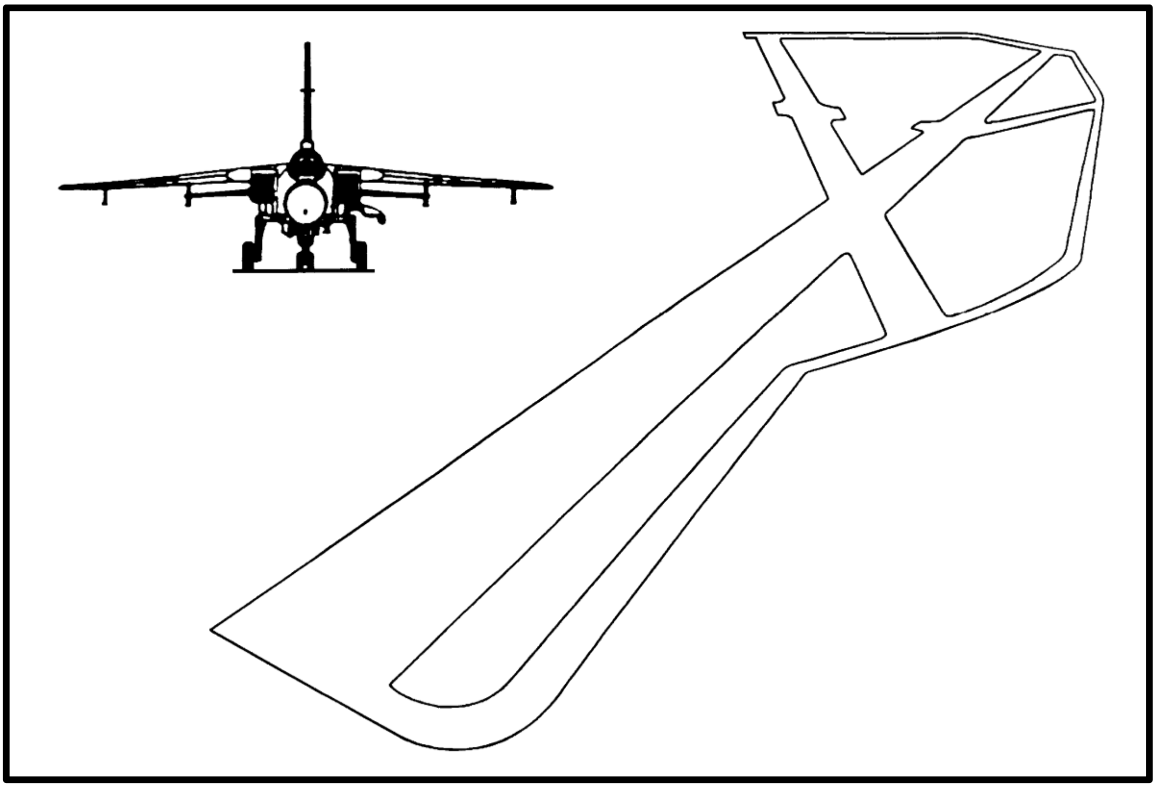
Design & Maintenance Guide 27

A Guide to Airfield Pavement Design and Evaluation



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Design and Maintenance Guide 27

A Guide to Airfield Pavement Design and Evaluation

3RD EDITION – FEBRUARY 2011

CONSTRUCTION SUPPORT TEAM
DEFENCE ESTATES
MINISTRY OF DEFENCE

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Foreword

This guide has been prepared under the patronage of the Construction Support Team, Defence Estates, Ministry of Defence to provide guidance on the structural design and evaluation of airfield pavements. It supersedes the previous edition published in 1989.

The design and evaluation methods presented in this guide are developments of previous methods, incorporating the benefits of additional experience and research.

The aircraft/pavement classification system incorporated in this guide is the ICAO ACN-PCN method. Methods for approximating the relationship with the previously used LCN/LCG system are included.

Further technical assistance regarding the contents of this document can be obtained from Defence Estates. Enquiries regarding this guide should be made to the airfield pavement technical Authority:

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Glossary

Term	*Abbreviation*	*Definition*
Aircraft Classification Number	ACN	A number expressing the relative effect of an aircraft on a pavement for a specified standard subgrade strength. A component of the ICAO ACN-PCN method.
All-up Mass/Weight		A term meaning the total mass/weight of the aircraft under defined conditions, or at a specific time during flight. (Not to be confused with MTOW).
Blast Pad		A length of pavement adjoining the runway end, designed to resist jet blast from aircraft standing on the runway before take off. Generally part of a stopway.
Bound Base Material	BBM	Any material equivalent to a granular sub-base or better, which uses a cement or bituminous binder.
British Standard	BS	A publication of the British Standards Institution.
California Bearing Ratio	CBR	An indication of the bearing capacity of a soil. It is determined by comparing the penetration load of a soil to that of a standard material.
Cement-Stabilised Soil		A relatively low quality cement-bound material produced by the addition of the cement to a natural soil. Mixing can take place in situ or in a mixing plant.
Cohesive Soil		A soil which contains clay; forms a coherent mass. For determining relative compaction requirement, cohesive soils are taken as those with a Plasticity Index greater than or equal to 6%.
Composite Pavements		Pavements consisting of mixed rigid and flexible layers.
Coverage		The application of a maximum stress on a point in the pavement surface.
Design Aircraft		The aircraft which imposes the most severe loading on the pavement.
Drylean Concrete	DLC	A low-strength Portland cement concrete generally used as a sub-base and/or base course under PQC or bituminous surfacing (see Rolled Drylean Concrete) or as a working course. Water content and strength requirements are specified.
Equilibrium Moisture Content		The moisture content at any point in a soil after moisture movements have ceased.
Equivalent Coverages		The number of Coverages by one aircraft which has the same damaging effect on the pavement as a given number of Coverages by another aircraft.

Term	*Abbreviation*	*Definition*
Flexible Pavement		A pavement which distributes the load primarily through the sheer strength of the materials.
Formation		The surface of the subgrade in its final shape after completion of the earthworks.
Frequency of Trafficking		The level of Coverages for which the pavement is designed. There are three categories. High, Medium and Low.
International Civil Aviation Organisation	ICAO	
Load Classification Group	LCG	A range of LCN values.
Load Classification Number	LCN	A number expressing the relative effect of an aircraft on a pavement or the bearing strength of a pavement. The original LCN classification system was developed in the UK in the late 1940s but in 1971 the method of calculating LCNs was altered and the LCN/LCG system introduced. LCN values from the two systems are not compatible.
Main Wheel Gear		The undercarriage leg used in ACN calculation.
Marshall Asphalt	MA	An asphalt designed by the Marshall method to meet strict specification requirements in order to provide a durable, high stability flexible surfacing material.
Maximum All-Up Weight	MAUW	The higher of MTOW and MRW.
Maximum Ramp Weight	MRW	Maximum Take Off Weight plus any taxi/runup fuel load.
Maximum Take Off Weight	MTOW	The maximum aircraft weight allowable at take off.
Mixed Traffic		A mixture of aircraft types using a pavement, all of which produce a calculable effect on the fatigue life of a pavement.
Mixed Traffic Factor	RMTF or FMTF	A figure used in converting Coverages by an aircraft with one ACN to equivalent Coverages by an aircraft with a different ACN. There are different MTF systems for Rigid (RMTF) and Flexible (FMTF) pavements.
Modulus of Subgrade Reaction	k	A measurement of the bearing strength of a soil obtained from a loading test with a 762mm (30 inch) diameter plate.
Movement Area		Pavements intended for use by aircraft, including runways, taxiways, aprons and other areas provided for the operation or maintenance of aircraft.
Multiple Slab Pavements		Pavements consisting of two or more concrete layers, with or without separating layers.
Non-cohesive soil		A granular soil; does not form a coherent mass. For determining relative compaction requirements, non-cohesive soils are taken as those with a Plasticity Index less than 6%.

Term	*Abbreviation*	*Definition*
Overlay		An additional layer or layers of structural pavements materials on an existing pavement.
Overload		Use of a pavement by aircraft with a classification (ACN) greater than the pavement classification (PCN).
Overslab		A concrete overlay.
Pass		An aircraft movement over a particular section of the pavement. Under certain conditions a pass may be taken as a movement by departing aircraft only.
Pass-to-Coverage Ratio		The number of passes of an aircraft on a pavement which produces one Coverage at a point in the pavement.
Pavement		A structure consisting of a layer or superimposed layers of selected materials, whose primary purpose is to distribute the applied loads to the subgrade.
Pavement Classification	PCN	A number expressing the bearing strength of a pavement for unrestricted operations by an aircraft with a classification (ACN) of the same number. A component of the ICAO ACN-PCN method.
Pavement Quality Concrete	PQC	A Portland cement concrete designed within strict limits to give a durable material in pavement applications.
Reflective Crack		A crack in a pavement layer induced by a crack in the underlying layer.
Relative Compaction		The percentage ratio of the dry density of the soil to the maximum dry density of that soil as determined in a compaction test.
Rigid Pavement		A pavement which distributes the load by means of its high flexural stiffness.
Rolled Drylean Concrete		A drylean concrete which is compacted by rolling to give a dense material.
Shoulder		A strip adjacent to the edge of a movement area prepared to provide a transition in strength and , if necessary, in grade between the movement area and the adjacent ground, to provide for use by aircraft in an accident or emergency.
Stopway		A defined rectangular area at the end of runway, designated and prepared as a suitable area in which an aircraft can be stopped if the take off is aborted.
Subgrade		The natural or made-up ground supporting the pavement.
Temperature Warping Stresses		Stresses due to a temperature gradient through the depth of the concrete slab.
Transport Road Research Laboratory	TRRL	

Term	*Abbreviation*	*Definition*
Twin Slab Pavement		A multiple slab pavement consisting of two slabs laid at the same time to obtain a thicker equivalent single slab thickness without compaction problems.
Wander		The width over which movements of an aircraft centre-line are distributed 75% of the time.
Westergaard's Constant	k	See Modulus of Subgrade Reaction.
Unrestricted Operations		A term meaning that the operator does not have to apply any limitations on use by an aircraft at a particular ACN.

