AECOM

Wick Airport – Runway 13-31 and Disused Runway 08-26

Pavement Evaluation Report

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EXECUTIVE SUMMARY

AECOM have been commissioned to carry out a pavement evaluation survey of Runway 13-31 and part of disused Runway 08-26 at Wick Airport. in order to assess the pavements' suitability for [redacted] Boeing C17A Globemaster [redacted]

This evaluation comprised a desktop study, fieldwork including non-intrusive (Falling Weight Deflectometer [FWD] and Ground Penetrating Radar [GPR] testing) and intrusive (core sampling and Dynamic Cone Penetrometer [DCP] testing) surveys and laboratory testing of recovered materials. This work was supplemented by analysis of survey data and provision of an interpretive report detailing pavement condition and capacity and proposed pavement strengthening recommendations.

As part of the interpretative reporting, all investigated pavements were grouped into five Characteristic Sections (A-E) of which two were further subdivided (A1, A2 and C1, C2).

Two methods of analysis were employed to assess the pavement condition and evaluate pavements suitability for Boeing C17 loading. The Pavement Classification Number (PCN) – Aircraft Classification Number (CAN) method as outlined in DMG27, was employed to review the suitability of existing pavements for overloading operations.

In addition, the BAA analytical method was utilized to calculate the pavements' structural capacity to support the proposed Boeing C17A movements, as well as evaluate remaining life of pavements post C17A loadings to ensure adequate capacity for anticipated future traffic volumes.

The analysis indicated the following:

- The majority of Runway 13-31 length (Characteristic Sections A2 and B) was found to have adequate Pavement Classification Number (PCN) for operations of BAe 146-100 as a design aircraft at the anticipated future traffic volumes.
- With the exception of Characteristic Section A2, the Runway 13-31 Pavement Classification Number was found to be insufficient to support the proposed number of Boeing C17 overload movements.
- Where PCN was insufficient at Runway 13-31, analytical calculation suggested limited pavement damage for all but Characteristic Sections C1 and C2 (at rigid/flexible pavement transitions) [redacted]
- The pavement bearing capacity of Characteristic Sections C1 and C2 is not sufficient to carry C17A movements.
- The disused Runway 08-26 pavement was revealed, with the exception of the first 75m of the survey extents (Characteristic Section D), to be heavily damaged and of apparent insufficient structural capacity to support proposed loading.
- The PCN-ACN analysis confirmed all Runway 08-26 pavement to not meet PCN requirements for either normal operation with BAe 146-100 as a design aircraft or Boeing C17A overloading.
- The analytical calculation suggested Characteristic Section D to be structurally sufficient with limited damage.

Analytical evaluation therefore revealed that three Characteristic Sections (C1, C2 and E) across both runways to have insufficient structural capacity [redacted] thus requiring structural strengthening.

Whilst full depth reconstruction to 275mm depth is recommended prior to C17A trafficking at Characteristic Section E on Runway 08-26, it is recommended that the C17A aircraft be allowed to traffic Characteristic Sections C1 and C2 on Runway 13-31, allowing the asphalt to crack, before inspection, evaluation and full reconstruction are undertaken.

Although remaining pavement areas are theoretically structurally sufficient so as to prevent permanent deformation of the subgrade, cracking of the concrete/asphalt material may be expected under the overload operations, but not so as to be classed as pavement 'failure'. Routine inspection and maintenance is therefore recommended.

In addition to structural treatment, it is highly recommended to carry out inspection prior and after each Boeing C17A movement by a competent pavement engineer.

1 INTRODUCTION

1.1 General

In September 2015, Highlands and Islands Airports (HIAL) approached AECOM requesting an advice on suitability of pavement structures at Wick Airport to accommodate

This report describes a pavement evaluation, undertaken by AECOM, of the selected pavement structures at Wick Airport.

The evaluation comprised the following areas:

- Runway 13-31;
- Northern part of disused Runway 08-26.

The exact extents of the site investigation are presented in Figure 1.

1.2 Purpose of the Evaluation

The purpose of this evaluation was to:

- Assess the condition of the existing pavement layers and foundation;
- Determine the current Pavement Classification Number (PCN) for the existing pavements in accordance with DMG27[1];
- Assess pavement capacity to support the anticipated Boeing C17A movements and evaluate remaining life of the pavements post Boeing C17 trafficking;
- Provide strengthening treatment recommendations to permit the desired trafficking of the airfield pavements;

1.3 Elements of the Evaluation

The evaluation comprised the following elements:

- Desktop Study;
- Structural evaluation using Falling Weight Deflectometer (FWD);
- Rotary Coring survey;
- Dynamic Cone Penetrometer (DCP);
- Ground Penetrating Radar (GPR);
- Laboratory based material testing on the extracted core samples;
- Analysis of all survey data obtained from this evaluation (in conjunction with traffic information) in order to achieve the objectives listed in Section 1.2.

[1] A Guide to Airfield Pavement Design and Evaluation, 3rd Edition, Defence Estate, February 2011

2 FIELDWORK

2.1 General

All the fieldwork was performed during two night-time shifts from the 22nd to the 24th of September 2015 between 21:00hrs to 06:00hrs.

The survey extents and site chainage referencing system, used for all survey activities discussed in this report, are detailed in Table 1 and Figure 1. The chainage system was set out along the marked centerline during the FWD survey, using the linear distance measurement instrument mounted on the FWD.

Table 1: Survey extents

LOCATION	DESCRIPTION OF START POINT	DESCRIPTION OF END POINT	LENGTH[1] (M)
RWY 13-31	West edge of Runway 13-31	East edge of Runway 31-13	1832
RWY 08-26	East edge of Runway 13-31	Plastic bollards positioned approximately 300m away from start point	300

Notes:
[1] Centreline length as measured during the FWD survey
RWY Denotes Runway

2.2 Falling Weight Deflectometer

AECOM used a Falling Weight Deflectometer (FWD), serial number 8002-192, to perform the deflection testing.

All AECOM FWDs undergo yearly servicing and manufacturer calibrations; attend the yearly UK Highways England sponsored Correlation Trials and have regular Relative Calibrations of the geophones to ensure inter-machine consistency.

The FWD survey was performed on all pavement areas within the scope of this evaluation. In addition to the run along the centerline, six other lines running parallel to the centerline were surveyed on each runway. The test spacing was staggered between adjacent offsets to give greater coverage.

The offsets are referenced left and right of the centerline relative to the direction of increasing survey chainage.

Details of the nominal FWD test spacing adopted for each pavement and survey line are presented in Table 2.

Table 2: FWD offsets and nominal test spacing

LOCATION	OFFSET FROM THE CENTRELINE [1]	NOMINAL TEST SPACING [2] (M)
RWY 13-31	0m	100m
	4m Left and Right	25m
	10m Left and Right	25m
	20m Left and Right	50m
RWY 08-26	0m	20m
	4m Left and Right	20m
	10m Left and Right	20m
	20m Left and Right	20m

Notes:
[1] Runway offsets referenced to Left and Right of centreline corresponding to the direction of increasing survey chainage.
[2] Test spacing dependant on slab positioning and pavement length.

Three loading drops were performed at each test location. A nominal test pressure of 1.0MPa with a 300mm diameter loading plate was applied during the FWD testing for all locations.

On rigid pavements, where the condition of the concrete layer is the primary concern, DMG27 recommends the FWD testing to be carried out at temperature below 15°C. The measured temperatures at each shift are shown in Table 3.

Table 3: Summary of recorded site temperatures

SHIFT	LOCATION	DATE	MEAN TEMPERATURES (°C)		
			AIR	SURFACE	100MM
1	Runway 13-31	22/09/2015	13.7	13.5	12.8
2	Runway 08-26	23/09/2015	14.1	13.9	13.1

2.3 Coarse Visual Survey

A course visual survey was performed during the FWD testing to assist in the assessment of the pavements. It should be noted that these observations were made during the hours of darkness under limited vehicle/mobile lighting and are not definitive. The following paragraphs summaries the findings.

2.3.1 *Runway 13-31*

The visual assessment revealed the runway pavements to be of flexible construction with concrete ends. Apart from evident spalling, no major surface distresses could be identified across the concrete sections, suggesting the concrete is performing well.

The flexible sections were found to be in good condition throughout. No major distresses could be identified.

2.3.2 *Runway 08-26*

The surveyed section was found to be of flexible construction. With the exception of chainage 0-75m, the pavement exhibited heavy damage and disintegration. Patches of vegetation were observed across the runway between 75m and 300m, suggesting material deterioration at depth.

2.4 Coring Survey

A Rotary Coring survey was performed to extract samples of the bound layers in order to determine the construction depths and confirm material types, at discrete locations, and provide samples for laboratory testing. A hydraulically operated coring trailer was employed, using water cooled diamond tipped core barrels of nominal diameter 162mm to provide a core sample of nominal diameter 150mm.

A total of 16 cores were taken across all the pavements included in this evaluation. The locations were selected to provide a representative number of samples across all investigated areas. Core locations are presented in Figure 2.

A site core log including locations details, approximate measurements, condition and site photograph, was completed for each core upon extraction. The cores where placed in individually labelled bags and then returned to AECOM's laboratory in Nottingham, UK for logging and laboratory testing.

As part of the logging process a PAK Marker spray test is performed on all asphalt materials present. This is used to initially identify the presence of poly-aromatic compounds typically found in tar. The spray test is not a definitive indicator of the presence of poly-aromatic compounds, which can be found in other pavement construction materials at lower concentrations, but it is a strong indicator of tar materials.

Core details are summarised in Table 4 and presented in Appendix B.

Table 4: Summary of core data

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	MATERIAL THICKNESS (MM) [1]	
				CONCRETE	ASPHALT
RWY 13-31	35	4m Left	1	290 V	-
	165	10m Right	2	285 V	-
	435	4m Left	3	-	325 V, T
	600	4m Right	4	-	325 V, T
	750	10m Left	5	-	310 V, T
	900	4m Left	6	-	365 V, T
	1050	10m Right	7	-	290 V, T
	1270	4m Left	8	-	305 V, T
	1490	20m Right	9	-	315 V, T
	1650	4m Left	10	-	260 V
	1770	4m Right	11	300	-
	1810	10m Left	12	315 V	-
RWY 08-26	50	4m Left	13	-	200 V, T
	135	10m Right	14	-	90 BU, V, T

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	MATERIAL THICKNESS (MM) [1]	
				CONCRETE	ASPHALT
RWY 08-26	200	4m Left	15	-	100 BU, V, T
	270	4m Right	16	-	95 BU, V, T

Notes:
[1] All thicknesses rounded to the nearest 5mm.
BU Denotes Broken Up
V Denotes Voided.
T Denotes presence of Tar as indicated by PAK test.

