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REHABILITATION OF CAUCASIAN HIGHWAYS

Azerbaijan / Georgia / Armenia

Shemkir to Gazakh Road - Azerbaijan

Pavement Design Evaluation

Design Review Report

August 2003

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Rehabilitation of Caucasian Highways
Azerbaijan Georgia and Armenia

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1st September, 2003
Subject: Design Review Report

Attention Mr. M Graille

Dear Sir

Please find attached the Design Review Report for your consideration.

Best regards

S I Dotchev
LBSA Project Manager's Representative

JACOBS

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Client:
Project: REHABILITATION OF CAUCASIAN HIGHWAYS — Job No: J23147
Title: PAVEMENT DESIGN REVIEW REPORT

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Executive summary

The M1 Highway forms part of the TRACECA corridor from Baku, Azerbaijan to Poti, Georgia by the Black Sea. The road connects the three capitals of the Trans-Caucasian Republics: Baku; Azerbaijan Tbilisi; Georgia and Yerevan; Armenia.

The World Bank has agreed to finance the rehabilitation and upgrading of the existing single carriageway Ganja to Gazakh road sections (Azerbaijan Highway Project). The section under review in this report is a 73km section CW-2003 Shemkir to Gazakh.

This is the design review report by a pavement design expert. The purpose of this review is to develop an overview of the design, summarise the current situation and anticipate follow up actions

The pavement rehabilitation was designed by KOCKS CONSULT GMBH as described in an Engineering Report (October 2002). KOCKS prepared contract drawings (July 2001), which included preliminary drawings for the pavement.

The visiting Jacobs pavement design engineer performed a Visual overview of the site to establish that the proposed design was commensurate with the projected levels of traffic.

A number of inconsistencies were observed in the KOCKS report, some of a major nature. The conclusions from the report and the design rely heavily on a FWD survey conducted in June 2001. The FWD analysis appears to be inconsistent with two design temperatures and reports bituminous stiffnesses for aged material in excess of those practically achievable for new material. Analysis of the grading from a wearing course has been compared to that of a base course. Traffic figures have been used which may not be applicable to the scheme under consideration.

The current level of traffic (two-way 2,000 to 3,000 vehicles per day) indicates that the section will not require upgrading to a dual carriageway during the current pavement design life (15 to 20 years).

Contents

1 Introduction

The M1 Highway forms part of the TRACECA corridor from Baku, Azerbaijan to Poti by the Black Sea. The road connects the three capitals of the Trans-Caucasian Republics: Baku, Tbilisi and Yerevan.

This report refers to the pavement design expert visit to Ganja, Azerbaijan in August 2003. The visit lasted from the 5th August 2003 to the 9th August 2003. The objective of the visit was the review of the pavement designs for the Rehabilitation and Upgrading of the Shemkir to Gazakh road section. These pavement designs are required as part of an overall project involving a World Bank credit for the rehabilitation of the Ganja to Gasakh highway. The scheme is fully within the International Development Agency funded Works Contract for the rehabilitation of the Ganja to Shamkir road section of motorway M1.

2 Methodology

The review was undertaken by pavement design expert Dr Michael Heelis, employed by Jacobs working in association with Louis Berger S.A..

The purpose of this design review has not been to undertake a detailed check on the Designer's work. The review has therefore aimed at developing an overview of the proposed pavement designs. In this process, it is inevitable that some quite detailed points will be observed in the course of looking at the functionality of the designs and these points have been noted as well as those of greater significance. Responsibility for the Design remains with the Designer who is quality assured to ISO 9001.

The first stage involved a review of all the documents in association with a site visit by the Pavement Engineer from 6th to 8th August 2003. This included a review of the preliminary design and the associated background information as detailed in the Engineering Report prepared by KOCKS in October 2002. A review of the drawings which form part of the Bidding Documents prepared by KOCKS Contract CW-2002 has also been performed, where this affects the design of the pavement structure. This process identified where designs were not consistent with the current visual condition of the road. The dominant issue in the review has been the appropriateness of the design.

The Review has been conducted with the following objectives:

- *To review the existing road conditions to identify the distress that has occurred to the existing pavement, which are not traffic volume or service life related, and to the drainage.*
- *To review and comment on the type and extent of sampling and testing .*
- *To review the Pavement Designs considering all pertinent factors and data including :*
  - *Geotechnical Results*
  - *Construction material results*
  - *Material availability, haulage and costs*
  - *Axle load survey*
  - *Traffic Volume and composition*
  - *Future maintenance requirements and cost.*

3 Review of Existing Road Conditions

3.1 General Descriptions

The Shemkir to Gazakh road forms part of the main road corridor extending from Alyat near the Caspian Sea to the Georgian Border. The section begins at the major roundabout Shemkir/Deliler/Gazakh/Baku (km 390.0 Site Chainage 0). The alignment of the road is consistent with existing standards traversing a flat rolling terrain with long straights and occasional bends. It is predominantly in a rural setting with no housing on either side of apart from where it passes through Tovuz, which is effectively bisected.