1 Introduction to Airfield Pavement Design in the United Kingdom

1.1 GENERAL

1.1.1. The design of an airfield pavement requires realistic methods of assessing the loading characteristics of aircraft and the structural response of the pavement. It has long been recognised that the severity of load-induced stresses in a pavement and subgrade depends on the gross weights of the aircraft using the pavement and the configuration, spacing and tyre pressures of their undercarriage wheels. The response of the pavement in resisting these stresses depends on its thickness, composition, the properties of materials used in its construction and the strength of the subgrade on which the pavement is built.

1.1.2. Through the years, these basic concepts have been developed and extended to include the effects of fatigue, environmental factors, mixed traffic, overload operations etc. Major developments in aircraft designs have required a continuing review of existing pavement designs and the trend up to now has been that new generations of aircraft demand pavements designs well ahead of any practical experience of previous aircraft use.

1.1.3. The design methods for airfield pavements have largely grown out of the experience of pavement performance. For rigid pavements, which rely on the flexural stiffness of concrete to distribute the loads from aircraft wheels to the subgrade, use has been made since the early 1940s of theoretical approaches developed by Westergaard and others. Because of difficulties encountered in developing a realistic mathematical model for flexible pavements, which depend on the mechanical strength of compacted aggregates, empirical design methods (e.g. the CBR method) are still commonly used.

1.2 DEVELOPMENT OF PAVEMENT DESIGN IN THE UNITED KINGDOM

1.2.1. In the UK, the history of airfield pavement design really began in 1937 when the first paved runways were constructed, using road experience as a guide. Flexible pavements were comprised of layers of brick or stone topped with two courses of tarmacadam and a sealing coat of mastic asphalt. Concrete pavements were either 150mm or 200mm thick slabs generally laid directly on to the ground after the removal of the topsoil. These early pavements soon failed under the increasing weight of new aircraft and were overlaid with 65mm thickness of tarmacadam and a 20mm thick sealing course of rolled asphalt. The overlays were remarkably successful on concrete and were the first composite pavements. The flexible pavements on the other hand, kept failing and were either replaced by concrete pavements or strengthened with further overlays of tarmacadam.

1.2.2. The Air Ministry Works Directorate, which was responsible for design, construction and maintenance of all airfields for the UK Government, constructed some 450 airfields between 1937 and 1945 without having the benefit of proven design methods. Nevertheless, extensive data on pavement performance, construction details and subgrade characteristics was collected and during the last stages of World War Two attention was being given to developing proper methods of design for airfield pavements.

1.2.3. The first design method[1], published by the Department* in 1945, used Westergaard's equations for calculating the stresses induced in a concrete pavement by aircraft loads and Bradbury's equations for calculating warping stresses induced by thermal effects. The cracking of the slaps was controlled by limiting the allowable flexural stresses in concrete.

1.2.4. As aircraft increased in all-up weight and a wider range of tyre pressures was introduced it became obvious that a system of classifying aircraft, according to the severity of stresses produced in the pavement, was necessary. A series of plate-bearing tests was put in hand to investigate the relationship between the load necessary to produce the failure of a pavement and the contact area over which the load was applied. The results of these tests led to the development of an empirical relationship expressed in the following form:

$$\frac{W_1}{W_2} = \left[\frac{A_1}{A_2}\right]^{0.44} \qquad (1)$$

W_1 and W_2 are the failure loads and A_1 and A_2 are the contact areas for two combinations with the same damaging effect.

1.2.5. In 1948, the Department published a load classification system[2] which assigned a Load Classification Number (LCN) to aircraft whose loads and contact areas (derived from tyre pressures) were linked by Equation 1. The LCN represented the relative damaging effect of wheel loads and tyre pressures of aircraft within a practical numerical scale ranging from 1 to 100. The LCN system is still used in some countries and at many military airfields.

1.2.6. During the early 1950s, a method of using plate-bearing tests was developed for evaluation of airfields. Publication TP104/51[3], issued in 1952, included a formal description of the LCN system which had by then been extended to cater for multiple wheel undercarriages, evaluation techniques using plate bearing tests and advice on overload operations. A year later, the Department published a paper[4] describing its latest thinking on the design concepts. Since good compaction of slabs thicker than 300mm was difficult to achieve with techniques available at that time, a twin slab with the corners of the upper slab supported at the centre of the lower slab was used to provide an equivalent construction. To deal with the corner case more accurately the Teller and Sutherland modification to the Westergaard corner case was incorporated into the design procedure. For the design of flexible pavements two methods were introduced – a method based on the CBR equation and the 'Search Plate' method which was abandoned later.

1.2.7. During the construction of flexible pavements including unbound granular materials, problems had sometimes been experienced in uniformly compacting the high quality materials to the levels required. These pavements produced poor performance in the short and long term. Experiments were therefore carried out using full-depth bound constructions by placing weak cement-bound layers beneath bituminous layers. These were very successful and full-depth bound constructions have been adopted as standard construction by the Department since 1954.

1.2.8. The cumulative developments in design methods and the associated construction practices were brought together in the Department's publication entitled 'Airfield Design and Evaluation'[5] produced in 1959. It included design charts for rigid and flexible pavements which used LCNs as the parameter for aircraft loading. For rigid pavements a procedure for allowing two levels of trafficking – channelised and non-channelised – was introduced. The possibility of using reverse design for evaluating the strength of airfield pavements was mentioned. As the compaction of concrete thicknesses greater than 300mm had become possible, the use of twin slabs was discontinued. Charts for the design of overlays on existing pavements were included.

* Throughout the guide 'the Department' refers to Defence Estates and its predecessors in the Directorate of Civil Engineering Services (Airfields Branch), the Ministry of Public Building and Works and Air Ministry Works Directorate.

1.2.9. Experience during the 1960s showed that the plate-bearing tests developed for flexible pavements gave over-optimistic results when such pavements had cement-bound bases. An alternative heavy rolling test was therefore introduced. It was also discovered that the strength of twin slabs and overlays on rigid construction was being overestimated. A new design technique, assuming a high subgrade strength on the surface of the underlying slab, was therefore developed.

1.2.10. At a symposium organised by the Institution of Civil Engineers in London on 12 November 1970, the Department summarised its state-of-the-art on design, evaluation and strengthening of airfield pavements. Three papers[6,7,8] presented at the symposium discussed the effects of multiple wheel undercarriages, limiting criteria for failure of rigid and flexible pavements, types of pavement which had proven to be most satisfactory and design of strengthening.

1.2.11. The Department's last guide, entitled 'The Design and Evaluation of Aircraft Pavements 1971[9], introduced the concept of Load Classification Groups (LCG) which categorised aircraft LCN valued into seven groups. Aircraft imposing similar stress levels on particular pavement thicknesses normally used in construction were placed in one group. This simplified the design and evaluation of pavements and was thought to be sufficiently accurate.

1.2.12. The Load/Contact area relationship used to develop the original LCN scale of relative loading severity was also modified as follows:

$$\frac{W_1}{W_2} = \left[\frac{A_1}{A_2}\right]^{0.27} \qquad (2)$$

This relationship was considered to be more appropriate for the aircraft which were in service at that time. The new LCN values derived from Equation 2 were different and unrelated to those derived in the original LCN system[†].

1.2.13. Although the LCG system was included in the 1977 edition of the ICAO* Aerodrome Design Manual[10] as one of the recommended methods for reporting pavement strength, it did not become popular outside the Department. The LCGs embraced too wide a range for practical use and the new LCNs were often confused with the previous LCN values.

1.2.14. Probably the most radical change in the 1971 publication was the formal recognition of the Department's construction practices, which had for some years discarded the conventional rigid and flexible pavement constructions by adopting cement- and bitumen-bound bases for flexible pavements and lean concrete bases for rigid ones. The design methodology for new pavement construction was modified to reflect these practices.

1.2.15. The 1971 design guide was substantially revised and updated and a new guide published in 1989[11]. The 1989 guide continued to build upon the development of previous concepts with the emphasis fixed firmly on the use of proven design techniques developed from past experience of pavement performance. Evaluation of concrete pavements nearing the end of their design life in the late 1970s indicated that the more frequent failure criterion was longitudinal halving cracking and this led to a more comprehensive fatigue model for calculating the allowable stress in rigid pavements.

1.2.16. The analysis was still based on Westergaard's theories but the design model was refined to include factors such as fatigue, growth in concrete strength with age and temperature warping stresses. The structural contribution of lean concrete bases was also re-appraised.

* ICAO – International Civil Aviation Organisation.

[†] All subsequent references to LCN are in terms of the 1971 LCN/LCG system.

1.2.17. The 1989 guide incorporated improvements to the CBR method based on full scale testing by the US Army Corps of Engineers[12] and introduced Equivalency factors for cement and bitumen bound base courses. Methods of equating multi-layer mixed constructions to model rigid or flexible constructions based on pavement performance experience were included.

1.2.18. The major change in the 1989 guide was the move from the previous LCN/LCG classification system to the ICAO Aircraft Classification Number - Pavement Classification Number (ACN-PCN) method linked to the design and evaluation methods.

1.3 CURRENT DESIGN PRACTICE

1.3.1. In 2006 a 2nd Edition of the guide was published, incorporating a number of developments in aircraft and airfield pavement construction and site investigations that had taken place since 1989. The 1989 guide was updated to cover:

(i) More damaging aircraft.
(ii) Tridem (6 wheel) main wheel gears.
(iii) Higher concrete strengths.
(iv) Increases in strength for Drylean Concrete in flexible pavements.
(v) Site investigation practice.

