground water and constant humid environment, the strength of cement bound bases seem to grow continuously. 25 Year old “walsbeton” has shown core strength exceeding 25 MPa. Stiff bases at this airport also has the beneficial advantage of spreading the loads onto weak soils which from the subgrade. Therefore, cement treated base was chosen as the principle backbone of the new runway.

The Netherlands, being a low country, a great portion of the land is reclaimed from the sea. There is hardly any local resource for good quality aggregate. Most of the aggregate are imported from neighbouring countries. At this airport, it was a practice and still is, to separately stockpile demolished materials from asphalt and concrete pavement, concrete structures and bricks from street pavements. During 1990, Schiphol Airport initiated a research programme to re-cycle the stockpile materials for cement treated base (CTB). (16, 17 & 18).

The research team included the present authors, Technical University Delft and a selected contractor. The conclusion of this programme was, that excellent CTB material can be produced from a mixture of re-cycled asphalt and concrete pavement bound with normal Portland cement. A puzzolomic cement is preferable as it improves in strength with age in a humid atmosphere. The best combination of re-cycled materials was 30 percent of re-cycled
asphalt with 70 percent of re-cycled concrete with a cement content between 150-200 kg m$^3$. At this cement content, the flexural strength at 28 days was between 20-24 percent of the compressive strength. A combination of 40 percent asphalt with 60 percent concrete recycled aggregate did not meet the strength requirements. Therefore the following parameters for CTB were chosen for pavement dimensions:

- Asphalt to concrete in the mix = 30 : 70 percent
- Compressive strength 28 days = 12.5 MPa (85% value)
- Flexural strength 28 days = 2.5 MPa (85% value)
- E-value – uncracked = 10,000 MPa (L = > 5 T)
- E-value – cracked = 4,000 MPa (L = T to 5 T)

In order to limit the temperature movement, the CTB is routinely pre-cracked at Schiphol Airport and a simple pre-cracking equipment was evolved for this purpose reported by Nataraj (15). Photo 1 shows pre-cracking followed on the new runway.

The fatigue model used for CTB layer (first crack between the pre-cracking) is at equation-2.

$$\text{Log (N)} = 11.7.8 – 12.12 (\text{SR}) \quad (13) \quad (2)$$

where: \( \text{SR} = \text{tensile stress}/28 \text{ day flexural strength} \)

Any strength gain beyond 28 days was used as a safety factor to account for plant production and compaction variations in the field.

6.3. Asphalt Surface Course
At Schiphol all operating surfaces for aircraft other than parking apron are made up with asphalt base and surface courses using un-modified asphalt. Historically a 270 mm total asphalt packet is performing well without any reflection cracks due to underlying lean concrete base (walsbeton) which was not pre-cracked.
A research programme including a pilot project was initiated by Schiphol during 1989-'90 to examine the use of Polymer Modified Asphalt (PMA) in place of unmodified asphalt. The details are extensively reported by Molenaar and Nataraj (19). The conclusion was that a total thickness of 190-200 mm of PMA was sufficient to prevent the reflection cracks. Since early ninety’s all existing 4 runways have been upgraded by a PMA overlay with SBS modified bitumen. The performance of these modified mixes has been extremely satisfactory. Therefore use of PMA for the fifth runway was retained. The PG (performance grade) required for the modified binder was PG 76-20. The mix design was based on performance requirements, especially resistance to reflection cracks (toughness) and resistance to permanent deformation as indicated in Table 2. Based on W-MAAT of 10°C and stiffness (S mix) of 5500 MPa, the fatigue equation used for PMA mixes based on (10) is as per equation -3.

\[ N = \left( \frac{k}{e_t} \right)^5 \quad k = 4655.10^{-6} \quad - (3) \]

where : \( N \) = coverage, \( e_t \) = tensile strain

Bitumen PEN 80-100 modified with SBS under the brand name “Sealoflex” was the modified binder chosen for this project, as it had proved its good performance at the airport. One of the lessons learnt is, the compaction standards of PMA mixes are difficult to achieve under the combination of wind and air temperature below 5°C, which occurs often in the Netherlands.

### Table 2 PMA Specifications

<table>
<thead>
<tr>
<th>Description</th>
<th>Test method</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Permanent deformation at 40°C</td>
<td>PrEN 12697-25, part B</td>
<td>max 1,0% *1, min 750 GPa.s *1.5</td>
</tr>
<tr>
<td>- Mix viscosity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Permanent deformation at 60°C</td>
<td>PrEN 12697-25 part B</td>
<td>max 2,0% *1, min 500 GPa.s *1.5</td>
</tr>
<tr>
<td>- Mix viscosity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance to reflection cracks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IDT at 0°C</td>
<td>PrEN 12697-23</td>
<td>Min 3,15 MPa, Av-3,5 MPa *2</td>
</tr>
<tr>
<td>Toughness at 0°C</td>
<td>PrEN 12697-23 *4</td>
<td>Min 11,5 Nmm/mm², Av-14,0 Nmm/mm² *2</td>
</tr>
</tbody>
</table>

*1 Average 4 tests
*2 Average 6 tests
*3 Test conditions: \( \sigma_c = 0,00 \) MPa; \( \sigma_h = 0,40 \) MPa; \( T_1 = 0,2 \) s; \( T_0 = 0,8 \) s; \( N = 10,000 \); blok puls
*4 Force-displacement curve
*5 Mix viscosity from slope of deformation test

7. STRUCTURAL PAVEMENT DESIGN

The structural design was made based on layered analysis using APSDS software (7), which was just then (1997-1998) available. Certain modifications were done to accommodate the
The fatigue equations mentioned earlier. APSDS is a powerful tool which can account the cumulative damage of all wheels and all aircraft operating on the pavement including wander. The wander of the wheel track chosen was 1546 mm as per FAA (4).