From Tovuz the road continues over gently undulating terrain normally on 1-2m height embankment but with sections in cut of up to 5-10m depth. The town of Agstafa is bypassed to the west after which the route turns sharply westward to run parallel to the Agstev river. The main centre of Gazakh is bypassed with the road routed along the southern and western limits of the town in an urban setting. The road section ends at km 463.8 (Chainage 73.8).

3.2 Existing Road Condition

The KOCKS Engineering Report is deemed to accurately reflect the existing pavement condition. The visual inspection conducted as part of this review suggests that the current pavement structure is approximately 5-10 years old. This has not been confirmed from documentary evidence. There is widespread deterioration across the whole width of the pavement however the severity of the deterioration would be classed as moderate according to TRL (United Kingdom Transport Research Laboratory) Overseas Road Note (ORN) 18.

Surface Rougness

The surface roughness provides a comfortable ride up to speeds of 100-120 kph although sections on the approach to Tovuz are such that a lower speed (80kmph) is required for a comfortable ride. The roughness of this section is considerably less than other roads in Azerbaijan in particular the road from Baku to Ganja.

Rutting Deterioration

No significant wheelpath or structural rutting was observed during the inspection in August 2003 although the section Site Chainage 48.6 to 49.7 has been recently patched and it is understood the principal mode of failure had been rutting in the wheel paths.

Visual Condition

Road condition data, in particular visual condition data, is presented in summary format only, and does not allow correlation of rehabilitation recommendations with the visual condition of the road. For example, areas with little current deterioration should correlate to sections with a 40mm overlay recommendation and sections indicating deterioration should require reconstruction or thicker overlays.

There are moderate lengths of edge deterioration and road shoulders are typically un-sealed. No heavy vehicles were observed over-running the pavement shoulders. Therefore it is assumed likely that the shoulders were unsealed when constructed.

This is not a practice that is typically recommended and the shoulders should be sealed.

Pavement Drainage

There appears to be sufficient cross fall on the pavement surface to prevent surface ponding of rainwater, and there is no reported problem with surface drainage from the actual pavement.

Where the route passes through rural areas the pavement surface is typically on embankment 1-2 m above the surrounding areas. Drainage from the road is directly onto adjacent fields where drains were either not initially constructed or have been filled in over-time. The proposed arrangements in the KOCKS Bidding Documents, which comprise sidedrains that shall be constructed when the height of the embankment is less than 1.0m should alleviate these problems if they are maintained. The longitudinal slope of the proposed design is a matter of concern as it is below the recommended standard (or 0.3 to 0.5%) in order to ensure water is efficiently removed from the road. The drainage of water from the side drains should be actively promoted in the final design, in order to prevent underlying pavement layers from being in saturated conditions for long periods and accelerating pavement deterioration.

Where the road passes through an urban environment provisions for road drainage are either non-existent or have deteriorated to such an extent as not to be effective. Residential accesses have been created across drainage systems which can lead to flooding during periods of wet weather. Provisions for urban drainage should be recommended.

Earthwork Failures

The earthworks are generally in good condition with few occurrences of localised settlement or slope instability. However there appears to be some localised failures on the approaches to the overbridge to the railway at Chainage km 60.400. This section is to be re-aligned and a new bridge is to be constructed. The new earthworks will be constructed alongside the existing earthworks which will then be required to support the new construction. The existing earthworks may provide insufficient support. The suitability of the existing embankment for the current design, in terms of required compaction, stability etc, should be checked prior to construction.

4 Review of Geotechnical Investigations

4.1 General Descriptions

The type and extent of the geotechnical and pavement structure surveys conducted as part of the KOCKS Engineering Report (October 2002) are outlined below.

Trial Pits

A total of 15 Trial pits to a target depth of 0.8m were excavated. The stratigraphy of each trial pit was noted and bulk samples were recovered (Appendix A.4 Table 1 of the KOCKS Engineering Report). The bulk samples were a nominal 50kg in weight however this appears to be excessive from a single trial pit and may indicate that the samples were combined to provide sufficient mass for subsequent laboratory tests. The reported grading curves do not conform to the TRL ORN 31 standard, the primary reason being excessive large size aggregate in excess of 50mm. This may have reduced the possible level of compaction of this layer during construction.