1.3.2. The basic design models were the same as those used for the 1989 guide. Detailed consideration was given to the adoption of a design methodology based on Multi-Layer Elastic Analysis. However, it was decided to maintain the traditional design methodologies because of the problems of dealing with joints in rigid pavements, material behaviour that changes significantly with trafficking, such as cement-bound bases in flexible pavements, and major aspects of pavement evaluation including multiple slab pavements and Type 2 and 3 composite pavements.

1.3.3. The use of high strength Drylean Concrete in flexible pavements was dealt with by the use of modified Equivalency Factors developed from full-scale testing of Drylean Concrete undertaken by Defence Estates and analysis by multi-layer elastic theory.

1.3.4. For tridem main wheel gear an additional rigid pavement design chart and new main wheel gear lines on the flexible pavement design charts were necessary because of the differences in the variation of the damaging effect with subgrade strength and coverages when compared to other main wheel gears. The use of the ACN for tridems on flexible pavements was complicated by the fact that at the time of writing ICAO had not formalised the calculation method. The flexible pavement design charts were based on ACNs for tridem main wheel gears calculated using the "interim" Alpha Factor promulgated by ICAO.

1.3.5. The key change in the 3rd Edition is the modification of flexible pavement ACNs following revisions to the ACN-PCN method promulgated by ICAO in September 2007. The revised ACNs have necessitated major changes to Charts 5, 6 and 8, which cannot be used with flexible pavement ACNs calculated using the original ACN-PCN method. In addition changes have been made to recommendations for longitudinal joint design and minimum top slab thicknesses for multiple slab pavements.

1.4 THE GUIDE

1.4.1. This guide supersedes all airfield pavements design and evaluation documents previously published by the Department. It is a development of the previous guide incorporating the latest pavement performance considerations, latest design thinking and advances in construction materials and aviation technology.

1.4.2. Many of the charts, figures and tables have been revised to accommodate recent developments such as the use of high strength drylean concrete, the emergence of larger, heavy aircraft with more complex main wheel gears, and changes to the ICAO ACN-PCN method. An additional Appendix on pavement structural investigation techniques has been included.

2 Classification of Aircraft and Airfield Pavements

2.1 GENERAL

2.1.1. Several methods of classifying the load ratings of aircraft and bearing strengths of airfield pavements have been in use for many years. The 1977 edition of the Aerodrome Design Manual, Part 3 published by the ICAO described four different methods which included the LCN and LCN/LCG systems originally developed in the UK. However, for safe and efficient use of airfield pavements, the ICAO has been striving to formulate a single universally accepted method of classification which would:

(i) enable aircraft operators to determine the permissible operating weights for their aircraft;

(ii) assist aircraft manufacturers to ensure compatibility between airfield pavements and the aircraft under development;

(iii) permit airport authorities to report on the aircraft they can accept and allow them to use any evaluation procedure of their choice to ascertain the loading the pavements can accept.

2.1.2. On 26 November 1981, the ICAO promulgated an internationally accepted reporting method known as the Aircraft Classification Number – Pavement Classification Number (ACN-PCN) method. Like the LCN and LCN/LCG systems the emphasis is on the evaluation of the load rating of aircraft, for which a standard procedure is specified, rather than evaluation of the pavement. The strength of the pavement is reported in terms of the load rating of aircraft which the pavement can accept on an unrestricted basis.

2.1.3. Following Defence Estates' tradition of using the aircraft classification as the load parameter for pavement design and evaluation, the ACN has been directly linked to the design and evaluation methods described in this guide. For pavements previously designed or classified in accordance with the LCN/LCG system, a procedure for conversion to PCNs is included in Appendix G. Since there is no precise relationship between LCN/LCG and PCN classifications, the conversions are only approximate.

2.2 DESCRIPTION OF THE ACN-PCN METHOD

2.2.1. A detailed description of the ACN-PCN method is given in the 1983 edition of the Aerodrome Design Manual, Part 3 published by the ICAO[11]. However, a brief description of the method and its application is given here.

2.3 AIRCRAFT CLASSIFICATION NUMBER (ACN)

2.3.1. The ACN of an aircraft expresses its relative loading severity on a pavement supported by a specified subgrade. ACNs are calculated using two mathematical models, one for rigid and the other for flexible pavements. The ACN of an aircraft is numerically defined as twice the single wheel load (expressed in thousands of kilograms) at a standard tyre pressure of 1.25MPa, which requires the same pavement thickness as the actual main wheel gear of the aircraft for a given limiting stress or number of load repetitions. The pavement thickness is known as the reference thickness.

2.3.2. The ACNs are reported separately for rigid and flexible pavements, four standard categories of subgrade (representing ranges of subgrade strength and characterised by a standard value at the middle of the range) and at Maximum Ramp Weight and a representative operating empty weight.

2.3.3. The method of calculating ACNs for aircraft on rigid pavements is set out below with reference to Figure 1:

(i) Calculate the reference thickness (t_c), the thickness of concrete slab which when loaded at the centre by one main wheel gear of the actual aircraft gives a maximum flexural stress of 2.75 N/mm^2 (f_{ct})* on a subgrade whose Modulus of Subgrade Reaction (k) is one of the standard values (see (iv)). The mathematical model for the stress calculation is the Westergaard solution for an elastic slab on a dense liquid subgrade (Winkler Foundation). The modulus of elasticity for concrete is taken as 27.6 x 10^3 MN/m^2 and Poisson's ratio as 0.15.

(ii) Calculate the single wheel load (W_R) which at a tyre pressure of 1.25MPa induces a flexural stress of 2.75N/mm^2, in slab of thickness t_c.

(iii) The ACN=2x $\frac{W_R}{1000} = \frac{W_R}{500}$ where W_R is in kgs.

(iv) Calculate ACNs for each aircraft for the following four categories of subgrade characterised in terms of a standard k.

Subgrade Category	**k**
High	150 MN/m^2/m
Medium	80 MN/m^2/m
Low	40 MN/m^2/m
Ultra Low	20 MN/m^2/m

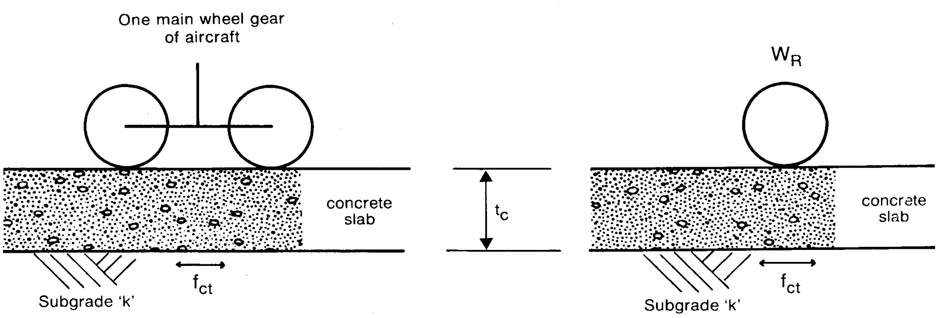


Figure 1 ACN Rigid pavement model

* (f_{ct}) – the flexural stress of 2.75 N/mm^2 for centre-case loading was selected by the ICAO to provide a realistic assessment of the relative loading severity of different aircraft in relation to thicknesses of rigid pavement construction on which they are likely to be operating. This may not necessarily be the allowable wheel load stress used in this guide, which varies depending on the flexural strength of the concrete and the load repetitions.

2.3.4. The method of calculating ACNs for aircraft on flexible pavements is set out below with reference to Figure 2:

(i) Calculate the reference thickness (t_f), the thickness of conventional flexible pavement which allows 10,000* load repetitions by one main wheel gear of the actual aircraft on a subgrade whose CBR is one of the standard values (see (iv)). The method of calculation is based on the CBR Equation and Boussinesq deflection factors.

(ii) Calculate the single wheel load (W_F) which at a tyre pressure of 1.25 MPa allows the same 10,000 load repetitions on a flexible pavement of total thickness t_f. The calculation is carried out using the following formula:

$$W_F = \frac{\frac{t_f^2}{200}}{\frac{0.878}{CBR} - 0.01249}$$

(iii) The ACN=2x $\frac{W_F}{1000} = \frac{W_F}{500}$ where W_R is in kgs.

(iv) Calculate ACNs for each aircraft for the following four categories of subgrade characterised in terms of a standard CBR.

Subgrade Category	CBR
High	15%
Medium	10%
Low	6%
Ultra Low	3%

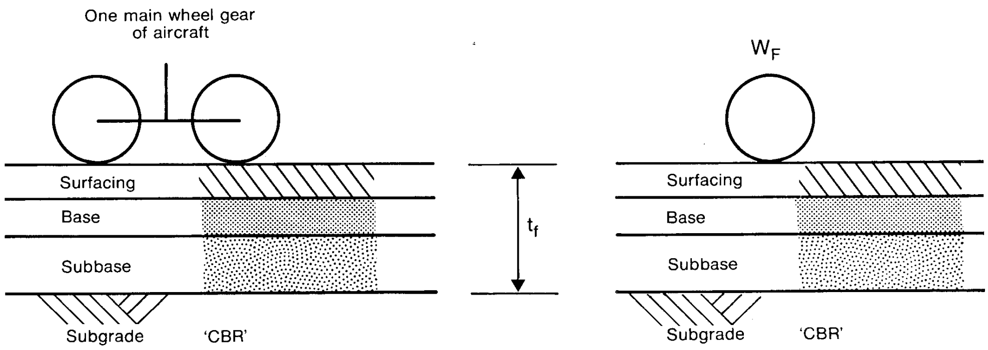


Figure 2 ACN Flexible pavement model

2.3.5. The ICAO has published ACNs for most civil aircraft[13]. For other aircraft, ACNs may be obtained from the manufacturers. A list of aircraft ACNs with main wheel gear types is given in Appendix B to this guide.