2.5 Dynamic Cone Penetrometer Testing

A Dynamic Cone Penetration (DCP) survey was performed at each core location, after extraction of the bound layers, to enable assessment of the underlying foundation and to determine (where possible) the granular layer thickness.

The penetration depth was targeted to achieve approximately 1m below the base of the bound layer. Full penetration (considered as penetration deeper than 800mm below the base of the bound layer) was not achieved at any of the 16 locations. Partial penetrations can be related to the presence of large sized aggregate and/or a particularly strong granular material and/or upper foundation, it does not definitively indicate a high CBR value for the subgrade.

The DCP penetration rate was correlated with the California Bearing Ratio (CBR) using the relationship detailed in IAN 73/06 (2009)[2] to provide a CBR profile with depth.

DCP testing revealed the upper foundation to range between 12% and 55% across Runway 13-31 and Runway 08-26, whilst the lower foundation was varied from 9% to >100% CBR.

Revision of historical records[3] appears to suggest similar variation of average subgrade CBR values ranging between 10% and 20% across Runway 13-31 and from 7% to 20% throughout Runway 08-26.

DCP median CBR results for the upper and lower foundation layers are summarized in Table 5. The DCP profiles of interpreted depth through the unbound foundation layers are presented in Appendix C.

Table 5: Summary of DCP data

LOCATION	CHAINAGE (M)	OFFSET FROM C/L	CORE NO.	DEPTH OF PENETRATION BELOW BOUND BASE [MM][1]	INTERPRETED UPPER FOUNDATION THICKNESS (MM) [1], [2]	MEDIAN CBR (%)[3]	
						UPPER FDN.	LOWER FDN.
	35	4m Left	1	565	200	22	44
RWY 13-31	165	10m Right	2	260	100	13	20-35
	435	4m Left	3	Penetration not possible			
RWY 13-31	600	4m Right	4	170	170	>100	-

[2] Highways Agency, IAN 73/06 Interim Advice Note, Design Guidance for Road Pavement Foundations (Draft HD25), February 2011
[3] Scott Wilson Kirkpatrick, Wick Airport – Pavement Investigation Report, October 1999

LOCATION	CHAINAGE (M)	OFFSET FROM C/L	CORE NO.	DEPTH OF PENETRATION BELOW BOUND BASE [MM][1]	INTERPRETED UPPER FOUNDATION THICKNESS (MM)[1], [2]	MEDIAN CBR (%)[3]	
						UPPER FDN.	LOWER FDN.
	750	10m Left	5	170	90	45	>100
	900	4m Left	6	225	70	55	>100
	1050	10m Right	7	Penetration not possible			
	1270	4m Left	8	Penetration not possible			
	1490	20m Right	9	Penetration not possible			
	1650	4m Left	10	Penetration not possible			
	1770	4m Right	11	475	245	55	17
	1810	10m Left	12	535	185	55	9
RYW 08-26	50	4m Left	13	170	80	39	88
	135	10m Right	14	170	170	95	44
	200	4m Left	15	Not performed due to large aggregate present directly underneath the bound layers			
	270	4m Right	16	105	105	70	-

Notes:
[1] Thicknesses rounded to the nearest 5mm.
[2] All the DCPs were refused before full penetration of 800mm below the base of bound layer.
[3] The CBR value is indicative only as the DCP test had to be terminated before the full penetration depth was achieved.

2.6 Ground Penetrating Radar Survey

A slow speed Ground Penetrating Radar (GPR) survey was undertaken on both Runway 13-31 and disused Runway 08-26. The GPR survey provided continuous profiles along the pavement which enabled the layer thickness variation and possible construction changes to be mapped.

A 900MHz ground coupled GPR antenna mounted on cart was used to collect the data. The GPR survey was performed alongside the FWD runs at offset of 4m to both left and right along Runway 13-31 and disused Runway 08-26, with additional transverse profiles taken across the pavement at 200m intervals.

The GPR data is summarised in the following sections, with the construction profiles presented in Appendix H.

2.6.1 *Runway 13-31*

The material interfaces found in the GPR data are typically in agreement with the core samples. In general, the consistent GPR signal responses from bound layers confirm a relatively uniform thickness of pavement materials throughout this investigated section. A localised thinning of asphalt layer was identified at transitions between concrete and asphalt.

2.6.2 *Runway 08-26*

The material interfaces found in the GPR data are typically in agreement with the core samples. The consistent GPR signal responses from bound layers between chainage 75m to 275m confirm a relatively uniform thickness of pavement materials at this section. From chainages 0m to 75m GPR signal response appears to suggest a gradual transition of thicknesses from the edge of Runway 13-31.

3 LABORATORY TESTING

3.1 General

A programme of laboratory testing was undertaken to determine the material characteristics. The testing was performed by AECOM's in-house UKAS accredited laboratory and by AECOM approved laboratory service provider.

The laboratory test results are presented in Appendix D and summarised below.

3.2 Concrete Material

3.2.1 *Compressive Strength Testing (CST)*

The compressive strength testing of concrete was undertaken by AECOM in accordance with BS EN 12504-1[4] and BS EN 12390-1[5], 3[6] and 7[7]. A total of four intact samples (from four cores) were tested to provide a representative overview of the existing concrete strength.

Concrete is only present at the Runway 13-31 ends. The aggregate type of concrete samples at the north end (13 End) has been identified as gravel whereas the samples taken at the south end (31 End) indicate the aggregate type to be crushed rock.

The compressive strength results are summarised in Table 6.

Table 6: Summary of compressive strength testing

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	BULK DENSITY (KG/M^3)	COMP.STRENGTH (N/MM^2)	EST. IN-SITU CUBE STRENGTH (N/MM^2) [1]
Runway 13-31	35	4m Left	1	2250	30.0	30.5
	165	10m Right	2	2220	29.2	29.0
	1770	4m Right	11	2430	72.8	73.0
	1810	10m Left	12	2440	62.3	62.5

Notes:
Comp. Denotes Compressive
[1] Based on an additional calculation set out in National Annex NA (informative) guidance on the use of BS EN 12504-1:2009.

[4] BS EN 12504-1:2009 Testing concrete in structures, Cored specimens, Taking, examining and testing in compression, BSI, September 2009.
[5] BS EN 12390-1:2000 Testing hardened concrete, Shape, dimensions and other requirements for specimens and moulds, BSI, March 2000.
[6] BS EN 12390-3:2009 Testing hardened concrete, Compressive strength of test specimens, BSI, May 2009.
[7] BS EN 12390-7:2009 Testing hardened concrete, Density of hardened concrete, BSI, May 2009.

3.3 Asphalt Material

3.3.1 *Indirect Tensile Stiffness Modulus (ITSM)*

The Indirect Tensile Stiffness Modulus (ITSM) testing was performed on 12 asphalt material samples, in accordance with BS EN 12697-26:2004[8], at three temperatures (10°C, 20°C and 30°C) using the Nottingham Asphalt Tester (NAT). The results of the ITSM testing were used to support the FWD back-analysed stiffness moduli and provide information on the temperature susceptibility of the materials.

Details of the stiffness results are presented in Appendix D and a summary is shown in Table 7.

Table 7: Summary of ITSM testing

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	LAYER	MATERIAL [1]	BULK DENSITY (KG/M³)	ITSM [MPA]		
							10°C	20°C	30°C
RWY 13-31	435	4m Left	3	2	HRA	2293	3260	1240[3]	910
				4	HRA	2289	3840	2550	1310
	600	4m Right	4	4	HRA	2249	3470	2570	1100
				5	HRA	2248	4140	2550	1350
	750	10m Left	5	4	HRA	2262	1670	820[3]	360
				5	HRA	2309	3590	1930	840
	900	4m Left	6	1	HRA[2]	2289	5130	2260	670
				5	HRA	2345	3260	1620[3]	670
	1490	20m Right	9	1	HRA[2]	2310	6150	2640	800
	1650	4m Left	10	3	HRA	2302	8630	5500	2300
				4	HRA	2260	6680	4620	2560
RWY 08-26	50	4m Left	13	2	HRA	2276	2430	1350[3]	760

Notes:
ITSM Denotes Indirect Tensile Stiffness Modulus
HRA Denotes Hot Rolled Asphalt
[1] Material type determined by visual identification supported by the historical records
[2] Denotes asphalt material which may be unduly susceptible to changes in temperature.
[3] Denotes a stiffness value (at 20°C) outside the 'typical' range expected for aged asphalt material.

Asphalt material stiffness relates to its load spreading ability, one of the key parameters used to assess pavement structural condition. In a structural layer, high stiffness indicates good load-spreading ability.

As presented in Table 7 above, the stiffness data range for each material at each test temperature shows variation in the individual results at 20°C, which ranged from 820 MPa to 5500 MPa suggesting a non-uniform pavement in terms of material properties with potential for development of isolated defects.

[8] BS EN 12697-26:2012 'Bituminous mixtures, Test methods for hot mix asphalt, Stiffness, BSI, March 2012.

It should be noted that aged material may yield higher ITSM values due to binder hardening and be more susceptible to fatigue cracking. Typical asphalt stiffness values in the UK for Hot Rolled Asphalt (HRA) range between 1750-4500MPa (for 100-50 pen, aged material) at the design temperature of 20°C.

With the exception of Core 9 and Layer 1 of Core 6, all material stiffnesses at 30ºC retained at least 40% of their stiffness measured at 20ºC. This suggests the material is typically non-susceptible to temperature variation. From general experience, well performing asphalt materials would be expected to retain at least 35% of their stiffness over this temperature range.

3.3.2 *Polycyclic Aromatic Hydrocarbons (PAH) Testing*

A PAK-Marker (PAH spray) test was used during asphalt core logging process to determine if Polycyclic Aromatic Compounds (PAC's), typically found in tar, are present in the binder of each asphalt layer. The PAK-marker is sprayed in a thin layer along the length of the core sample and after drying out the colour is assessed. No colour changes indicate bitumen whilst a change in colour towards yellow may indicate the presence of PACs.