Trial pits were performed both in the carriageway and at the carriageway edge adjacent to the shoulder The cross-section reported in the KOCKS Report indicated that at the carriageway edge there was 80 to 110mm less bituminous material than in the carriageway. This is probably one of the principal causes of the edge deterioration observed along the road section.

Dynamic Cone Penetrometer Testing

Dynamic Cone Penetrometer (DCP) testing according to the standard specified in TRL ORN 18 was performed at a nominal 1km spacing along the entire road section. The results from the tests allowed the thickness of the relevant layers to be identified along with the nominal California Bearing Ratio (CBR) strength (of the granular sub layers). The raw data from these test are not available in the KOCKS Engineering Report and therefore the testing methodology cannot be confirmed. However the results appear to be consistent with the existing road structure and identify two underlying pavement layers. The upper layer typically has a CBR in excess of 100% and the second is of inferior strength CBR 15% overlying a subgrade with CBR 4-5% (KOCKS Appendix A.4 Table 2). The upper layer can be classified as a good quality granular sub-base in line with applicable standards. A typical capping layer would have a CBR in excess of 30%. This requirement is normally specified in order to ensure that overlying layers can be compacted efficiently. The overall performance of these underlying layers appears to be good as there are few signs of structural rutting or localised settlement.

Percussion Borings

On the alignment of the proposed second carriageway, 13 no. smaller percussion borings have been carried out. The resulting soil profiles indicate that there is clay subgrade but no other information from this survey is presented in the KOCKS Report.

In order that a suitable pavement design for the dualling route can be established the strength of the subgrade must be confirmed prior to the start of construction.

Coring Survey
A further 18 no. 60mm diameter cores were taken through the pavement surface at a spacing of approximately 5km along the route. The stratigraphy of the pavement was identified and presented in KOCKS Appendix 4 Table 3.

Falling Weight Deflectometer Testing
A Falling Weight Deflectometer survey (FWD) was conducted, testing the pavement structure at staggered 100m intervals. The subsequent analysis of the pavement was used to identify the proposed rehabilitation regime and will be discussed in Section 4.2

Conclusions
The extent of the investigation of the current pavement structure appears to be sufficient to identify the current construction and the strength of the relative layers and identify rehabilitation requirements.

There is insufficient information to assess the suitability of the ground and drainage for the provision of a dual carriageway along this route. The adequacy of the geotechnical survey for bridge construction is outside the scope of this report.

4.2 Falling Weight Deflectometer Survey

A Falling Weight Deflectometer (FWD) Survey of the entire length of the section was conducted and the results reported in the KOCKS Report.

The deflections measured by the FWD or examples of the analysis have not been reported and therefore the results from the survey cannot be validated.

4.2.1 Back Analysed Stiffness

The methodology for the back-analysis of the FWD results has used the Method of Equivalent Thicknesses as recommended by International Experts such as Ullditz (1999). However, the preferred method of back-calculation recommended by the UK Transport Research Laboratory is to use Burmisters equations (UK Design Manual for Roads and Bridges (DRMB) Volume 7 HD 29/94).

Typically, results from the sensor at a distance from the centre of the loading plate can provide information at approximately the same depth downwards in the structure. The typical height of the embankment is 1-2 m and therefore the information reported to be about the subgrade at a depth of approximately 1.27 m may reflect the condition of compacted imported material at the base of the embankment.

The sensor closest to the loading plate is at a distance of 210mm. Typically this would mean that the minimum thickness of the surface layer for analysis should be of the same order i.e. 200-300mm. Layers with a thickness of 30 to 60mm have been analysed in the Engineering Report produced by KOCKS. The results for these layers are unlikely to be consistent with the in-situ pavement stiffnesses.

The back-analysed stiffness is normally adjusted to a design temperature typical of the site conditions. The design temperature for the Section Shekmir to Tivuz road section (length 40.2km) is quoted as 25°C. The design temperature for the Tovuz to Agstafa road section (length 67.1 km) is quoted as 35°C. No indication as to the reason for the change in design temperature has been given.

A typical sample of new bituminous material will have a maximum stiffness of 7000MPa (DRMB) at 20^{0}C.The stiffness of bitumen reduces with higher temperatures and at the proposed design temperature of 25^{0}C or 35^{0}C (according to the KOCKS report) a maximum stiffness of 4000MPa would be expected. The reported stiffnesses in the KOCKS Engineering Report for new asphalt layer:

- New Asphalt layer < 100mm Stiffness 2000MPa
- New Asphalt layer > 100mm Stiffness 3000MPa

The poor visual condition of the bituminous material would indicate that a lower stiffness would be expected for the existing bituminous material. However, the back-analysed stiffnesses for the existing bituminous material in the report are regularly in excess of 10,000MPa indicating that they are better than new material. This would appear to indicate that the FWD Analysis is inconsistent.