2.4 PAVEMENT CLASSIFICATION NUMBER (PCN)

2.4.1. By the definition of the ACN-PCN method, the PCN is the ACN of the aircraft which imposes a severity of loading equal to the maximum permitted on the pavement of unrestricted use.

2.4.2. PCNs are reported as a five part code as follows:

Part i — The PCN Number: The highest permitted ACN at the appropriate subgrade category.

Part ii — The type of pavement: R=rigid, F=flexible. If the actual pavement is of mixed construction the engineer will need to decide whether the behaviour and mode of failure of the pavement are likely to be those of a rigid or flexible one, then classify accordingly. For guidance on the classification of such pavements, see Chapter 7.

Part iii The pavement subgrade category:
A = High
B = Medium
C = Low
D = Ultra Low
The ranges of subgrade strength covered by these categories are shown in Table 1. Note that these strength ranges are not equivalent for rigid and flexible pavements.

Table 1 PCN Subgrade Categories

Subgrade Category	Pavement Type	Characteristic Subgrade Strength	Range of Subgrade Strengths
A – High	Rigid Flexible	150 MN/m²/m CBR 15%	All k values above 120 MN/m²/m All CBR values above 13%
B – Medium	Rigid Flexible	80 MN/m²/m CBR 10%	60 - 120 MN/m²/m CBR 8% to CBR 13%
C – Low	Rigid Flexible	40 MN/m²/m CBR 6%	25 to 60 MN/m²/m CBR 4% to CBR 8%
D – Ultra Low	Rigid Flexible	20 MN/m²/m CBR 3%	All k value below 25 MN/m²/m All CBR values below 4%

Part iv The maximum tyre pressure authorised for the pavement:
W = High, no limit.
X = Medium, limited to 1.5 MPa (217 psi)
Y = Low, limited to 1.0 MPa (145 psi)
Z = Very low, limited to 0.5 MPa (73 psi)
Refer to Chapter 8 for guidance on high tyre pressure operations.

Part v Pavement design/evaluation method:
T = Technical design or evaluation (see Chapters 5, 6 and 7 for detailed guidance).
U = By experience of aircraft actually using the pavement (see Appendix H for guidance

2.5 PAVEMENT CLASSIFICATION FOR LIGHT AIRCRAFT

2.5.1. The ACN-PCN method is not intended for reporting the strength of pavements meant for light aircraft, i.e. those with a weight less than 5700kg.

2.5.2. The bearing strength of a pavement intended for use by light aircraft should be classified in terms of the following data:

(i) Maximum allowable aircraft weight.
(ii) Maximum allowable tyre pressure.

2.6 THE DESIGN ACN

2.6.1. The design ACN, as used in this guide, is based on the Design Aircraft; which is normally the aircraft with the highest ACN on the actual subgrade.

2.6.2. The actual weight of aircraft when using the pavement must be considered in determining the design ACN. The Maximum All-Up Weight figure will normally be used, but lighter weights are appropriate (see also Section 4.8) where:

(i) the runway length imposes restrictions on the operating weights,
(ii) the pavement is only used by landing aircraft (e.g. fast turn offs) and
(iii) the pavement is only used by unladen aircraft (e.g. the accesses to maintenance hangars).

To compute an ACN at a weight between the published values it is assumed that ACNs vary linearly with weight.

2.6.3. The design ACN should also relate the actual value of the subgrade under a pavement. The ACNs listed in B are for four standard subgrade categories. If the value of actual subgrade is not the same as that of a standard subgrade, the design ACNs are to be calculated by linear interpolation or extrapolation of ACNs for the standard subgrades. The procedure is illustrated in Examples 2.1, 2.2 and 2.3.

2.6.4. The high category subgrade for flexible pavements is for CBR 15%. When designing pavements for subgrades with CBR greater than 15% the following rules may be applied:

(i) Single and Dual Main Wheel Gears

Take the ACN for CBR >15% to be the same as the ACN for CBR 15%.

(ii) Dual-Tandem Main Wheel Gears

Take the ACN for CBR ≥20% as equal to 0.95 x the ACN for CBR 15%.

Values for CBRs between 15% and 20% can be obtained by linear interpolation e.g.

ACN for CBR 17% = 0.98 x the ACN for CBR 15%.

(iii) Tridem Main Wheel Gears

Take the ACN for CBR ≥20% as equal to 0.97 x the ACN for CBR 15%.

Values for CBRs between 15% and 20% can be obtained by linear interpolation.

2.6.5. For rigid pavements, the effect of the higher subgrade values is less significant and it is therefore acceptable to assume that:
ACN for k>150 $MN/m^2/m$ = ACN for k of 150 $MN/m^2/m$.

2.6.6. For pavements which would subsequently be difficult to strengthen, it may be appropriate to design for a higher ACN e.g. for aprons adjacent to hangars and terminal buildings. Hangar floors designed in accordance with Chapter 5 will have an inbuilt element of over-design (see also para. 7.11).

2.6.7. Where a design ACN of less than 10 is being considered a check should be made to ensure that the pavement is strong enough for the expected use by aircraft servicing vehicles.

2.7 OVERLOAD OPERATIONS

2.7.1. Provided the PCN for a pavement is equal to or greater than the ACN of the aircraft and the operating tyre pressure does not exceed the PCN limitation, unrestricted use of the pavement by that aircraft (or those with lower ACNs) is permitted. The term 'unrestricted use' of a pavement is not specifically defined. However, it is a pavement design parameter which should reflect current and forecast use over an appropriate design life before major maintenance is required. See Chapter 4 for further guidance on pavement use and design life.

2.7.2. Unless a pavement is subject to extreme overloading it is unlikely to fail suddenly or catastrophically. Nevertheless regular overload operations can substantially reduce the design life of the pavement. The Aerodrome Authority may wish to carry out an assessment of the financial implications of increase maintenance or premature failure. Each aerodrome authority in the UK is free to decide on its own criteria for permitting overload operations as long as pavements remain safe for use by aircraft. See Chapter 8 for more detailed guidance on overload operations.

EXAMPLES ILLUSTRATING THE ASSESSMENT OF ACNs AND THE REPORTING OF PCNs

Example 1

Given a rigid pavement on a subgrade of k = 30 $MN/m^2/m$. The Design Aircraft for the pavement has been identified as the Boeing 747-400.

Determine the design ACN and the PCN for the pavement.

From Appendix B:

	Subgrade Category	
	Low (k 40 $MN/m^2/m$)	Ultra Low (k 20 $MN/m^2/m$)
B747-400	ACN 74.4	ACN 84.1

$$84.1-(84.1-74.4)\times\frac{(30-20)}{(40-20)}=79.75 \qquad (1)$$

(i) By interpolation, the design ACN for k = 30 $MN/m^2/m$ is:

(ii) Having designed or evaluated the pavement for ACN 70 at k = 30 the PCN is reported as follows:

From Table 1 the subgrade category is Low (i.e. k is between 25 and 60 $MN/m^2/m$) for which the code is 'C'. The PCN is reported as the ACN of the aircraft on the standard subgrade category, therefore assuming there is no tyre pressure limit for the concrete pavement, the PCN is *75/R/C/W/T.*

Example 2

Given a flexible pavement on a subgrade whose actual CBR is 5%. The Design Aircraft for the pavement has been identified as the Boeing 747-400.

Determine the design ACN and the PCN for the pavement.

From Appendix B:

	Subgrade Category	
	Low (CBR 6%)	Ultra Low (CBR 3%)
B747-400	ACN 72.5	ACN 94.1

(i) By interpolation, the design ACN at CBR 5% is:

$$94.1-(94.1-72.5)\times\frac{(5-3)}{(6-3)}=79.7 \qquad (2)$$

(ii) Having designed or evaluated the pavement for ACN 79 at CBR 5%, the PCN is reported as follows:

From Table 1 the subgrade category is Low (i.e. CBR is between 4% and 8%) for which the code is C. Assuming there is no tyre pressure limit the PCN is 73*/F/C/W/T*.

Example 3

Given an existing flexible pavement on subgrade which is known to be in the 'Low' category. Experience of aircraft use shows that B737-200s have regularly used the pavement without causing any apparent damage to it.

Determine the classification of that pavement.

From Appendix B, B737-200 on a Flexible Pavement Low subgrade has an ACN of 30.9.

If tyre pressure limit is 1.5MPa then the PCN is 31/F/C/X/U.

NB See Appendix H for advice on the reliability of classifications based on aircraft use.

3 The Subgrade

3.1 GENERAL

3.1.1. The subgrade is the natural soil or made-up ground which supports the pavement and the wheel loads imposed on it. The pavement spreads and thus reduces the high pressures immediately under the loaded areas to pressures which the subgrade can tolerate without unacceptable deformation. Thorough evaluation of the subgrade is very important, especially for flexible pavements where the required thickness depends greatly on the sheer strength of the soil. This evaluation of the subgrade includes the determination of subgrade strength and the assessment of factors which can affect the uniformity of the subgrade with time: e.g. shrinkage and swelling, frost action and mud pumping. It is also important to ascertain the vertical profile of the soil types, densities and moisture contents.

3.2 SOIL CLASSIFICATION AND EVALUATION OF SUBGRADE STRENGTH

3.2.1. Several soil classification systems have been developed in order to relate solid description to engineering properties. The most common is the extended Casagrande Soil Classification shown in Appendix A. The group symbols used for coarse-grained soils are derived from particle size distribution, and those for fine-grained soils are mainly derived from the plasticity index and liquid limit. The tests to assess these parameters are fully described in BS 1377-2: 1990,[15] while the Casagrande system is described in Reference 17. The Casagrande system enables the soil to be assessed for its likely behaviour as a subgrade, including its sheer strength, shrinkage, drainage properties and susceptibility to frost heave. Although an experienced engineer can often estimate the sheer strength and load/deflection values for a subgrade from the classification tests, it is often necessary to carry out further tests specifically to measure these characteristics.