The presence of PAC's is of particular importance if the material is to be disturbed, either for recycling or disposal purposes. Further analytical testing, such as the total Polycyclic Aromatic Hydrocarbon (PAH) test by using a Gas chromatography flame ionization detector (GC-FID) is required to confirm a positive result and determine the precise quantities of volatile compounds present in the binder.

Asphalt found to test positive for the possible presence of PAC's using the PAK-Marker test during the core logging process, was selected for PAH testing, in order to confirm the presence of tar and determine concentration levels. The level of Benzo(a)Pyrene and total PAH are of particular concern due to their known or suspected carcinogenity.

A total of six samples from 16 cores were selected for PAH testing, results of which are summarized in Table 8. The detailed analytical test results are presented in Appendix D.

Table 8: Summary of chemical analysis PAH tests

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	LAYER	SUMMARY OF PAH CHEMICAL ANALYSIS	
					BENZO(A)PYRENE (MG/KG)	PAH (MG/KG)
Runway 13-31	435	4m Left	3	7	<0.1	<1.6
	600	4m Right	4	7	10	250
	750	10m Left	5	6	<0.1	<1.6
	1490	20m Right	9	6	11	110
Runway 08-26	135	10m Right	14	2	<0.1	28
	270	4m Right	16	3	**120**	**2100**

Notes:
[1] PAH denotes Polyaromatic Hydrocarbons
Red Bold text denotes exceeds acceptable threshold values

Acceptable threshold values for Benzo(a)Pyrene and PAH are 100mg/kg and 1000mg/kg, respectively[9]. The Benzo(A)Pyrene and PAH levels measured in Core 16 Layer 3, extracted from disused Runway 08-26,

[9] Wilkinson, D., O'Farrell. D., Symonds, C., Road Materials Containing Tar, Asphalt Professional, No36, January 2009.

exceed these thresholds. If disturbed, either for the rehabilitation of the taxiway or for other purposes, this material would attract a premium disposal cost due to the tar based constituents identified compared to non-tar based asphalt planings. In addition, the options to re-use the asphalt planings from this layer(s) is limited to cold mix recycling which would encapsulate the tar, hot mix recycling would not be permitted as this will release the tar compounds.

3.3.3 *Waste Acceptance Criteria (WAC) Tests*

If asphalt material identified as exceeding PAH thresholds is to be disposed of, further testing is required in order to classify the material according to the Waste Acceptance Criteria (WAC). The Landfill (Scotland) Regulations 2003 and subsequent amendments introduced in 2003 and 2013 states the criteria for different WAC for Invert Waste, Stable Non-Reactive Hazardous Waste (SNRHW) and Hazardous Waste. The WAC test involves applying an acceptance leaching test which requires the taking of a representative sample of waste and subjecting it to leaching in water under specific test conditions. The WAC test looks at several criteria when classifying the waste material including Total Organic Carbon (TOC), Total Benzene/Toluene/Ethylbenzene/Xylenes (BTEX), Total PAH and Total Petroleum Hydrocarbons (TPH).

A total of two samples were selected for WAC testing, results of which are summarized in Table 9. The detailed analytical test results are presented in Appendix D.

Table 9: Summary of chemical analysis WAC tests

LOCATION	CHAINAGE (M)	OFFSET FROM CENTERLINE	CORE NO.	LAYER	DEPTH TO TOP OF LAYER (MM)	IDENTIFIED BY THE SPRAY TEST	TOC [1] (%C)	TOTAL BTEX [2]	TOTAL PAH [3] (MG/KG)	TPH [4]
Runway 08-26	50	4m Left	13	3	13	Yes	2.30	0.04	2	**14000**
	200	4m Left	15	3	50	Yes	1.30	0.04	**7300**	**36000**

Notes:
[1] TOC denotes Total Organic Carbon
[2] BTEX denotes Benzene, Toluene, Ethylbenzene and Xylenes
[3] PAH denotes Polyaromatic Hydrocarbons
[4] TPH denotes Total Petroleum Hydrocarbons
Black Bold text denotes exceeds Inert Waste
Red Bold text underlined denotes exceeds Hazardous WAC

Both tested samples were found to exceed limits for Inert Waste. However, the limits for SNRHW and/or Hazardous Waste have not been exceeded by any of the tested specimens.

4 FWD DATA ANALYSIS

4.1 Interpretation of FWD Data

The FWD testing was performed using a 300mm diameter loading plate and geophones located at standardised pre-determined radial distances of 0m, 0.3m, 0.6m, 0.9m, 1.2m, 1.5m and 2.1m from the centre of the applied load.

Due to variations in the pavement response, the contact pressure applied by the FWD varies slightly from test to test. In order to compare test points for this survey, the data was normalised to the target contact pressure of 1MPa. The normalised deflection readings are tabulated in Appendix E. The key deflection parameters, which are used in the back-analysis in order to calculate the pavement layer stiffness performance, are described in Table 10.

Table 10: Summary of key deflection parameters for back-analysis

FWD DEFLECTION PARAMETER	PERFORMANCE INDICATOR
Central Deflection (d1)	Overall pavement response
Deflection Difference (d1-d4)	Response of all bound layers
Outer Deflection (d6)	Foundation response

Profiles of the key deflection parameters have been plotted against chainage and are shown in Appendix F. In general, higher deflections indicate poorer performance and/or thinner pavement layers, with 'peaks' indicating distressed areas or cracks. A study of the deflection profiles enables the relative condition of the foundation and pavement layers to be assessed qualitatively.

The deflection profiles appear to indicate that the bound pavement layers generally have the greatest influence on the overall pavement performance. This is shown by all bound layers parameter (d_1-d_4) typically mirroring the overall pavement response parameter (d_1).

On Runway 13-31 higher deflections are generally located at transitions between flexible and rigid construction. In addition some localised deflection peaks are evident around survey chainage 0m and beyond 250m at the 4m to the right from centreline offset.

Consistently high deflections are evident throughout entire Runway 08-26 section with the highest deflections recorded from chainage 75m onwards.

4.2 Characteristic Sections

The pavements were divided, alphabetically, into 5 Characteristic Sections based on:

- Structure type, e.g. flexible or rigid;
- Average material layer thicknesses.

As indicated in Table 10, characteristic sections A and C were further divided, numerically, into sub-sections based on:

- Pavement response to FWD loading based on the deflection profiles; typically the overall pavement deflection response indicated by d1;
- Variation in construction materials properties;
- Location and associated likelihood of different structure properties.

The Characteristic Sections identified are detailed in Table 11, together with their extents, and the average interpreted layer thickness used for the back-analysis procedure. The extents of each section are presented schematically in Figure 2.

Table 11: Characteristic Sections

LOCATION	CHARACTERISTIC SECTION	SECTION LINEAR LENGTH (M)	CHAINAGE (M)		INTERPRETED LAYER TYPE AND THICKNESSES [1] (MM)	
			FROM	TO	PQC	ASPHALT
Runway 13-31	A1	274	0	274	300	-
	A2	91	1741	1832	300	-
	B	1390	310	1741	-	310
	C1	36	274	310	-	150
	C2	41	1700	1741	-	150
Runway 08-26	D	75	0	75	-	200
	E	225	75	300	-	100

Note:

[1] Thicknesses rounded up to the neared 5mm.

A summary of the 50th (median level) and 85th (i.e. only 15% of the deflections were found to be higher) percentile FWD deflection parameters for each section are shown in Table 12.

Table 12: Normalised deflection parameters

LOCATION	CHARACTERISTIC SECTION	DEFLECTION PERCENTILE	INTERPRETED LAYER TYPE AND MEAN DEFLECTION PARAMETERS (MM X 10^{-3}) [1]		
			OVERALL	BOUND LAYER(S)	FOUNDATION
Runway 13-31	A1	85th	261	122	97
		50th	153	58	54
	A2	85th	199	85	72
		50th	130	48	54
	B	85th	287	206	41
		50th	230	155	31
	C1	85th	582	481	36
		50th	464	353	24
	C2	85th	563	459	33
		50th	460	362	29

LOCATION	CHARACTERISTIC SECTION	DEFLECTION PERCENTILE	INTERPRETED LAYER TYPE AND MEAN DEFLECTION PARAMETERS (MM X 10^{-3}) [1]		
			OVERALL	BOUND LAYER(S)	FOUNDATION
Runway 08-26	D	85th	590	463	40
		50th	417	314	29
	E	85th	1404	1364	25
		50th	1095	1008	11

Notes:
[1] FWD deflections have been normalised to a contact stress of 1MPa

4.3 Back-Analysis of FWD Data

All the points obtained from FWD testing were included in a detailed back-analysis procedure to determine the effective stiffness of the pavement layers. The analytical method used is outlined in Appendix A.

DMG27 gives guidance and limitations on the complexity of the pavement structure for analysis. This includes modelling all asphalt layers as one combined layer, having a maximum of four layers including the foundation layer and a minimum layer thickness of 75mm.

A two-layer pavement structure was used to analyse all Characteristic Sections; comprising the concrete or asphalt as layer 1 (depending on the section) and the foundation as layer 2.

The FWD data back-analysis software Elmod 6 was used to derive the pavement layer stiffnesses for each test location based on the pavement layer thicknesses detailed in Table 11. The back-analysis was carried out using Linear Elastic Theory which treats the foundation as a linear material, as specified in DMG27.

4.3.1 *Back-analysed Material Stiffness*

The back-analysed stiffness results are presented in Appendix G in the form of correlations between the effective layer stiffness obtained from the back-analysis and the appropriate deflection parameter for each Characteristic Section. A summary of the 50th and 15th percentile material stiffnesses, corresponding to the 50th and 85th percentile deflection levels of the appropriate deflection indicator, respectively, is presented in Table 13 for each characteristic section.

Table 13: Back-analysed stiffnesses

LOCATION	CHARACTERISTIC SECTION	STIFFNESS PERCENTILE	CORRELATED BACK-ANALYSED LAYER STIFFNESSES (MPA) [1]		
			CONCRETE	ASPHALT	FOUNDATION
Runway 13-31	A1	85th	**9000**	-	**90**
		50th	21100	-	150
	A2	85th	13500	-	110
		50th	26500	-	150
	B	85th	-	2000	170

LOCATION	CHARACTERISTIC SECTION	STIFFNESS PERCENTILE	CORRELATED BACK-ANALYSED LAYER STIFFNESSES (MPA) [1]		
			CONCRETE	ASPHALT	FOUNDATION
Runway 13-31		50th	-	2800	220
	C1	85th	-	2900	130
		50th	-	4400	190
	C2	85th	-	3100	140
		50th	-	4300	160
Runway 08-26	D	85th	-	1400	100
		50th	-	2300	190
	E	85th	-	**≤500**	**80**
		50th	-	**<1000**	100

Notes:
[1] Concrete and asphalt stiffness values rounded to 100MPa. Foundation stiffness values rounded to10MPa.
[2] Asphalt material stiffness is shown at site temperature along with the stiffness adjusted to the reference temperature of 20°C in parentheses.
100 Red denotes material stiffness classified as "Poor" in accordance with DMG27.
100 Underlined values represent percentile typically used for design purposes.