In order to check the accuracy of the analysis technique, it is common practice to provide data comparing the measured deflection data and the deflections calculated using the stiffness output from the analysis. No such data is provided and therefore the analysis cannot be validated.

The recorded FWD survey length is 107.1km and is in excess of the scheme length of 73.8km.

Results reported for the FWD analysis have been observed to be inconsistent with International practice. It has not been possible to perform a check of the analysis as the consultant used by KOCKS has not supplied examples of the following data which would be required to review the FWD analysis (see UK Design Manual for Roads and Bridges (DRMB) Volume 7 Section 29/94):

- Measured deflection data (Maximum and Differential data is normally reported)
- Pavement Temperature at the time of testing (only the design temperature has been supplied)
- Correction method for the Pavement Temperature
- Design Temperature (Why have two design temperatures been adopted?)
- Design (Deterioration) Curves for each layer
- Data on the error between the calculated and measured deflection bowls.

The final designs in terms of overlay rely totally on the results of the FWD survey. Further conclusions about the appropriateness of the recommended rehabilitation regime, for instance the recommendation of an overlay compared to an inlay, cannot be assessed from the current analysis.

4.3 Laboratory testing

4.3.1 Existing Bituminous Material

The bituminous content and grading of the aggregate in the existing bituminous material from the trialpit survey was undertaken. The recorded bitumen content was typically approximately 4% for the wearing course and 3% for the underlying bituminous layers. A typical design bituminous content of a bituminous macadam would be in the range of 4.5-5.5%. Some reduction from the design content would be expected due to the fact that the bitumen is being recovered from aged samples. The seasonal temperature range would also have some bearing on the choice of wearing course bitumen content.

In Appendix A4 Table G-7 to G-10 the KOCKS report compares the grading of the in-situ material with that taken from TRL ORN 31. The grading of the upper wearing course layer is compared to a Hot Rolled Asphalt Base Course layer (Ref. BC3) and the second asphalt layer is compared to a Road Base mix (RB3).

The wearing course grading of the in-situ material has excess material in the particle sizes 0.01 to 1mm. Additionally there is too much material with aggregate size in excess of 20mm. There is material of 30mm particle size in a nominal 40mm layer. This larger aggregate will cause particular problems when compacting the layer during the construction process leading to an excess of voids. The visual deterioration of the surface layer is typical of such a problem.

In addition, the large aggregate has become polished with use and this will decrease the skid resistance of the surface when wet. This is also typical of using un-crushed aggregate for bituminous layers. It may also prevent aggregate interlock with overlying layers when these are placed in the current scheme and care must be taken in the preparation of the surface prior to the overlaying process.

4.4 Existing Granular Material

The sub-base material from the trialpits was also taken to the laboratory for grading analysis. Similar to the bituminous material the grading was compared to that in TRL ORN 31 in the KOCKS Engineering Report. Aggregate with particle sizes in excess of 50mm was found and this may affect the compaction of layers on site. The thickness of the sub-base layer varied considerably but was typically 200mm. Despite the excess of coarse material the CBR strengths typically reported are in excess of 100%. This figure should be treated with caution as the Dynamic Cone Penetrometer (DCP) probe may have hit large aggregates and will give un-typical readings.

4.5 Subgrade

It is believed that the strength of the subgrade was assessed by several different methods. Where the DCP test penetrated through the sub-base layers the underlying CBR was reported to be typically 4-5%. The stiffness of the subgrade from the FWD tests has been converted to a strength using equations from TRL ORN 18. The conversion by such a method is typical where the strength and stiffness of a material are confused. It is not always the case that a stiff material is also very strong as such effects as aggregate interlock and moisture content may affect the relationship between strength and stiffness of soils.

The pavement design of the section km 398 to 402 (Page 31 of the KOCKS Report) Chainage km 8.000 to 12.000 is apparently based on DCP tests which did not fully penetrate the imported granular material. On page 24 the CBR of 12% is attributed to a section at km 402 to 412 which would correspond to a site Chainage 12.000 to 22.000. The natural subgrade under the proposed new alignment for the dual-carriageway is classified with a typical CBR of 2% at the same location.

Without supporting information and confusion of the extent of the stiffer underlying material, it will be extremely risky to construct any section without a 300mm capping layer. A conservative design would take the in-situ CBR of the subgrade to be 2%. The adoption of this approach would lead to the prevention of premature deterioration and possible pavement failure.