3.2.2. The subgrade strength characteristics required for pavement design are the Modulus of Subgrade Reaction (k) and the California Bearing Ratio (CBR) for rigid and flexible pavements respectively. The design values chosen must be representative of the soil under the pavement after construction. Therefore, they should be based upon a relevant moisture content and density.

3.2.3. In selecting a design moisture content, consideration must be given to seasonal variations and the likelihood of the post-construction moisture content being higher than the pre-construction in situ value. There are some useful guidelines for certain conditions:

(i) A method of ascertaining the post-construction moisture content is to examine the subgrade under an existing adjacent pavement. The accuracy of the assessment will depend upon the similarity of pavement widths, subsoil drainage and permeability of the surface layers.

(ii) In very dry climates, if no water is present, the in situ value of the natural subgrade is likely to be representative.

(iii) In cohesive soils which are homogeneous with depth the moisture content at 1m down may be representative[17].

(iv) In the absence of any other information the moisture content of cohesive UK soils, except those containing a high proportion of montmorillonite, seldom exceeds the plastic limit plus 3%.

3.2.4. Selection of a representative density will depend on the in situ density, and the degree of compaction likely during construction (see Section 3.6).

3.2.5. The test for k is a large scale in situ test, which measures the behaviour of the subgrade as a whole and therefore tends to compensate for variations of density and moisture content with depth. The CBR test only measures the properties of a very small volume of the subgrade and it is more difficult to find a representative design value. However, in practice the Modulus of Subgrade Reaction test is difficult to carry out and in some situations it may be sufficient to assess k from the CBR value. Appendix A includes an approximate relationship between CBR and k. Use this with caution, particularly when considering soils uncommon in the UK (e.g. Laterites, corals and volcanic clinker/ash).

3.3 THE MODULUS OF SUBGRADE REACTION (K)

3.3.1. The Modulus of Subgrade Reaction k is determined from loading tests carried out on the subgrade using a standard 762mm (30in.) diameter plate. The plate is loaded to give increments of deflection of 0.25mm (0.01 in.). The pressure on the plate is plotted against settlement and the k value is taken as the slope of the line passing though the origin and the point on the curve corresponding to 1.27mm (0.05 in.) deflection. See Reference 17 for a full description of the test method.

3.3.2. As the 762 mm plate test is an in situ test it is difficult to ensure that the density and moisture content of the soils are appropriate to the post-construction conditions. It is best to do this test on a section prepared to the appropriate density (e.g. during compaction trails). An adjustment for moisture content is described in Reference 17.

3.4 THE CALIFORNIA BEARING RATIO (CBR)

3.4.1. The strength of the subgrade for flexible pavement s is measured in terms of the California Bearing Ratio (CBR) of the soil. The CBR test compares the force required to drive a plunger into the test material to a set penetration at a given rate, with the force required to cause the same penetration in a standard crushed limestone. A full description of the test is given in BS 1377-4: 1990. It is also possible to do field (in situ) CBR tests (BS 1377-9: 1990).

3.4.2. The laboratory CBR test should be carried out at a range of densities and, for each density, at a range of moisture contents. This gives a series of curves of CBR against moisture content from which a value applicable to the required condition can be obtained.

3.4.3. In conditions where it is difficult to choose a design moisture content, the test can be done on 4-day soaked samples in order to give a reasonably conservative value[18,19] These conditions could include:

(i) Subgrades where there is a considerable variation of moisture content with depth, in an otherwise homogeneous soil. This is likely when the water table lies near to or within the depth of soil being considered (i.e. the recommended depth of boreholes as shown in Table 4).

(ii) Areas where there is a large annual variation in moisture content due to a fluctuating water table, or possibly a spring thaw.

(iii) Tropical monsoon climates.

3.4.4. A surcharge should be applied in the CBR test to allow for the weight of the overlying soils and pavement construction. Defence Estates has adopted 6 kg as a standard surcharge weight.

3.4.5. When carrying out in situ CBR tests care should be taken to ensure that the density and moisture content are appropriate, as with the 762mm plate test. In situ tests are most useful for testing soils under existing pavements, but two points should still be considered:

(i) Stones close to the plunger area may produce unrealistically high results;

(ii) Because the test only affects a limited volume of the subgrade it will not include the presence of weaker underlying layers. It is therefore essential to know the soil profile at depth.

3.4.6. Laboratory tests on granular materials can give unrealistically high results because of the confining effect of the test mould. In situ tests may give lower figures but are often inappropriate because of the difficulty in testing at the relevant density and moisture content. The Casagrande Soil Classification can be used as a guide to selecting a design CBR value. It is recommended that the maximum design CBR values for flexible pavements are 20% for full-depth bound construction and 30% for unbound constructions (see para. 6.4.5).

3.4.7. Selecting a representative design CBR value can be difficult if the CBR varies considerably with depth. There is no problem if the CBR increases with depth as the critical value is the lowest one, i.e. at the formation. If the CBR decreases with depth (e.g. a layer of sand or gravel overlying a clay), designing on a high CBR value representative of the top layer could overstress the weaker underlying layer, but designing for the CBR of the lower layer will lead to an uneconomic pavement. In this situation Figure 7, Figure 8 and Figure 9 can be used to obtain an equivalent CBR for the two layer system. (See para. 3.8.3 and Example 3.2).

3.5 SUBSOIL DRAINAGE

3.5.1. Providing subsoil drainage may be desirable for several reasons:

(i) To increase subgrade strength by reducing the moisture content of the soils.
(ii) To reduce the chances of the moisture content increasing above that assumed in the selection of a design subgrade strength.
(iii) To drain the formation and pavement layers during construction.
(iv) To drain any unpaved shoulders after construction.
(v) To drain granular layers in an unbound pavement structure after construction. In this case the drainage is more likely to be essential rather than desirable as explained in para. 3.5.5.

3.5.2. There are a number of reasons for changes in the moisture content of subgrades, including:

(i) seepage flow from higher ground adjacent to the pavement.
(ii) changes in the water table level.
(iii) transfer of moisture to and from soil adjacent to the pavement.
(iv) percolation of moisture through the pavement.

3.5.3. Maximum benefit can be obtained from subsoil drainage if it is designed to reduce the moisture content of the soils prior to and during construction (e.g. by stopping seepage flow or lowering the water table). After construction the drainage should work to maintain the moisture content at or below that achieved during construction (e.g. by continuing to stop seepage flow, by preventing a rise in the water table or by removing water entering through the pavement or from the adjacent soil.)

3.5.4. It is possible to drain the formation and pavement layers during construction by shaping and by protecting the formation and installing subsoil drains before construction starts.

3.5.5. The large width of runways and other airfield pavements often makes it uneconomic to lower or control the water table because the shape of the draw-down curve would require drains to be installed at impracticable depths. In this case the pavement should be designed for a higher water table. However, it is important that the water table is kept at least 300mm below granular pavement layers to prevent them becoming saturated and to minimise the pumping of fines into the layers by repetitive aircraft loading. A geotextile fabric can also be used as a separator to control the latter problem. Ideally the same control of the water table level should be applied to other pavements to prevent undue deterioration of their materials. If necessary the formation should be elevated to raise the pavement far enough above the highest likely water table.

3.5.6. In assessing whether to install subsoil drainage, careful consideration should be given to the economic gains from potential benefits as compared to the cost of the system. Factors to be considered include the actual effectiveness of the system (which will partly depend on the permeability of the soil), the availability of a convenient outfall and the problems of installing drainage before the min construction starts.

3.6 COMPACTION OF THE SUBGRADE

3.6.1. With the exception of those soils listed in 3.6.4 (i) and (ii) the subgrade should be compacted to increase its density and sheer strength, and to prevent excessive settlement under traffic.

3.6.2. Control of settlement due to repetitive loading by traffic is achieved by obtaining specific relative compaction levels in the subgrade. (See Table 2 and Figure 3, Figure 4, Figure 5 and Figure 6). As the subgrade under a rigid pavement is less highly stressed than under a flexible one the relative compaction requirements are less stringent under rigid pavements. Figure 3, Figure 4, Figure 5 and Figure 6 were developed from various compaction trials.[20,21,22]

3.6.3. If the relative compaction requirement cannot be met, the subgrade should be removed and replaced with fill or overlaid with an additional layer of fill, sub-base or base material. The aim is that the uncompacted subgrade should be at a depth beneath the formation where the in situ relative compaction is equal to or greater than that required. This additional material can be taken as enhancing the subgrade, as long as the relative compactions still comply with those required at the new subgrade strength.

3.6.4. The amount of compaction possible in a soil will largely depend on the natural density and moisture content, but certain soils raise particular problems. These are:

(i) High and medium plasticity clays;
(ii) silts and very fine sands with a moisture content at or approaching saturation level;
(iii) uniformly graded non-cohesive materials.

3.6.5. High plasticity and some medium plasticity clays (see the Casagrande Soil Classification) are liable to show a serious decrease in strength when compacted at high moisture contents, especially when over consolidated. In the UK the natural moisture content of these soils is normally well above the optimum for heavy compaction so their undisturbed densities and strengths can rarely be improved by further compaction. In their undisturbed state, these soils give relative compactions ranging from 85-92% and CBRs ranging from 2-5% at typical moisture contents. From Table 2 and Figure 3, Figure 4, Figure 5 and Figure 6, these relative compactions are similar to or slightly lower than those required immediately under the pavement. However, experience in the UK has shown that rigid pavement with lean concrete bases constructed on medium and high plasticity clays provide good long-term performance without excessive settlement. It is therefore Defence Estates' practice to cause the least possible disturbance when constructing on these soils. Once exposed, the subgrade is usually covered as soon as possible to protect it from the weather and to provide a working area for further construction operations.