4.4 Discussion of FWD Results

Reference is made to DMG27 which relates back-analysed material stiffness to material condition, as summarised in Table 13. The percentile values that are commonly compared for design purposes are the 15th percentile of asphalt material; the 50th percentile of concrete materials; and the 15th percentile of the foundation.

Table 14: Typical back-analysed stiffness values (DMG27)

CONDITION	STIFFNESS (MPA)			
	PAVEMENT QUALITY CONCRETE (PQC)	LEAN CONCRETE (LC)	ASPHALT	FOUNDATION
Excellent	>30,000	>15,000	>7,000	>200
Good	20,000 to 30,000	8,000 to 15,000	4,000 to 7,000	100 to 200
Average	10,000 to 20,000	3,000 to 8,000	1,000 to 4,000	-
Poor	<10,000	<3,000	<1,000	<100

4.4.1 *Concrete*

Concrete (Pavement Quality Concrete) material was encountered solely at the ends of Runway 13-31.

Apart from the identified spalling, no major distresses could be found during coarse visual survey, suggesting the concrete sections are in relatively sound condition. This seems to be confirmed by the 50th percentile back-analysed stiffness of the concrete layers which shows Characteristic Sections A1 and A2 to be in *Good* condition.

The low 15th percentile back-analysed stiffness, particularly at Characteristic Section A1, is likely to be indicative of some localised material deterioration which may be attributable to the combination of repeated loading and age. This should be flagged as a risk area, and may require localised slab replacement if cracked under loading.

The compressive strength testing (CST) conducted on two samples extracted from Characteristic Section A1 indicated notably poorer compressive strength than samples extracted from Characteristic Section A2. This is likely to be related to the different aggregate types and possible voiding present in the respective concrete.

4.4.2 ***Asphalt***

4.4.2.1 Runway 13-31

The coarse visual survey carried out during the FWD testing survey did not reveal any major signs of distress. From historical work records provided to AECOM it is known that a 50mm inlay and surface dressing was carried out recently throughout Runway 13-31 (13m either side from centreline).

The 15th percentile back-analysed stiffness of the asphalt materials on Runway 13-31 indicates that the material is in *Average* condition.

PAK-Marker test revealed that the lower asphalt layers may contain Poly-Aromatic Compounds (PACs) typically found in tar. As a result, PAH and WAC testing were employed to assess the content quantitatively. The test confirmed levels of Benzo(A)Pyrene and PAH within allowable disposal limits.

4.4.2.1 Runway 08-26

The coarse visual assessment revealed the surfacing of Characteristic Section D to be in relatively good condition. However, the surfacing material encountered in Characteristic Section E was found to be mainly disintegrated and loose. Patches of vegetation observed across the area suggests material damage at depth. These seem to be reinforced by the visual assessment of core samples extracted from the area and the 15th percentile correlated back-analysed stiffnesses, which indicated the asphalt layer of Characteristic Section D and E to be in *Average* and *Poor* condition, respectively.

It is worth noting that the derived back-analysed asphalt stiffnesses used in material condition assessment have not been temperature-corrected for Section E as the material was found to be mainly disintegrated. In accordance with DMRB HD 29/08[10] the temperature dependency of the stiffness of severely cracked asphalt tends to be far less than that of intact materials hence the temperature correction should not be normally applied.

PAK-Marker test revealed that the bottom asphalt layer may contain Poly-Aromatic Compounds (PACs) typically found in tar. As a result PAH testing was employed to assess the content quantitatively. The test confirmed presense of Benzo(A)Pyrene and PAH in excess of allowable disposal limits.

4.4.3 ***Foundation***

The 15th percentile back-analysed stiffness of foundation material throughout the majority of Runway 13-31 is considered to be *Good* with the exception of Characteristic Section A1 where *Poor* foundation condition was

[10] Highways England, Design Manual for Roads and Bridges, Volume 7, Section 3, Part 2 HD29/08 Data for Pavement Assessment.

found. At Runway 08-26, the 15^{th} percentile back-analysed stiffness of foundation was found to be ranging between *Good* and *Poor*. Nonetheless, the difference between correlated foundation stiffnesses used to categorise foundation condition, appear similar. The values are very close to the threshold (i.e. 100MPa) between *Good* and *Poor* foundation hence should be treated with caution.

The back-analysed stiffness is a measure of the overall foundation performance, encompassing weak/strong sub-layers including any granular fill or sub-base type material that may be present. The DCP testing, where penetration is possible, provides CBR profiles with depth and can highlight weaker layers within the pavement structure.

A full penetration was not possible at the majority of test locations. This was possibly due to large aggregate or a stiff foundation. Where adequate penetration was achieved a minimum foundation CBR of 9% was found.

The measured foundation CBR appear to correlate with the previous pavement investigation carried out in 1999[11].

[11] Scott Wilson Kirkpatrick, Wick Airport – Pavement Investigation Report, October 1999.

5 PAVEMENT CLASSIFICATION NUMBER AND REMAINING LIFE

The Pavement Classification Number (PCN) has been calculated in accordance with DMG27 Edition 3[12]. In addition, the BAA method[13] has been utilised to estimate the number of load repetitions before pavement failure as well as to calculate remaining life of pavement structures.

5.1 Traffic Data

5.1.1 *General*

[redacted]

Table 15: Boeing C17A ACN details for flexible pavements

AIRCRAFT	WEIGHT (kg)	LOAD ON MAIN GEARS (%)	TYRE PRESSURE (MPa)	MAIN GEAR TYPE	FLEXIBLE PAVEMENT – CHARACTERISTIC CBR (%)			
					15	10	6	3
Boeing C17A Globemaster	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

Note:
* The presented indicative ACN values for the maximum proposed MTOW of the C17A have been calculated by interpolation between minimum and maximum take of weight as quoted in Transport Canada – Aircraft Loading Tables.

Table 16: Boeing C17A ACN details for rigid pavements

AIRCRAFT	WEIGHT (kg)	LOAD ON MAIN GEARS (%)	TYRE PRESSURE (MPa)	MAIN GEAR TYPE	RIGID PAVEMENT – CHARACTERISTIC K-VALUE (MN/m^3)			
					150	80	40	20
Boeing C17A Globemaster	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

Note:
[redacted]

[12] DIO, Design and Maintenance Guide 27, A Guide to Airfield Pavement Design and Evaluation, Edition 3, February 2011
[13] Pavement Design Guide for Heavy Aircraft Loading, Aircraft Pavements, BAA Technical Services, July 1993
[14] Transport Canada, Aircraft Loading Tables, August 2004

The evaluation of the likely pattern of movements of the Boeing C17A highlights the need for backtracking. This means that the anticipated number of aircraft movements will approximately double.

From historical reports[15], it is known that main design aircraft at Wick Airport is the BAe 146-100 with a MTOW of 37,308kg and ACN ranging between 16.3 and 23.9. It is also known that in spite of the allowance for annual traffic growth of 2%, the volume of traffic at design aircraft over a period of 20 year is *Low* (i.e. less than <5,000 equivalent BAe146-100 coverages) in accordance with DMG27 (refer to paragraph 5.1.2 for details).

5.1.2 *Traffic to Defense Estates (DMG27)*

A mixed traffic analysis was undertaken in accordance with DMG27 to determine the number of coverages of the design aircraft. The selection of equivalent design aircraft is usually a combination of the most damaging aircraft with a reasonably high number of annual departures. DMG27 defines *coverages* as *the number of times a particular* point *on the pavement is expected to receive a maximum stress as a result of a given number of aircraft passes.* The number of anticipated movements of each aircraft type over the design life is calculated, based on forecast data. Then each aircraft is converted to an equivalent number of coverages of the specific design aircraft to give total number of coverages of the design aircraft over the design life. This is done by considering various factors including the weight, gear layout, damaging effect of each aircraft, pavement type and subgrade condition.

Previous traffic records[16] have been used to determine design aircraft and calculate number of equivalent coverages. As per previous report, BAe 146-100 has been selected as the design aircraft, whilst the 20-year design coverages have been calculated based upon a pass-to-coverage ratio for Runway pavement.

DMG27 considers three standard traffic level categories of Low, Medium and High, relating to 10,000, 100,000 and 250,000 *coverages* respectively. The *Low* traffic level has been calculated for the Wick Airport runways.

5.1.3 *Traffic to BAA method*

The BAA "Pavement Design Guide for Heavy Aircraft Loading" method typically utilises the complete aircraft mix. The stress/strain is calculated at the critical locations within the pavement structure for each aircraft, before calculation of the 'damage factor' for each aircraft loading in terms of the ratio of applied load repetitions to allowable load repetitions to failure. This is then totalled for all aircraft movements providing a Cumulative Damage Factor (CDF). The design approach makes the assumption that Miner's Rule for fatigue damage is applicable, and the CDF should therefore be limited to 1 in order to ensure satisfactory performance. Therefore, the damage associated with each of the aircraft loading scenarios (reciprocal of number of loads to failure) is multiplied by the expected number of loads during the design period and the CDF is calculated.

15 URS, Wick Airport – Pavement Evaluation, July 2013
16 URS, Wick Airport – Pavement Evaluation, July 2013

5.2 PCN Calculations

5.2.1 *General*

The ACN/PCN method of designating airfield pavements was originally developed in 1981 by ICAO[17]. The name "ACN/PCN" is derived from the classification of aircraft according to their Aircraft Classification Number (ACN), and pavements according to their Pavement Classification Number (PCN).

It should be noted that the ACN/PCN system is a method of reporting the relative strength of a pavement in order to evaluate airfield operations. It is not intended to be used for pavement design.

5.2.2 *ACN/PCN Method*

The ACN is a function that expresses the relative severity of loading on a pavement when supported by a subgrade of particular strength. The ACNs are reported separately for rigid and flexible pavements, for four standard categories of subgrade (representing ranges of subgrade strength and characterised by a standard value at the middle of the range), Maximum Take Off Weight (MTOW) and representative Operating Empty Weight (OEW).