4.6 New Material

In order that that proposed rehabilitation gives good performance and achieves the proposed design life, it is necessary that the quality control of new material is effective. Material outside the proposed grading curves will result in a sub-standard pavement which will not achieve the proposed design life. Primarily this will be due to insufficient compaction of the surface layer which will lead to fretting (aggregate loss) from the bituminous surface, and subsequent moisture contamination of the primary pavement structure layers. The provision for a 40mm overlay will be problematic on site if a suitable supply of graded material is not obtained.

The trials pits where 100mm less bituminous material is found at the road edge indicate the problem and highlight the requirement for proper construction quality control.

Note that where a nominal overlay thickness is proposed, i.e 80mm, this is the minimum thickness of bituminous material to be placed. In order to maintain the crossfall across the road width it may be necessary to place additional material in the centreline of the road.

5 Review of Pavement Design

5.1 Geotechnical Results

According to the KOCKS Engineering Report (October 2002) geotechnical and pavement structure surveys were conducted and the following are the type and extent of the surveys.

5.2 Construction material results

The existing construction materials appear to be comparable with modern working practices although excessive coarse material is present in the bituminous and granular layers. The suitability of the current construction materials has been discussed in Section 4. Due to the quality of the likely supplies of construction aggregate, it is recommended that the minimum overlay requirement be increased from 40 to 50-60mm. Where an overlay thickness is specified, it is the minimum that is to be applied across the width of the carriageway and not that applied at the carriageway centreline

The preparation of new materials should be carefully supervised and controlled in order to ensure that only quality products with optimum load bearing characteristics are used.

5.3 Materials Availability, Haulage and Costs

The availability of suitable material cannot be checked during the time constraints of this report. However, the KOCKS report identifies at least four possible locations for aggregate and according to the available laboratory tests results (repeated in Appendix 2.4 Tables 10 and 11) these are suitable for use in road construction. The condition of the current road indicates that material of sufficient quality and suitability has been available in the past. The distance from site is typically less than 1.5km.

It is considered unlikely that between the composition of the KOCKS report (October 2002) and this review the information from the identified resources will have changed significantly. It is unlikely that there will be a significant changes to local materials in terms of availability or cost.

The KOCKS Report indicates that bitumen can be produced locally in the state capital Baku (400km) although it may have excessive paraffin content. The contractor should attempt to source a bitumen of better quality. Marshall stability tests should be conducted in order to confirm that the proposed construction material is of the best quality possible taking into account the source of aggregate and bitumen.

5.4 Axle load Survey

The results from the axle load survey (1998 to 1999) are reported in Page 5 Table 2.2 of the KOCKS Report. The reported typical axle weight is in the range of 4 to 5 tonnes for all vehicles apart from large buses which have an axle weight of 8.09 tonnes. The legal maximum in Azerbaijan for a road axle is reported to be 9 tonnes. Typically the legal maximum internationally is 8 tonnes. Note that a typical 2 axle truck in Azerbaijan would has an approximate gross weight of 5-6 tonnes and is able to carry a 6 tonnes payload. The figures supplied imply that the average payload on the section is less than 1 tonne which is inconsistent with observations made on site during the August visit.

The accuracy of the axle weight survey could be tested by conducting a road side survey of the cargo weights of trucks according to their documentation in conjunction with the local police check point.

5.5 Traffic Volume and Composition (including Directional Analysis)

The traffic volumes and therefore the overall design traffic figures are split into two sections. The first is from Shemkir to Tovuz and the second from Tovuz to Gazakh. The average traffic volume for Tovuz to Gazakh is taken from the count station at KP 438 between Tovuz and Gazakh and is 2,400 vpd in 2001. It has been impossible to establish whether the reported levels of traffic are single or bi-directional figures.

The traffic volume for Shemkir to Tovuz is taken as an average from 2 count stations at KP 280 between Yelakh and Goran and KP320 between Goran and Ganja. The section under consideration is approximately 80 km beyond the second count station and is past the major conurbation of the regional headquarters of Ganja. The origin destination surveys conducted as part of the survey indicate that approximately 12% of traffic surveyed was on a journey that stopped in Ganja from the area around the state capital, Baku. This agrees with the visual assessment of traffic on the scheme under consideration compared to the route between Baku and Ganja. The use of the traffic counts from KP280 and JP320 are therefore likely to over estimate the levels of traffic between Shamkir and Tovuz.

It is recommended that the volumes of design traffic are confirmed by a random manual count along the section. In addition the mix of vehicle types can be confirmed at the same time. Subsequent to the visit by the Pavement Engineer, a 12 hour survey of the traffic flows was conducted Jacobs request as detailed in Appendix A. The single direction design traffic for the section was calculated to be approximately 9.0 million standard axles using the revised equivalency figures for axle weights.