3.6.6. In tropical monsoon climates the compaction of high and some medium plasticity soils can present different problems (see also Section 3.10). In the dry season these soils will generally have a natural moisture content well below the optimum for heavy compaction, and thus if too highly compacted they are likely to swell in a later wet season. But if compacted at too high a moisture content, a low dry density will be achieved and the soil is likely to shrink during a dry period. Special care is therefore needed to achieve a moisture content and degree of compaction which reduces subsequent swelling or shrinkage to acceptable levels. In general the appropriate moisture content for compaction will be just above the optimum moisture content.

3.6.7. Silts and very fine sands with moisture contents at or approaching saturation level cannot be compacted. If it is not practical to drain these areas or remove and backfill them, the pavement design should be based on a very poor subgrade strength which reflects a saturated condition. With the pavement designs being based on a low CBR the density requirement is unlikely to be critical. To reduce the effect of poor and variable subgrade support however, a flexible or a rigid pavement design should incorporate a lean concrete base (See Chapters 5 and 6).

3.6.8. It is difficult to achieve compaction of uniformly-graded non-cohesive materials. One method of overcoming this is to compact through a thin layer (75-100mm) of a well-graded material. This layer will have no significant effect on the subgrade strength (CBR or k), which should be taken as that of the compacted underlying material.

3.6.9. To determine relative compaction requirements under flexible pavements using Figure 3, Figure 4, Figure 5 and Figure 6.

(i) select the relevant Figure for the soil type;
(ii) select the relevant main wheel gear type;
(iii) enter the design subgrade CBR on the left hand vertical axis;
(iv) make a horizontal projection to meet the relative compaction line;
(v) make a vertical projection to meet the design ACN;
(vi) make a horizontal projection to the right hand vertical axis and read off the depth requirement.

See Example 3.1 for an application of this procedure.

3.7 VERY WEAK SUBGRADES (EXCEPT PEAT)

3.7.1. Subgrades with CBRs less than 3% of k less than 20 $MN/m^2/m$ include saturated or nearly saturated high plasticity clays and silts. The support to the pavement provided by these soils is non-uniform. In the long-term the performance of the pavements will therefore be unpredictable and likely to be subject to premature localised failure.

3.7.2. Wherever practical these soils should be removed and backfilled with suitable fill material. As a lesser alternative Section 3.8 sets out a procedure for improving subgrade support by overlaying with suitable fill material. A thick layer of fill will provide a more uniform support to the pavement, although high plasticity clays may suffer long-term consolidation and loss of pavement shape.

3.8 SUBGRADE IMPROVEMENT

3.8.1. On poor subgrades an economic option may be to use suitable fill material which is available locally to improve the effective subgrade support to the pavement and thereby reduce the thickness of pavement required.

3.8.2. For flexible pavement design Figure 7, Figure 8 and Figure 9 set out a method of assessing the subgrade improvement provided by suitable fill material. Figure 8 and Figure 9 relate ACNs, existing subgrade CBRs, and thickness of fill material to an enhanced CBR design value at the top of the fill. The fill material must have a CBR value of not less than 15% at its anticipated equilibrium moisture content and must be compacted to the requirements of Table 2.

3.8.3. To determine the design CBR for a two layer subgrade where the CBR of the upper layer is greater than the CBR of the lower one:

(i) Select the relevant main wheel gear type.
(ii) On Figure 7 enter the CBR of the lower layer on the horizontal axis, make a vertical projection to meet the curve for the CBR of the upper layer ad then a horizontal projection to the vertical axis. Read off an Equivalency Factor from the vertical axis. This represents the load-spreading ability of the soil in the upper layer compared with that of a granular sub-base material.
(iii) Divide the thickness of the upper layer by the Equivalency Factor to obtain ‘t’. Calculate t^2/ACN on the vertical axis where ACN represents the loading severity of the Design Aircraft on the CBR of the lower layer.

(iv) On Figure 8 and Figure 9 enter the CBR of the lower layer on the horizontal axis and the value of t^2/ACN on the vertical axis. Make horizontal and vertical projections until they intersect. The design CBR on the subgrade is shown by the curve closest to the intersection.

See Example 3.2 for an application of this procedure.

3.8.4. For rigid pavement design, Figure 10 sets out a method for assessing subgrade improvement provided by a granular sub-base.

3.8.5. The pavement on the improved subgrade should then be designed for the ACN of the Design Aircraft corresponding to the uprated CBR or k value.

3.9 CONSTRUCTION PRACTICE

3.9.1. Experience has shown that if the moisture content of the subgrade is allowed to increase during construction the final equilibrium strength will be lower than if it had not. It is therefore important that the specification requirements for protecting the formation are compiled with, or the design CBR value should be reduced accordingly.

3.9.2. Construction traffic can damage or reduce the natural strength of the subgrade. The use of the formation in areas of cut should be restricted to the minimum plant and equipment essential for the overlying construction. For subgrades particularly prone to damage (e.g. high plasticity clays and silts) a working course of drylean concrete or granular sub-base/capping layer should be placed on the subgrade before construction continues. In fill areas construction traffic should be restricted to prevent damage to compacted layers and the subgrade. To allow reshaping and recompaction, rut depths in granular layers should not exceed about 40mm.[23]

Table 2 Relative Compaction Requirements for Subgrades

PAVEMENT TYPE	FILL/EMBANKMENT AREAS		CUT AREAS	
	COHESIVE	NON-COHESIVE	COHESIVE	NON-COHESIVE
Rigid incorporating a strong cement-bound base	90%	95%	The top 150mm If k ≥ 40 – 90% k < 40 – 85%	The top 600mm If k ≥ 50 – 95% If k < 50 – 90%
Rigid without strong cement-bound base	90%	The top 150mm – 98% The remainder – 95%	The top 150mm If k ≥ 40 – 85% If k < 40 – 80%	The top 150mm If k ≥ 50 – 98% If k < 50 – 95% Between 150mm and 600mm If k ≥ 50 – 95% If k < 50 – 90%
Flexible	The top 225mm – 95% The remainder – 90%	The top 225mm – 98% The remainder – 95%	Refer to Figure 3 and Figure 4	Refer to Figure 5 and Figure 6

Notes to Table 2 and Figure 3, Figure 4, Figure 5 and Figure 6

(i) For the purpose of determining relative compaction requirements non-cohesive soils are those for which the fraction passing the 425 micron sieve size has a plasticity index (P1) of less than 6.

(ii) The density requirements are expressed as a percentage of the maximum dry density given by BS 1377-4: 1990, Section 3.5 or 3.6.

(iii) The compaction requirements in Figure 3, Figure 4, Figure 5 and Figure 6 apply to natural subgrades below flexible pavements. The relative compaction required at a particular depth in the subgrade is a function of the vertical stress induced at that depth by the aircraft wheel loads and the number of load repetitions over the life of the pavement.

(iv) See Section 3.7 for subgrades less than CBR 2%.

(v) Subgrades which cannot realistically be compacted to the requirements in Table 2 and Figure 3, Figure 4, Figure 5 and Figure 6 should be removed and replaced with fill or overlaid with an additional depth of fill, sub-base or base material. This additional depth of construction should be sufficient to ensure that the requirements for relative compaction with depth beneath the pavement are achieved.