The corresponding PCN is the value of ACN which applies an unrestricted loading to the pavement of equal severity to the maximum allowed for a pavement to survive a design life. Generally, to allow for unrestricted movements (or load applications of aircraft) a pavement should have a greater or equal PCN to the corresponding ACN of the aircraft.

The PCN number is reported as a five-part code as follows:

PCN/a/b/c/d where: PCN = the highest permitted ACN for unrestricted use

a = the type of pavement: R=rigid, F=flexible
b = the subgrade category
c = the maximum tyre pressure allowed:
W = high, no limit
X = medium, limited to 1.5 MPa
Y = low, limited to 1.0 MPa
Z = very low, limited to 0.5 MPa
d = pavement design or evaluation method
T = technical
U = by experience of in-service aircraft.

The pavement subgrade categories are:

A = High
B = Medium
C = Low
D = Ultra Low.

The ranges of subgrade strength covered by these categories are shown in Table 17.

[17] International Civil Aviation Organisation (ICAO), Aerodrome Design Manual, 1981.

Table 17: ACN/PCN subgrade categories

SUBGRADE CATEGORY	PAVEMENT TYPE	CHARACTERISTIC SUBGRADE STRENGTH	RANGE OF SUBGRADE STRENGTHS
A – High	Rigid	k=150MN/m^3	All k values above 120MN/m^3
	Flexible	CBR 15%	All CBR values above 13%
B – Medium	Rigid	k=80MN/m^3	k=60-120MN/m^3
	Flexible	CBR 10%	CBR 8-13%
C – Low	Rigid	k=40MN/m^3	k=25-60MN/m^3
	Flexible	CBR 6%	CBR 4-8%
D – Ultra Low	Rigid	k=20MN/m^3	All k values below 25MN/m^3
	Flexible	CBR 3%	All CBR values below 4%

Notes:
k Denotes Modulus of Subgrade Reaction
CBR Denotes California Bearing Ratio

5.2.3 *Concrete Flexural Strength*

The flexural strength (or Modulus of Rupture) is a critical parameter in the performance of the rigid pavements. The definitive measure of flexural strength requires the laboratory testing of beam samples. No historical laboratory test data of beam samples has been made available nor has been carried out as part of this pavement investigation due to project duration constraints.

The DMG27 suggests a standard ratio of 1:10 between compressive strength to flexural strength, however the guidance permits using a different ratio if justifiable. The desk study exercise carried out as part of this evaluation revealed that a ratio of 1:12 was used in 1999 for both concrete ends of Runway 13-31.

The visual assessment of the extracted concrete samples suggests that aggregates used for material production vary between Characteristic Sections A1 and A2. It was found that the lower compressive strength concrete from Section A1 contains gravel aggregate whilst the concrete extracted from Section A2 comprises crushed rock aggregate. The guidance provided by Nick Thom in "Principles of Pavement Engineering"[18] suggests using a ratio 1:12 for gravel aggregate and 1:10 for granite aggregate materials. For the purpose of this report ratio, 1:12 and 1:10 have been used for Characteristic Section A1 and A2, respectively.

5.2.4 *Characteristic Subgrade*

Due to shallow DCP refusal, the subgrade characteristic performance was based on combination of most recent DCPs, FWD response and available historical records[19]. It should be noted that elastic modulus (in MPa) for subgrade is required as part of the BAA method analysis, and thus has been calculated based upon the determined k-value and CBR percentage for rigid and flexible pavements, respectively, in accordance with the equations provided in the BAA "Pavement Design Guide for Heavy Aircraft Loading" guidance. The selected subgrade categories are shown in Table 18.

[18] Thom, N, Principles of Pavement Engineering, 2008
[19] Scott Wilson Kirkpatrick, Wick Airport – Pavement Investigation Report, October 1999

Table 18: Subgrade Categories

LOCATION	CHARACTERISTIC SECTION	PAVEMENT TYPE	K-VALUE (MN/M^3)	CBR (%)	ELASTIC MODULUS (MPA)	EXISTING SUBGRADE CATEGORY	HISTORICAL SUBGRADE CATEGORY
Runway 13-31	A1	Rigid	48	-	99	C	C
	A2	Rigid	37	-	81	C	C
	B	Flexible	-	10	103	B	B
	C1	Flexible	-	10	103	B	B
	C2	Flexible	-	10	103	B	B
Runway 08-26	D	Flexible	-	10	103	B	B
	E	Flexible	-	10	103	B	B

5.2.5 *PCN Calculations*

The PCN of each Characteristic Section has been calculated in accordance with the UK airfield design standard DMG27, published in February 2011 by Defence Estates (now Defence Infrastructure Organisation).

The PCN for Runway 13-31 and Runway 08-26 has been calculated based on the actual average (characteristic) parameters determined during the fieldwork and supported with historical records. This includes pavement structure, subgrade condition and PQC compressive strength.

The calculated PCN values are presented in Table 19.

Table 19: PCN of Characteristic Sections

[illegible]

[redacted]

According to pavement overloading guidance included in DMG27, occasional overloading operations are permitted with an aircraft ACN equal or lower than 50% of the pavement PCN.

Based on the average characteristic conditions used for the analysis, six out of seven analysed Characteristic Sections were found to have PCN deficiency [redacted] Therefore, according to DMG27 the movements of the C17A will not meet the necessary criteria for normal overloading operations as stated in the guidance and thus should only be allowed in emergency situations.

5.3 BAA calculations and pavement life

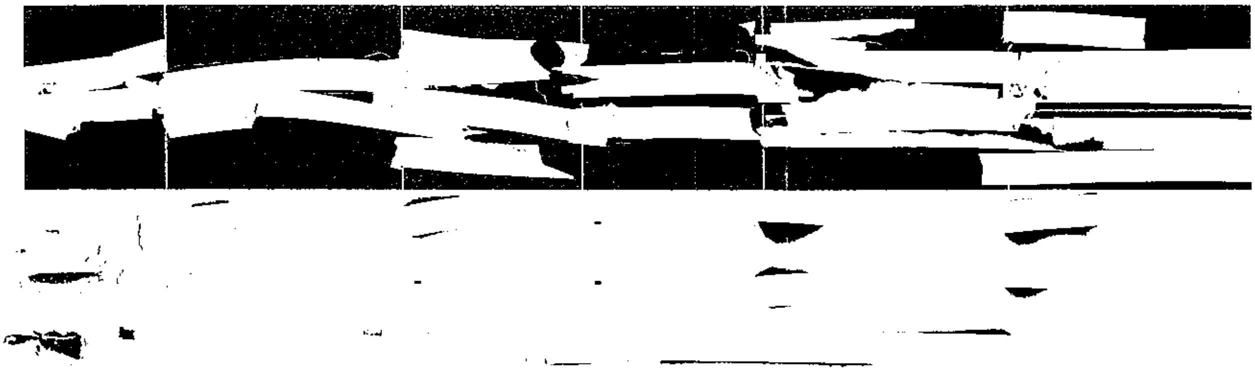
The pavement life was also calculated analytically, using the performance criteria outlined in BAA's "Pavement Design Guide for Heavy Aircraft Loading". The analytical evaluation was conducted to check how many loadings of Boeing C17A [redacted] can be received by the existing pavement structures before a failure occurs. An additional analysis was carried out to calculate remaining life of pavements subsequent to [redacted] in terms of equivalent coverages of BAe 146-100 design aircraft.

The pavement structures were modelled using AECOM in-house multi-layered linear elastic software "MPTRN" to calculate the maximum flexural stress at the base of the PQC layer and maximum vertical strain at the top of subgrade, for rigid and flexible pavement types, respectively, for each aircraft load case.

Table 20 presents the pavement life based on the actual parameters determined during the fieldwork.

Table 20: Existing Pavement Life

[redacted]



It can be seen that 16no. loadings of the Boeing C17A will reduce pavement life of Characteristic Sections C1 and C2 below the traffic volume threshold required by the Wick Airfield to accommodate future trafficking at design aircraft (BAe 146-100) without deformation of the subgrade.

The pavement condition at Characteristic Section E has been confirmed to be mainly disintegrated and with an inadequate structural capacity to carry out anticipated loading. The condition of the pavement is deemed beyond repair and thus will require full depth reconstruction or overlay treatment to allow trafficking by the anticipated loads.

Whilst Sections B and D are deemed to be sufficient so as to prevent failure through subgrade permanent deformation, cracking of the asphalt material is to be expected under the overload operations, but not so as to be classed as pavement 'failure'. Routine inspection and maintenance after Boeing C17A loading is therefore recommended.

It should also be noted that the flexural strength value for Section A1 is based upon only 2no. compressive strength results, and as such may not be indicative of true average flexural strength. If compressive strength was to be lower, it is anticipated that earlier cracking of the concrete may occur.

6 TREATMENT RECOMMENDATIONS

It has been found that Characteristic Sections C1, C2 and E will require strengthening in order to achieve required structural capacity to allo Boeing C17 movements and ensure adequate remaining life for the future traffic at Wick Airport.

It is worth noting that in accordance with DMG27, with the exception of Characteristic Section A2, the existing Pavement Classification Number and/or bearing capacity is not sufficient to carry the proposed C17 loading even under the permitted overloading procedure included in this guidance. However, the analytical calculations carried out in accordance with the BAA guide indicate the pavement damage to be within an acceptable level so as to prevent permanent deformation of the subgrade at Characteristic Sections A, B and D. Despite this, asphalt and concrete cracking and/or subgrade deformation may occur after the C17 loading, particularly if the subgrade has been saturated post extreme weather conditions and/or in places where there is a localised loss of support underneath the existing bound layers.

Structural treatment options have been proposed for Characteristic Sections C1, C2 and E, where analytical calculations show pavement structure to be insufficient to support the proposed future trafficking. Treatment options have been proposed in accordance with DMG27 in order to comply with the PCN requirements for trafficking of the Bae 146-100 at ACN 20 at *low* coverages (inclusive of C17A movements).

Whilst it is recommended that treatment be carried out at Characteristic Section E prior to C17A movements, it is recommended that the C17A aircraft be allowed to traffic Characteristic Sections C1 and C2, allowing the pavement to crack, before inspection, evaluation and full reconstruction is performed.

The recommended pavement structural treatment options are presented in Table 21. Where pavement strengthening or reconstruction is recommended, the thickness stated is the minimum thickness that is required and excludes construction tolerances.