The provision for creating two more lanes on this section of road to rise to dual carriageway standard has been discussed. Typically traffic levels would have to increase to approximately 20,000 to 25,000 vehicles per day before the upgrade to a dual carriageway would be deemed to be appropriate. The current level of traffic at 2,000-3,000 vpd indicates that there is currently no requirement for the provision of a dual carriageway along this section.

Although the volumes of traffic in both directions are approximately the same it has not been possible to compare the weights of vehicles in each direction. In addition, once dualling of the section has been completed the levels of heavy traffic in Lane 2 of the then dual carriageway will be considerably less than in Lane 1 (typically 60% of HGV's will travel in Lane 1). The design outlined in the KOCKS report does not take into account the different levels of traffic in each lane post-dualling and therefore the level of rehabilitation that is currently required. This may be because the proposed date for the dualling has not been confirmed and it is more conservative to design the existing rehabilitation assuming long term utilisation of the road as a single carriageway.

The level of traffic growth has been assessed using the Azerbaijan National Gross Domestic Product (GDP) as an indication of the likely growth in commercial traffic.

The report was published in October 2002 when the effects of the World-wide downturn in trade could not be properly assessed. The assumptions behind the traffic model should be revisited bearing in mind the latest economic outlook of the World economy and that of Azerbaijan.

5.6 Future Maintenance Requirements and Costs

The future maintenance requirements are difficult to assess in light of the concerns about the proposed design traffic and the economic model used to predict future traffic growth.

The relatively good visual condition of the current pavement would indicate that if quality materials and modern construction methods are used during rehabilitation work the underlying pavement will perform well structurally. The surface will need to have regular maintenance in order to ensure that moisture does not enter the unbound layers which will accelerate the deterioration of the pavement structure. A surface seal will be required in 5-7 years time with possibly a replacement of the wearing course in 10-14 years time if the structure of the pavement remains in good condition.

To prevent premature failure of the pavement it is essential that the drainage, both surface and sub-soil, is maintained in optimum working condition. Side drains and carriageway shoulder should be cleared of vegetation on an annual basis. Blocked drainage culverts and lateral drains where the longitudinal profile is flat should be maintained in good working order. The controlling organisation's attitude to regular low-cost on-going maintenance compared to high-cost major rehabilitation will dictate the level of serviceability of any road section on a long-term basis.

5.7 Horizontal Alignment

The horizontal alignment has followed the existing carriageway. Limited lengths are to be re-aligned to bring the route in line with applicable standards for a single carriageway. The impact in the future of the proposal to upgrade the section to a dual carriageway and the corresponding impact on the alignment has not been considered. Dual carriageways typically have horizontal curves with much larger radii than single carriageways. The horizontal radii for single carriageways are often determined to dissuade drivers from overtaking manoeuvres around bends, whereas this requirement is no longer valid for dual carriageways. The different horizontal design parameters have not been considered in the KOCKS report.

6 Other observations

The following are additional observations on the two documents KOCKS Engineering Report and Bidding Documents which are outside the main scope of this report but have come to the attention of the Pavement Engineer during his review.

6.1 Longitudinal Profile

There are long sections with a flat longitudinal profile e.g. Chainage 15.400 to 15.900 and 43.300 to 43.700 gradient 0.358%, 45.2 to 45.500 gradient 0.055%. A minumum of 0.5% is often used internationally in order to ensure free draining in the longitudinal direction of not only the road surface (which also would normally have a cross fall of 1.5%) but also the drains at the side of the road. Although culverts are provided at either end of such sections there are no provisions for draining water away from the road structure.

Sections where the longitudinal profile has a gradient of less than 0.5% are unlikely to have drain which empties rapidly and provision for the lateral movement of water away from the pavement structure should be provided in order that the future maintenance cost is reduced. Long-term vegetation growth on the road shoulder in such areas may also prevent effective drainage and should be cut-back on a regular basis during maintenance operations. A section on the current alignment at Site Chainage 48.6 to 49.7km has failed in the past and been recently patched and the underlying problem is probably insufficient longitudinal profile and associated drainage problems.

Improvements in the vertical alignment will require the current level of the pavement surface to be raised or lowered. Where the level is to be raised by less than 200 mm it may be possible to achieve this by the addition of bituminous material. Where the level is greater than 200mm, additional granular material will be required to maintain an economic design. This should not be placed directly on top of the existing bituminous layer. Water will not be able to freely drain through the bituminous layer leading to saturation of the granular layer and poor load supporting performance and accelerating pavement deterioration. Similarly the design of the shoulders should allow for free drainage of both granular and sub-base layers.

Where the vertical alignment requires that the level of the road is reduced, it will be necessary to ensure the placement of both a new granular sub-base layer and bituminous surfacing material (minimum thickness 200mm sub-base and 200mm bituminous material). The vertical design could be improved by minimising the length of sections where the level of the road is reduced as this would also reduce the amount of new material that would have to be placed.