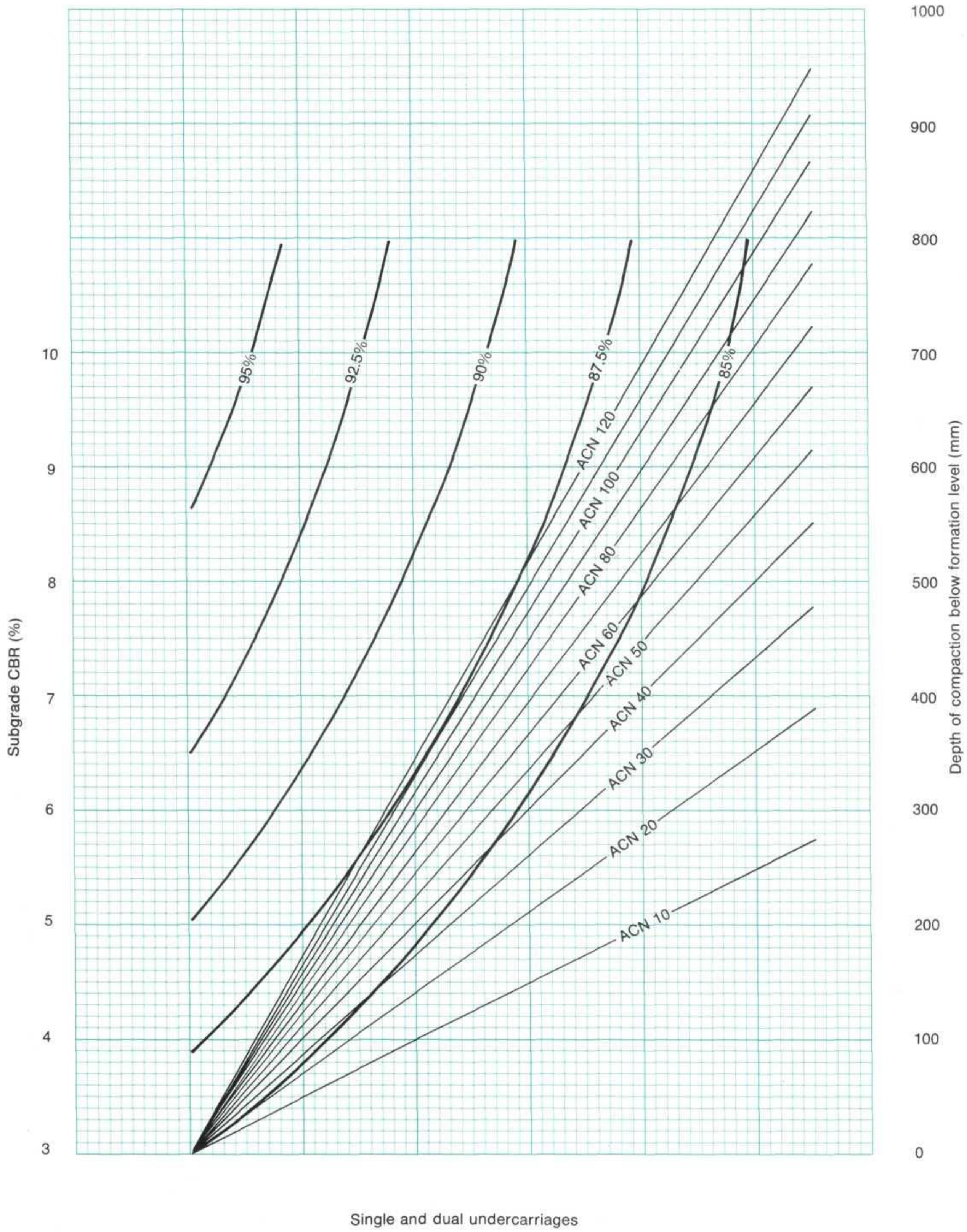


Figure 3 Relative compaction requirements for subgrades under flexible pavements - Single and dual main wheel gears - Cohesive soils

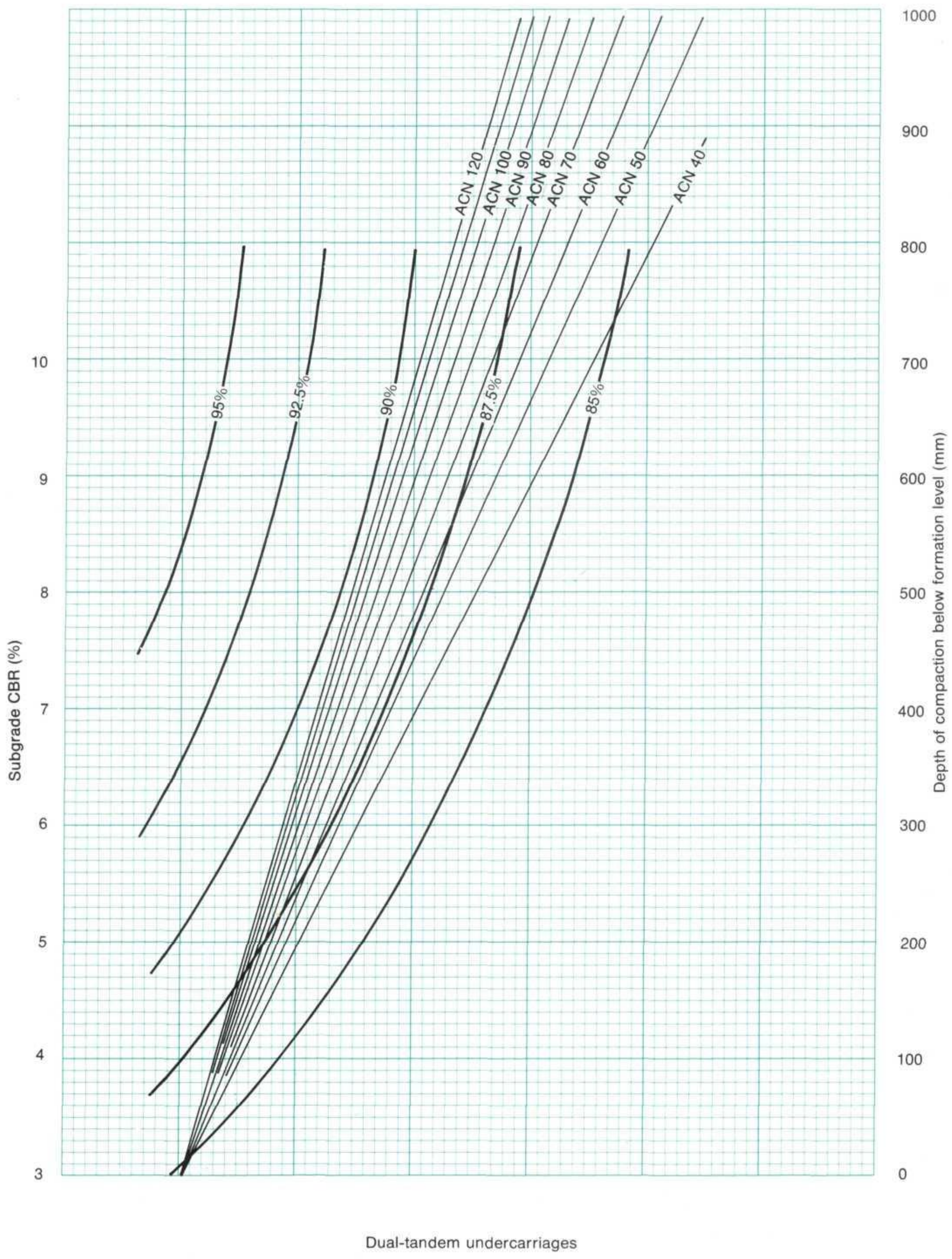


Figure 4 Relative compaction requirements for subgrades under flexible pavements - Dual-tandem and tridem main wheel gears - Cohesive soils

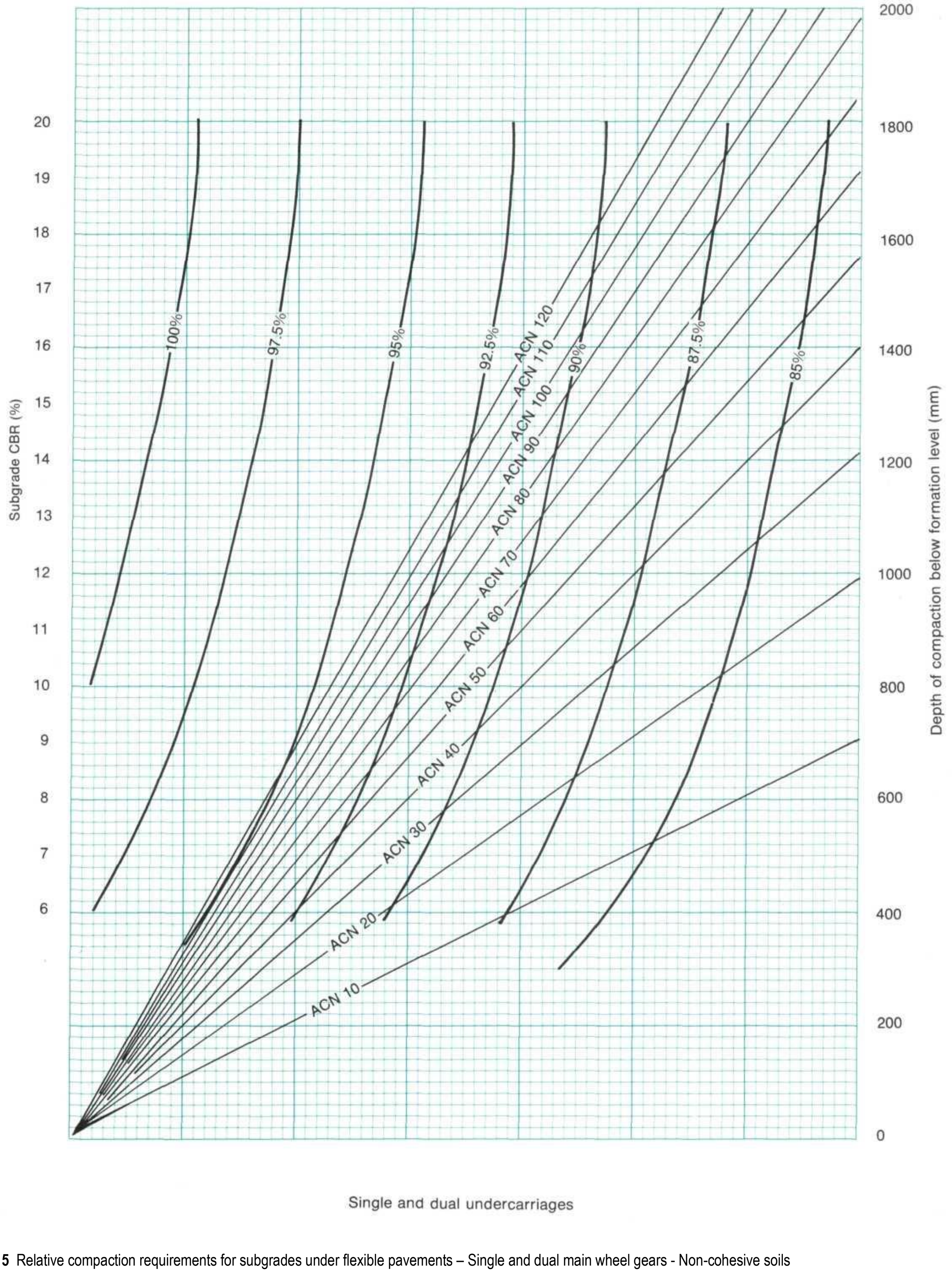


Figure 5 Relative compaction requirements for subgrades under flexible pavements – Single and dual main wheel gears - Non-cohesive soils