Table 21: Structural Treatment Options

LOCATION	CHARACTERISTIC SECTION	STRUCTURAL TREATMENT
Runway 13-31	A1	*n/a* [1]
	A2	*n/a* [1]
	B	*n/a* [1]
	C1	*Full depth reconstruction 275mm (100mm Marshall Asphalt surfacing on HSBBM)*
	C2	*Full depth reconstruction 275mm (100mm Marshall Asphalt surfacing on HSBBM)*
Runway 08-26	D	*n/a* [1]
	E	*Full depth reconstruction 275mm (100mm Marshall Asphalt surfacing on HSBBM)*

Notes:
HSBBM Denotes High Strength Bound Base Material in accordance with DMG27
[1] Although theoretically structurally sufficient so as to prevent subgrade permanent deformation, cracking of the concrete/asphalt material may be expected under the overload operations, but not so as to be classed as pavement 'failure'. Routine inspection and maintenance after Boeing C17A loading is therefore recommended.

In addition to the treatments identified in Table 21, the following non-structural treatments are also recommended:

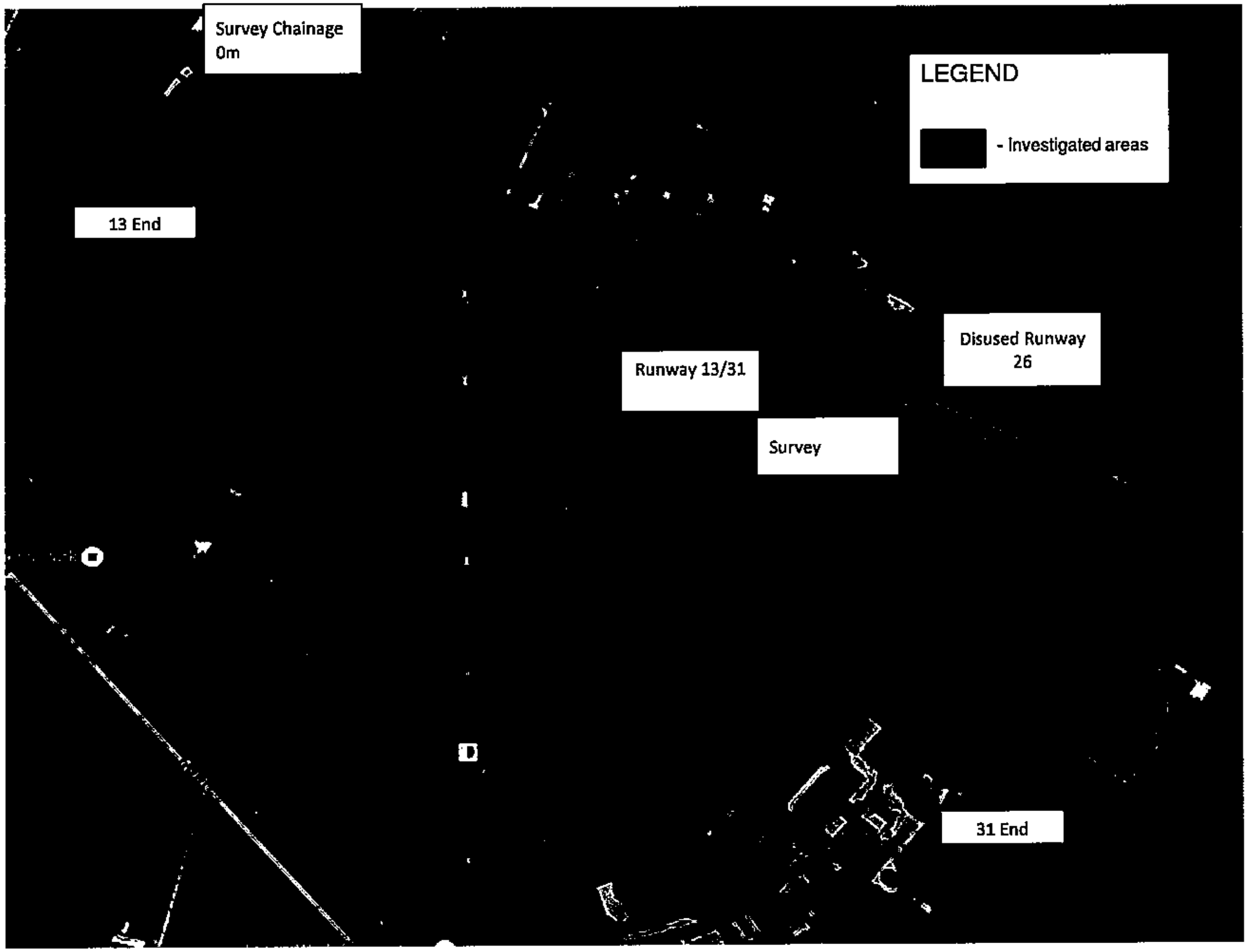
- Any existing drainage system should be cleared and adequately maintained.

- Routine maintenance should be performed throughout the pavement life, including asphalt crack sealing and patching, replacement of surface course, joint sealant repairs, concrete slab spall repairs or if necessary localized slab replacement, retexturing for friction and rubber removal.

IMPORTANT NOTE:

Regardless of the strengthening option chosen, it is highly recommended to carry out thorough pavement inspections by adequately trained / qualified pavement engineer prior and after each Boeing C17A pass, as outlined in DMG27 with regards to overload operations.

Figure 1. Survey Areas



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Project No.	47076156
Date	Oct '15

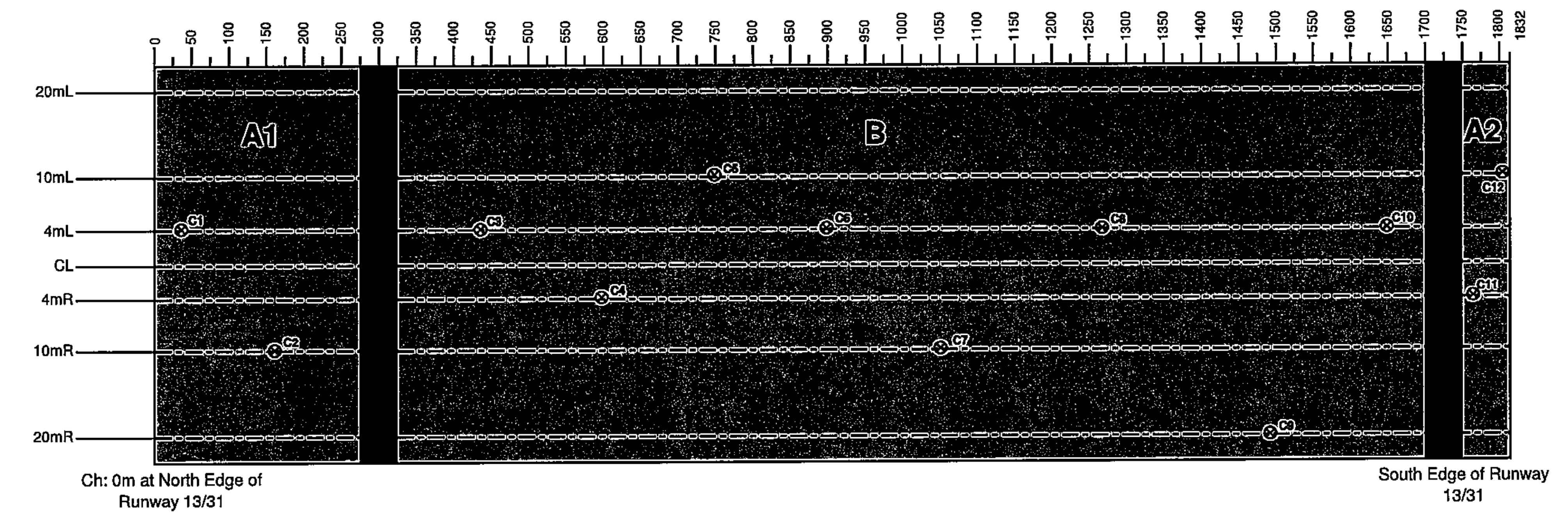
FIGURE 1

Wick Airport - Pavement Evaluation

Site Location Plan

AECOM

Figure 2. Site Survey Schematic



- Characteristic Section A1 & A2 - Ch: 0m to 274m & 1741m to 1832m - 300mm Concrete
- Characteristic Section B - Ch: 274m to 1741m - 310mm Asphalt
- Characteristic Section C1 & C2- Ch: 274m to 310m & 1700m to 1741m - 150mm Asphalt

-·-·- FWD profile locations

⊗ C4 Core location and number reference

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Date	Oct '15

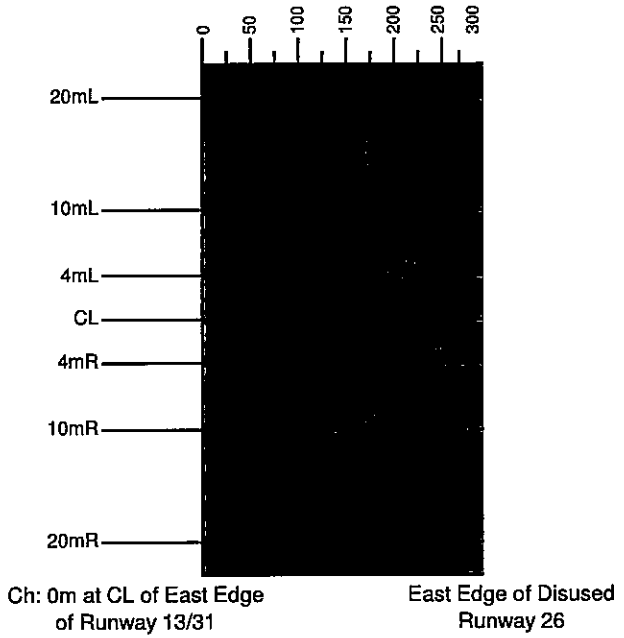
FIGURE 2a

Wick Airport - Pavement Evaluation

Runway 13/31

Characteristic Sections, Core and FWD Locations

AECOM



- Characteristic Section D - Ch: 0m to 75m - 200mm Asphalt
- Characteristic Section E - Ch: 75m to 279m - 100mm Asphalt

—·—·— FWD profile locations

⊗ C14 Core location and number reference

N

Prepared	AG
Checked	FM
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FIGURE 2b

Wick Airport - Pavement Evaluation

Disused Runway 26

Characteristic Sections, Core and FWD Locations

AECOM

Appendix A. Structural Evaluation with the Falling Weight Deflectometer (FWD)

STRUCTURAL EVALUATION WITH THE HEAVY FALLING WEIGHT DEFLECTOMETER (H/FWD)

Reasons for evaluation

At any stage in the life of a pavement it may be necessary to evaluate its structural capacity for one of the following reasons:-

1) It is to be used by heavier aircraft.
2) It is to take an increased number of aircraft.
3) It is showing distress and the reasons for this need to be known in order to rectify the problem and design appropriate strengthening.
4) In order to assist planning of future maintenance.
5) In order to give increased flexibility to airport operations.
6) In order to monitor rate of deterioration and hence depreciation of an important asset.