The long term proposal to upgrade to a dual carriageway and the associated increase in carriageway surface will exacerbate the problem of poor drainage.

6.2 Details of Changes in Construction

The proposed long sections in the bidding documents appear to be inconsistent. In one case at Chainage 18.650 the long-section details a constant downward slope of 1.484% with a change from 80 to 120mm overlay mid way down the slope. Any such change will require a localised change in slope in order to feather out the transition and will lead to an uneven vehicle ride which may cause localised failures due to axle bounce of heavy vehicles. Transitions in levels of overlay requirement could be more easily accommodated at changes in gradient of the longitudinal profile

6.3 Report Layout

The sections contained with in the report and the data from different surveys has not be collated in a logical manner. The geotechnical surveys have been extensive. However the results from the FWD survey have been used without comparison to either other surveys or the visual condition of the road.

7 Comparison with International Design Standard

In order to compare the rehabilitation proposals including the construction of the re-aligned and/or reconstructed sections of the scheme, the UK TRL Overseas Road Note 31 was used to prepare a design by Jacobs see Appendix A of this Report.

The design assumptions are presented in Appendix A of this Report were as follows for the entire length of the scheme.

- Design 16.5 msa over a design period of 20 years
- Subgrade strength Ch. km 0.000 to 40.200 CBR 2% Ch.40.200 CBR 5-7%
- Semi-structural Surface and bituminous roadbase

For Chainage km 0.000 to 40.200, the required construction will be 225mm of bituminous material over 225mm of granular sub-base over 350mm of capping. The extent of the section with a stiffer subgrade of CBR 12% should be re-established on site in order to utilise a pavement design with a reduced granular material thickness (approximately 200mm of sub-base).

From Chainage Km 40.200 the required construction would be 225 mm of bituminous material over 275 mm of granular sub-base.

In each case the bituminous material should consist of a 50mm wearing course and 150mm base.

The existing thickness of bituminous material is approximately 100mm at the road edge and 170mm in the carriageway according to the trial pits in the KOCKS Report. In order to strengthen the entire cross-section and prevent premature failure of the road edge, as observed currently, it would be necessary to place a minimum of 125mm across the width of the road. Areas of severe deterioration, such as pot-holes, edge cracking or crocodile cracking should be broken out and replaced prior to the overlay process.

A reduced overlay thickness may be appropriate where the current construction is thicker than 100mm across the entire road. Prior to the placement of a reduced bituminous overlay the current condition of the pavement should be assessed in order to confirm that the underlying layers of the current pavement are performing satisfactorily, ie. There is good drainage and little surface deterioration.

Sufficient repairs to any existing deterioration should be conducted prior to overlay operations. Where there is observed edge deterioration to the bituminous surface it is essential that a full thickness of new bituminous material is placed. Drainage paths in the pavement structure should be maintained in order to ensure adequate structural performance.

8 Conclusions

The visiting Jacobs Pavement Design Expert, Dr Michael Heelis, conducted a site visit to the Rehabilitation and Upgrading of the Shemkir to Gazakh Road section in August 2003. A visual inspection of the site was performed and the Engineering Report and Bidding documents prepared by KOCKS CONSULT GMBH were reviewed.

A number of inconsistencies were observed in the report some of a major nature. The conclusions from the report and the design for the rehabilitation works rely heavily on a FWD survey conducted in June 2001. The FWD analysis appears to be inconsistent with two design temperatures and reported bituminous stiffnesses for aged material in excess of those achievable. Analysis of the grading from a wearing course has been compared to that of a base course. Traffic figures and axle loads have been used which may not be applicable to the scheme under consideration.

The structural data cannot be re-analysed in the time frame before the start of construction and therefore it is recommended that a design based on existing International Design standards and the existing geotechnical survey are implemented.

Despite the above observation, the design in the KOCKS Engineering Report is comparable to International Design Standards, providing:-

- The extent of the section with a design subgrade CBR of 12% is re-established.
- The level of design traffic is established.

In addition, the following observations are made on the recommendations contained in the KOCKS Report:-

- Where reconstruction of the existing alignment has been recommended and there is little visual deterioration the existing overlay recommendation may be over-conservative depending on the design traffic.
- Where an overlay of less than 100mm has been recommended, it should be confirmed that the drainage conditions of the pavement foundations are sufficient and the pavement is currently in a good visual condition.
- A lack of drainage ditches and poorly maintained culverts, as well as a longitudinal profile with insufficient gradient (<0.5%), have attributed to the poor drainage condition and deterioration of the existing road pavement and should be rectified.
- Marshall Stability testing of the proposed bituminous mixture should be conducted to ensure an optimum design.
- Quality control on site should be carefully supervised.