The APSDS allows to compute cumulative damage factor (CDF) across the width of the pavement for all the layers. The layer where CDF approaches unity (1) indicates the end of fatigue life.

The pavement model used is shown below:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (mm)</th>
<th>E (MPa)</th>
<th>Fatigue Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMA</td>
<td>thickness = 200 mm</td>
<td>E&lt;sub&gt;1&lt;/sub&gt; = 5,500 MPa</td>
<td>(3)</td>
</tr>
<tr>
<td>CTB</td>
<td>thickness = variable</td>
<td>E&lt;sub&gt;2&lt;/sub&gt; = 10,000 MPa or 4,000 MPa</td>
<td>(2)</td>
</tr>
<tr>
<td>Sand Layer</td>
<td>thickness = 500 mm</td>
<td>E&lt;sub&gt;3&lt;/sub&gt; = 45 MPa</td>
<td>(1)</td>
</tr>
<tr>
<td>Subgrade</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The structural computations were divided into 2-phases.

Phase-1: Traffic from 2001-2015 (Table-1). During this period CTB layer is the main load carrying layer and its thickness should be such that CDF for phase-1 should not exceed 1 using equation-2.

At the end of this phase it is assumed that there will be a load induced crack in CTB between the pre-crack spacings (para-3).

At a characteristic flexural strength of CTB at 28 days of 2.50 MPa, the required thickness to reach a CDF-1 is approximately 680 mm.

Due to high stiffness of CTB, the CDF of PMA layers and subgrade was extremely low and not an issue.

The CTB layer thickness was hence rounded off to 700 mm to account for thickness variation during construction.

Phase-2: Traffic from 2015 – 2030 (Table 1).

The assumption during this period was, that CTB will have a cracked modulus of 4000 MPa as against 10,000 MPa (un-cracked). For this model only the subgrade permanent strain criteria was deemed critical.

The CDF of subgrade for this traffic was 0.08 which is extremely low.

8. RISK ANALYSES

A number of “what if” analyses were made to have a perspective on the perceived risks. These involved efficiency of settlement measures, localised weak soils, performance of CTB and reflection crack potential due shear movements of CTB joints and cracks.

Due to the large thickness of CTB (700 mm), it had to be placed and compacted in a number of layers. The previous experience at the airport had shown, that the contractors could
efficiently compact the CTB up to 350 mm in one layer. Computations indicated that, a good bond between layers was extremely important. De-bonding of CTB layers would drastically increase the risk of pavement failure and should be avoided.

It was also found that any strength increase of CTB beyond 28 days should not be considered in the structural analysis and thus an additional safety is built in to the layer. However, CTB should be from a central plant with specified quality control during the mixing, field compaction and curing.

With regard to reflection cracks due to the stiff CTB layer, precracking of all the layers of CTB is a must and effective plate length should be restricted to between 2.5 to 3.5 m. Experience at Schiphol Airport had proved the pre-cracking effectively limits temperature movements. Due to the short crack spacing, the crack width after shrinkage is very narrow (<2 mm) and load transfer is efficient due to aggregate interlock. Due to this and also due to large thickness of CTB, vertical shear movements under aircraft load is insignificant. The risk of reflection cracks due to Mode I and II movements of CTB is significantly reduced. The presence of 200 mm PMA, with a specified toughness criteria further restricts the development of reflection cracks. Based on similar constructions, the performance of heavily loaded new taxiways at this airport (± 10 years old) have been exemplary.

The final runway pavement chosen is at Figure-6. The new 5th runway at Schiphol Airport (named “Polderbaan”) was opened to traffic during February 2003 (Photo 2). The declared PCN is 95/F/C/W/T for unlimited traffic. The construction of the pavement needed 400,000 tons of CTB and 120,000 tons of Polymer Modified Asphalt.

Figure 6: Pavement construction 5th Runway
9. CONCLUSIONS

From the foregoing the following conclusions are relevant:

a) Airport pavement for heavy traffic can be constructed on weak soils provided sufficient attention is paid to the consolidation characteristics of the insitu soil.

b) Significant consolidation time can be saved by using innovative consolidation methods such as IFCO system.

c) Such methods will also reduce the environmental aspect of transporting large quantities of surcharge material used for traditional methods.

d) A balanced airport pavement, almost maintenance free, can be achieved using recycled semi rigid construction and performance based polymer modified asphalt surfacing.

e) It is recommended that all main under carriage wheels to be considered while designing new airport pavements, which is a departure from the current ICAO and FAA standards.

f) When cement stabilised bases are used, it should be mandatory to precrack all layers to a suitable pattern and bond between layers should be ensured. This will reduce the risk of reflection cracking.
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