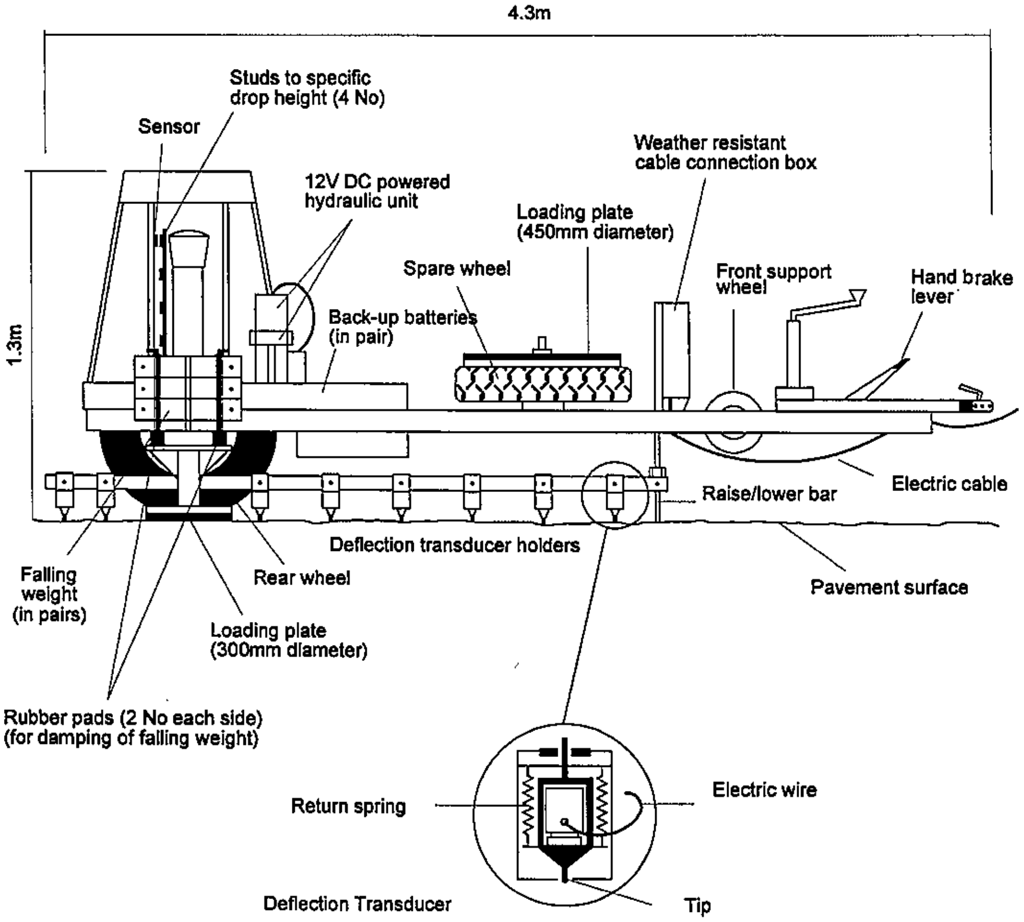
Much of the data required for such an evaluation can be rapidly obtained using a Heavy Falling Weight Deflectometer, which is trailer mounted and towed behind a standard road-going vehicle.

Operation of Heavy Falling Weight Deflectometer

The Heavy Falling Weight Deflectometer (H/FWD) is an apparatus for non-destructive testing, which loads the pavement in a controlled manner such that the load pulse resembles that from moving traffic. On Airports the Heavy FWD (or HWD) is often used, which enables loadings up to 20 tonnes to be applied to the pavement. Deflections of the pavement surface, at increasing radial distances from the load, are recorded automatically. Deflection d_1 is measured at the centre of the load and d_7 furthest from the load.

The arrangement for deflection measurements during the fieldwork is such that the shapes of deflection bowls up to 2.2m in radius can be recorded. In addition, when testing across joints or cracks, it enables data on load transfer to be obtained. The manoeuvrability of the H/FWD, allowing accurate positioning of the deflection measuring equipment, is essential for carrying out this work.

FALLING WEIGHT DEFLECTOMETER



Side Elevation of Transducers

The magnitude of the applied load is also recorded. This can be adjusted by changing the mass of the falling weights, or the height from which they are dropped, in order to obtain a contact pressure on the pavement surface which approximates to the pressure exerted by the tyres of the vehicles using the pavement.

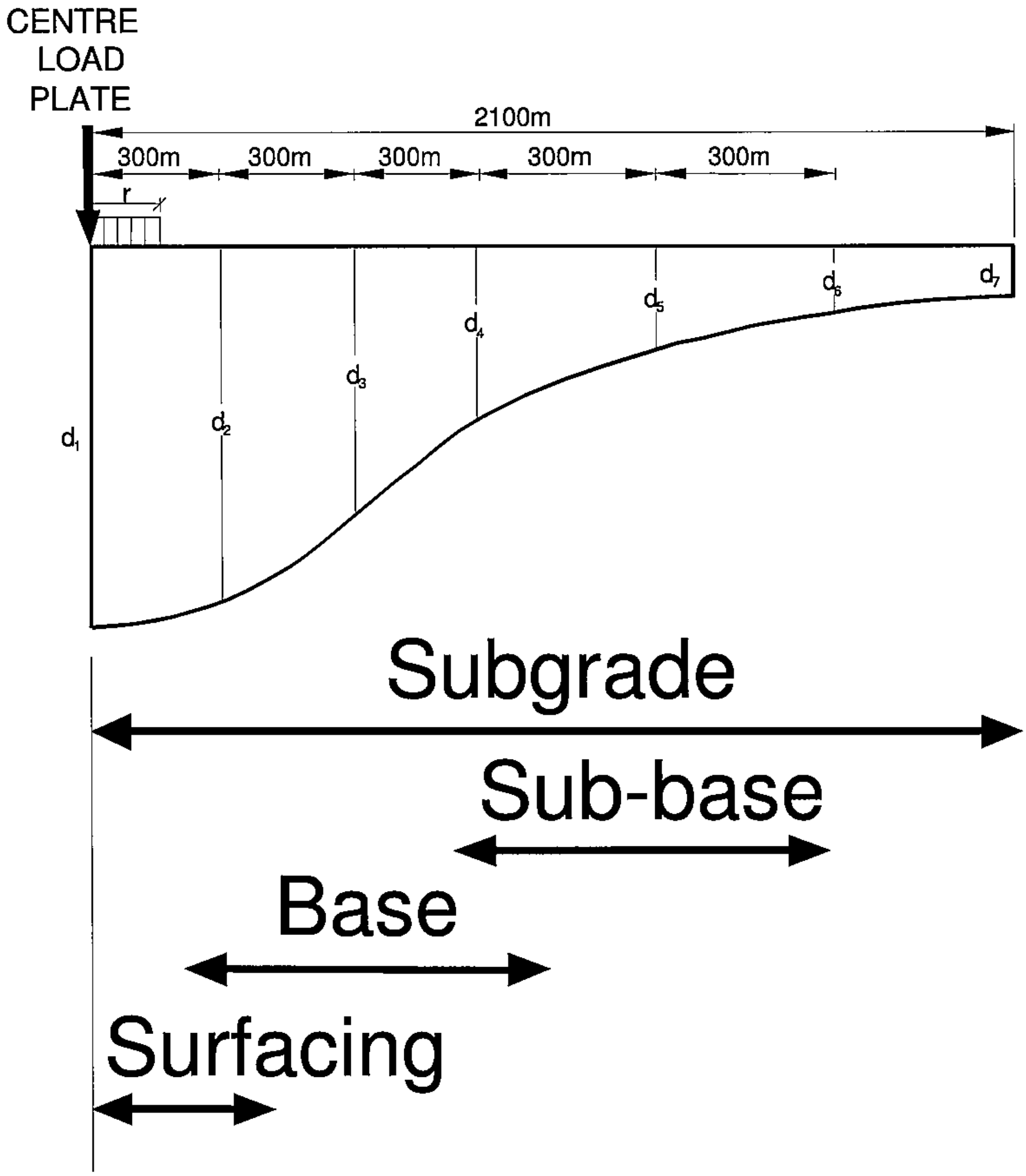
Data Handling of H/ FWD Results : Excluding Joints and Cracks

The H/FWD results are tabulated in the form of normalised deflection bowls for each H/FWD test, (i.e. those adjusted to a common contact pressure of the H/FWD loading platen on the pavement surface). In addition statistical analyses are performed for various deflection criteria. The chosen criteria are often d_1, d_7, and (d_1-d_4) although this depends on the pavement structure to some extent. The central deflection d_1 gives an idea of overall pavement performance, whilst the deflection difference (d_1-d_4) indicates the condition of the bound pavement layers. Deflection d_6 or d_7 is used as an indicator of subgrade condition.

The statistical analyses indicate the spread of results by giving maximum, minimum, 85, 50 and 15 percentile values for the three deflection criteria. (The 85 percentile value is such that 85% of the measured deflections are less than or equal to it). Alternatively the mean and standard deviations are calculated. The statistics are used later to obtain representative effective stiffnesses for the various pavement and subgrade layers (see "Interpretation of H/FWD Data") which are then used for the assessment of residual life and the design of strengthening measures.

In addition, profiles of deflection criteria are often plotted, in order to show the variation of pavement layer and subgrade stiffness along a length of pavement. These profiles are of assistance for developing appropriate strategies for future maintenance and strengthening measures.