The current level of traffic (two-way 2,000 to 3,000 vehicles per day) indicates that the section will not require upgrading to a dual carriageway during the current pavement design life (15 to 20 years).

Appendix A - Independent Pavement Assessment

A road –side traffic count was conducted on the instigation of the Pavement Engineer and reported to the Jacobs Project Manager in the United Kingdom on the 11^{th} August 2003.

An additional cause of concern regarding the analysis of the axle weights is contained Table 2.2 where the equivalency factors appear to be calculated using average truck weights. According to TRL ORN 31 the calculation as performed in the report of determining the equivalency factor from the average axle load is incorrect and leads to large errors. This erroneous methodology has been adopted in the KOCKS report to calculate the design traffic.

The equivalency factors for each type of vehicle can be calculated on the legal maximum in Azerbaijan which is a 9 tonne axle. Typically where axle weights are poorly policed the observed axle weights are in excess of legal requirements especially when goods are being moved internationally. Each single axle of 9 tonnes is equivalent to approximately 1.5 standard axles. In this case the equivalency factors for Bus, 2 axle, 3 axle, 4 axle and 5 axle trucks would be 3.0, 3.0, 4.5, 6.0 and 7.5 respectively.

The total 2-way traffic flows over a period 8am to 8pm (between 7^{th}-10^{th} August 2003) was evaluated at a point at Chainage km421.0 (Site Chainage km30.2). The composition of the traffic with approximately 80-85% of light traffic is consistent with the traffic surveys reported in the KOCKS Report.

Vehicle Type	2-way Traffic Flow	Percentage of total
Cars	1930	70.9
Pick-ups	291	10.7
Buses	45	1.7
Motorcylces/Tractors	62	2.3
Tricks 2 Axle	201	7.4
Trucks 3 axle	134	4.9
Trucks 4 axle	34	1.2
Trucks 5 axle and over	26	1.0
TOTAL	2723	100

Note: Buses total is low, because buses mostly travel at night.

The following assumptions were then used to provide a value for the design traffic over a 20 year period.

- A multiplier of 1.3 (3.0 for buses) was used to provide a 24 average traffic count
- The equivalency factors based on the maximum legal axle limit was used
- A design Period of 18 years was used (as per KOCKS Report)
- A Summer Season factor of 0.94 was used to take into account annual variation of traffic. (Source KOCKS Report)
- The equivalency factors for Cars, Pick-ups and Motorcycles/Tractor was assumed to be negligible compared to other vehicle types.
- The survey site was at Chainage km30.2 which was between Shemkir and Tovuz. The level of traffic between Tovuz and the end of the site at Gazakh should also be confirmed. A significant reduction in the number of

vehicles with heavy axles was not observed between the two sections although the KOCKS Report indicates a reduction from 3000 to 2000 vehicle per day.

Vehicle Type	2-way Traffic Flow	24 hour traffic	Equivalency Factor	Standard Axles	
Buses	45	135	3	405	
Tricks 2 Axle	201	261	3	783	
Trucks 3 axle	134	174	4.5	783	
Trucks 4 axle	34	44	6	264	
Trucks 5 axle and over	26	33	7.5	247	
			TOTAL	2482	Standard axles/day

A 20 year design traffic would be based on the reported level of traffic multiplied by the seasonal factor (0.94) for 365 days a year over 20 years in each direction. The design traffic would therefore be calculated as 8.5 million standard axles.

The calculation does not take into account a growth factor as the 8 fold increase in traffic in 20 years reported in the KOCKS report would appear to be over optimistic. An increase in traffic to 24,000 vehicle per day would require the route to be upgraded to a dual carriageway dependent on the level of service required and additional pavement design would be required once this traffic level has been exceeded. An increase in the traffic of approximately 6% per year would lead to a 3-4 fold increase in the traffic over 20 years and a corresponding design traffic of **15.6 million standard axles**

At a level of 15.6 msa, the section would be rated a T 7 according to TRL Road Note 31. According to the TRL design guide, the bituminous thickness required will be a minimum of 225mm (surface course and road base).

The subgrade CBR strength of 2% has been established for the section Chainage km 0 to 40.2. The required thickness of granular material would typically be 225mm of sub-base over a 350mm capping layer. Where the CBR of the subgrade is increased to 12%, the required thickness could be decreased to 200mm of sub-base. The extent of this stronger subgrade has not been established in the KOCKS report.

For Chainage km 40.2 onwards the CBR of the subgrade is 4-5% and the required thickness of sub-base would be 225mm with a 200mm capping layer.