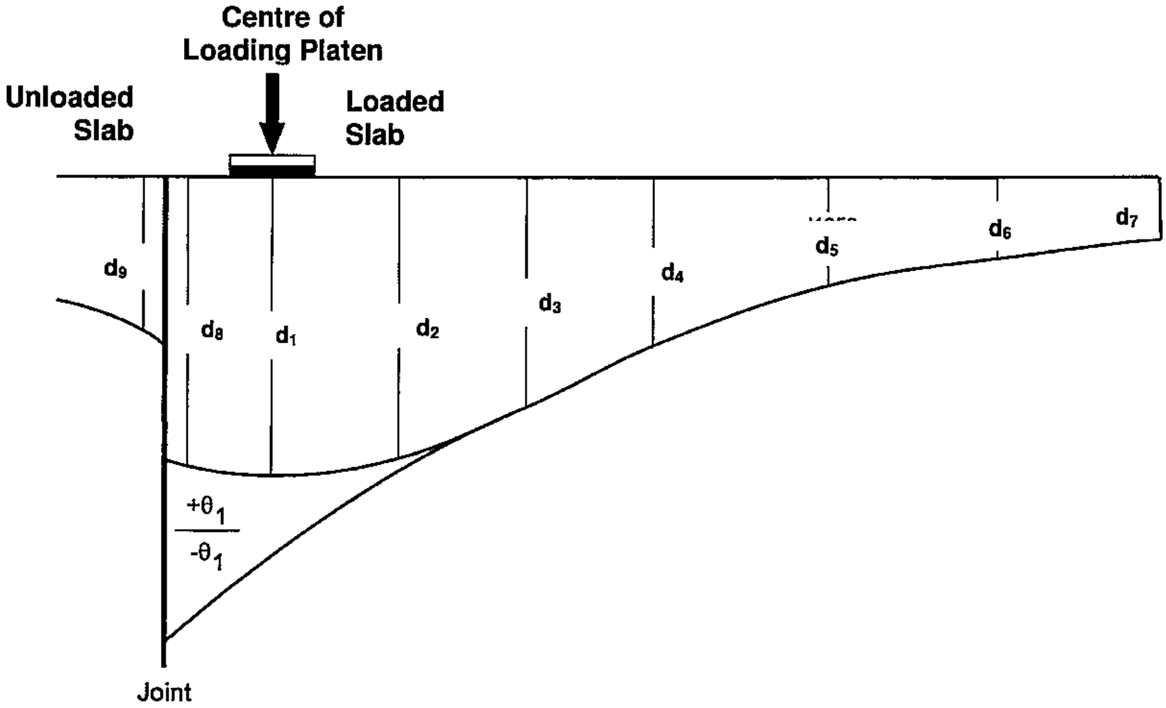
H/FWD DEFLECTION BOWL



Data Handling of H/FWD Data : Joints and Cracks

The results of HWD testing across joints are also tabulated, followed by a separate statistical summary of indicators for both load transfer (d8-d9 and d9/d8) and slab support (θ_1), as defined in the attached figure.

INDICATORS OF LOAD TRANSFER AND SLAB SUPPORT AT A JOINT



If a small step occurs across the joint or crack, this indicates good load transfer whilst a large step indicates poor load transfer. Poor load transfer is representative of non-existent, corroded, broken or loose dowel bars.

If the slab curvature of the loaded slab is positive, then this indicates good slab support - if negative, then slab support is poor. If the slab curvatures either side of the joint or crack appear to be parallel to each other, then this indicates good slab support - if divergent, then slab support is poor. Poor slab support may represent a stage of deterioration where the concrete slabs have become weakened due to loss of sub-base support.

The statistical summaries indicate the spread of results by showing maximum, minimum and average values for the chosen indicators of load transfer and slab support. For indicators (d_8-d_9), high values relate to the poorest behavior; for (d_9/d_8), low values are the poorest and for θ_1, negative values indicate poor behavior.

Interpretation of H/FWD Data

All the deflection bowls obtained from the H/FWD data are analysed using a computer program which determines the effective stiffness of the various pavement and subgrade layers, by matching measured deflections to computed values. The program uses a multi-layer linear elastic model for the pavement layers and a non-linear model for the unbound material below formation level (fill and natural ground), the stiffness of which is stress dependent. The thickness of pavement layers is required for this back-analysis procedure, and these are usually determined by a coring survey, in conjunction with Ground Radar where appropriate.

The stiffness of bituminous materials is influenced by temperature and duration of loading, and so an adjustment must be made to the back-analysed stiffnesses based on in-situ measurements, before a residual life can be calculated. This is done for each bituminous bound layer using the pavement temperatures recorded during FWD testing and knowing the equivalent speed of the FWD load pulse on the pavement. Similarly the effective stiffness of the subgrade may need adjusting to allow for the loadings applied by some of the heavier aircraft, although use of the HWD largely negates this.

Once effective elastic stiffnesses have been obtained, pavement evaluation can be carried out in two ways, either using -

a) **Analytical principles**

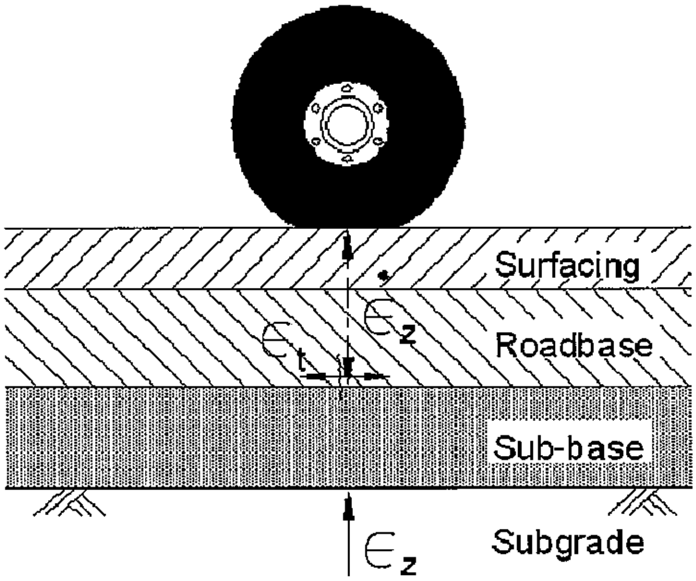
or b) **A reversal of the design process based around manuals or charts.**

a) **Analytical:** This method of pavement evaluation uses the same principles as Analytical Pavement Design, except that the effective stiffnesses and material properties represent the pavement condition in-situ. The attached figure illustrates the three primary modes of distress which are considered for flexible pavements. Using multi-layer elastic theory to determine the interactions between the different pavement layers, the stresses/strains generated in the pavement are calculated for the required loading (design aircraft). Based on typical properties of the pavement materials, relationships can be obtained between the stress/strain criteria and the number of load applications before the pavement reaches its design life.

Wherever possible materials testing is performed on samples cored from the pavement to assist in characterisation of material behaviour. The results from the Indirect Tensile Stiffness Modulus test

(ITSM) in the Nottingham Asphalt Tester (NAT) provide data on stiffness at different temperatures and/or strains which is used to assist interpretation of the FWD data. The Repeated Load Axial Test (RLAT) is used to define visco-elastic material properties. The RLAT data is then used, in a multi-layered viscous model of the pavement, to predict the progression of non-structural rutting within the bituminous materials. A third test in the NAT, the Indirect Tensile Fatigue Test (ITFT), helps to define the resistance of the material to fatigue cracking. After NAT testing, compositional analysis is performed on the samples and the binder rheology is characterised in the Bohlin Dynamic Shear Rheometer (DSR). This is important for identifying the presence of any binder modifiers, but the results are also used to compare with the US (SHRP) binder specifications and to obtain both Pen and Softening Point.

For concrete slabs the life of the pavement is assessed by calculating the stresses which are induced under the passage of the design loading. The most significant stresses are the horizontal tensile stress at the bottom of the concrete in a slab centre location, and the tensile stress at the top of the concrete at an edge or corner location. The former is calculated using the pavement model determined from the back-analysis, and then factors are applied (based on developments of the Westergaard equations) to determine the edge or corner stresses. The stresses caused by restraint of temperature induced movements can also be considered. The life prior to onset of fatigue cracking can then be determined from the ratio of induced stress to concrete strength. For a lean concrete base, the design criterion relating to fatigue is maximum tensile stress at the bottom of the layer.



ϵ_t – HORIZONTAL TENSILE STRAIN REPRESENTS FATIGUE CRACKING IN BOUND LAYERS

ϵ_z – VERTICAL SUBGRADE STRAIN REPRESENTS PERMANENT DEFORMATION OF THE SUBGRADE

$\dot{\epsilon}_z$ – STRAIN RATE REPRESENTS VISCOUS DEFORMATION IN BOUND LAYERS

STRAIN PARAMETERS CONSIDERED IN ANALYTICAL PAVEMENT DESIGN

The most common reporting system worldwide for pavement strength is by using the ACN/PCN method. The PCN (Pavement Classification Number) is derived using analytical principles as follows:-

i) The effective stiffness of the subgrade, derived from analysis of FWD data is equated to a subgrade category (High, Medium, Low, Ultra-low).

ii) Using loadings appropriate to a range of aircraft types, including the critical one (i.e. the most damaging aircraft for that pavement type), the pavement life can be assessed in terms of the number of load coverages and hence passes.

iii) The PCN of the pavement corresponds to the Aircraft Classification Number (ACN) of that aircraft which can be carried for the required number of coverages. The PCN may be less than the ACN of the critical aircraft, in which case a limiting number of coverages can be determined.

iv) If the PCN of the existing pavement is inadequate, the effect of different overlay thickness on the overall life of the pavement can be obtained for a selection of overlay materials, and suitable remedial measures designed. Alternatively, full or partial reconstruction options can be assessed, using stiffnesses for the lower pavement layers and subgrade which are based on the back-analysed values.

b) **Reverse Design:** In addition to the above approach, use is made of existing design documents and design charts, in particular those produced by ICAO, FAA, BAA and PSA. The effective elastic stiffnesses obtained from analysis of FWD data enable an engineering assessment of the condition of the various pavement layers, and, therefore, choice of appropriate condition factors to be used in the design equations or charts.

For both the above approaches, software is now available which enables the damaging effect of a particular aircraft mix to be quickly computed for a particular pavement. This removes some of the gross simplifications inherent in the Pass-to-Coverage approach. However, many of the existing design documents only consider a single failure mechanism for flexible pavements - that is overstressing of the subgrade. This is unlikely to be the predominant failure mode for modern, thick pavement structures. It is therefore important to ensure that appropriate analytical techniques are used to assess failure modes such as cracking, and also that performance related materials testing is performed to assess engineering properties and durability aspects.