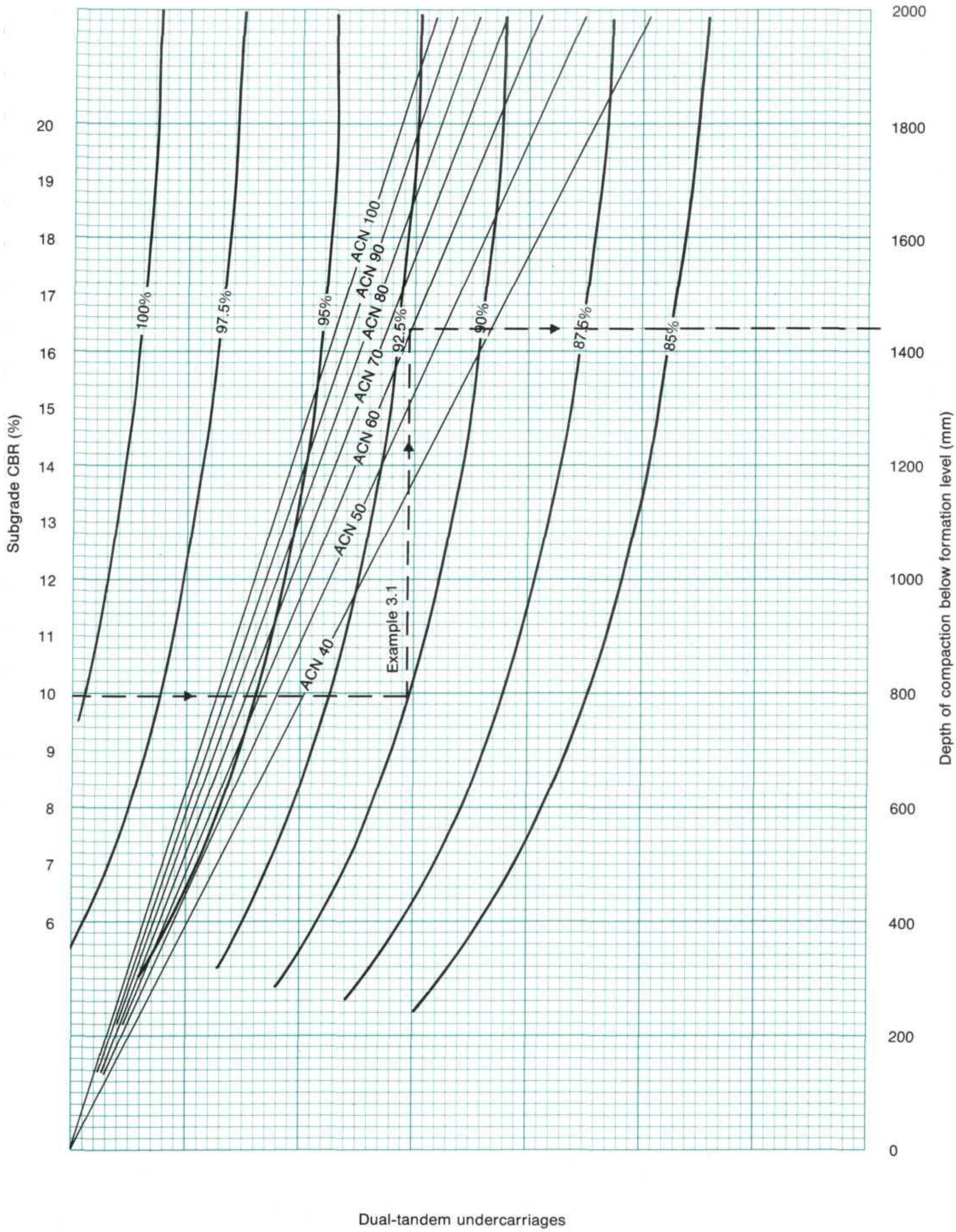


Figure 6 Relative compaction requirements for subgrades under flexible pavements – Dual-tandem and tridem main wheel gears - Non-cohesive soils

3.10 EXPANSIVE SOILS

3.10.1. Some soils can show large volume changes when the moisture content changes. This can lead to loss of uniform support to the pavement, a reduction of bearing capacity of the soil, and bumps, hollows and cracks in the pavement. Generally the problem is only severe in climates where a long hot dry period is followed by a rainy season; the subgrade dries and shrinks during the hot season, but then expands rapidly as the rainy season increases the moisture content. As an appropriate the Plasticity index gives a good indication of the expansive nature of a soil; values less than 20 are non-expansive; between 20 and 40 are moderately expansive; and above 40 can be highly expansive. For more accurate assessment a technique related to the shrinkage limit and expected range of moisture content is described in Reference 18. Problems can also occur if an expansive soil is compacted in too dry a condition or allowed to dry out during construction.

3.10.2. The effect of expansive soils can be much reduced by careful control of moisture content during construction and the degree of compaction achieved (see para 3.6.6). If future expansion is still likely to be excessive, soil swell can be limited by, for example, providing sufficient fill/overburden.

3.11 FROST ACTION

3.11.1. For the UK and similar climates, material within 450mm of the pavement surface should not be susceptible to frost. Where the subgrade is frost susceptible the thickness of the base/sub-base must be increased if the proposed total thickness of construction is less than 450mm.

3.11.2. Tests for frost susceptibility has been carried out by TRL on a variety of materials used as subgrades, sub-bases and bases both in research and during routine testing for motorway and trunk road projects. Test results and other aspects of frost susceptibility are contained in TRL Report No LR90.[24] The Frost Test method described in LR90 was latter updated by TRL[26]. The current test method is given in BS 812-124:1989.

3.12 PEAT

3.12.1. Subgrades of peat are highly compressible and have very little bearing capacity. Pavements constructed on them can suffer from serious differential settlement, so peat should usually be removed and replaced with a suitable fill. A possible option is to surcharge the peat with fill for a long time to reduce the short term consolidation substantially. But this makes a long and phased construction necessary and in the long term the performance of the pavement will be unpredictable; there will probably be localised failures and general loss of shape. This alternative should not be used for pavements whose longitudinal and transverse profiles are critical; e.g. runways and major taxiways. Consider it, however, for stopways.

3.13 SPRING THAW AND PERMAFROST

3.13.1. In certain parts of the world where frost conditions are severe, pavements must be designed for the effects of spring thaw and permafrost. Both the spring thaw and intermittent or partial melting of a permafrost layer can considerably reduce the load-carrying capacity of the pavement.

3.14 GROUND INVESTIGATION

3.14.1. It is essential that an adequate ground investigation is carried out to obtain the necessary soils information. Recommendations for the spacing and depth of trial pits or boreholes are given in Table 3 and Table 4 Groundwater movements should be monitored over a suitable period, preferably at least one year.

Table 3 Frequency of Trial Pits/Boreholes

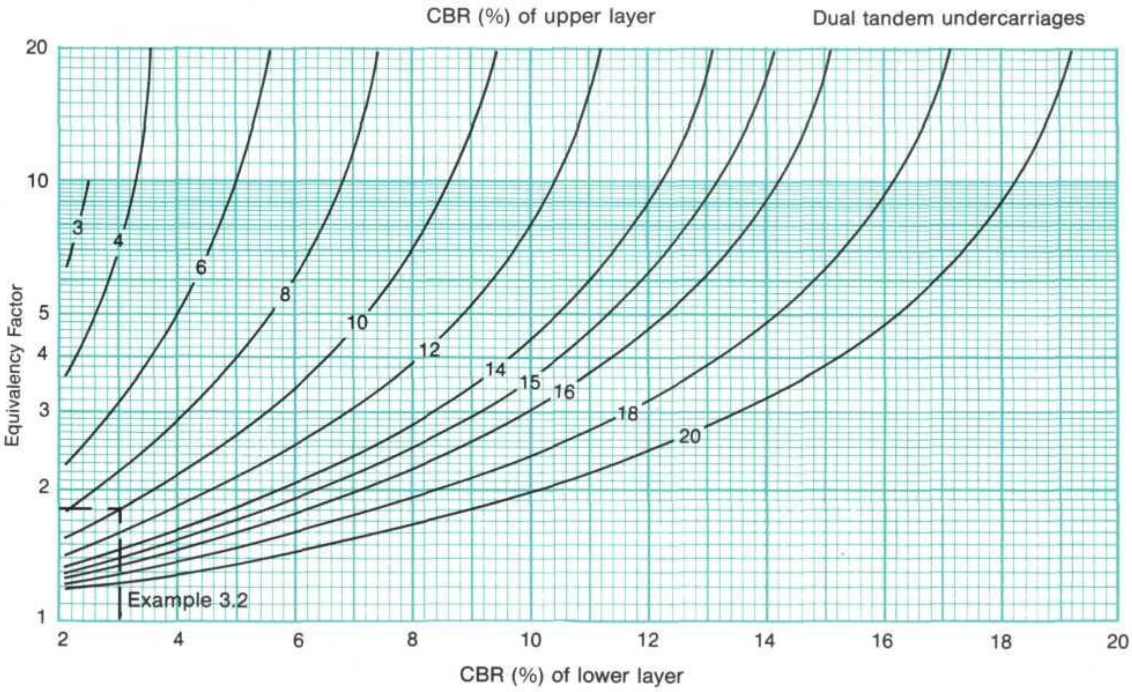
Location	Frequency
Runways/Taxiways	1 every 50m staggered across centre line
Aprons and other areas	To be positioned on a 30m square grid.

Table 4 Depth of Trial Pits/Boreholes (mm)

(Below proposed formation in areas of cut and existing ground level in areas of fill)

ACN of the Design Aircraft on a Flexible Ultra Low Subgrade	Subgrade Category (as ACN-PCN method)			
	Ultra Low	Low	Medium	High
20	600	800	1000	1000
40	800	1200	1400	1400
80	1500	2000	2200	2400
120	1800	2400	2600	3000

NB: If it is certain that the construction will be a rigid pavement then the depth can be reduced to 50% of these figures, subject to a minimum of 600mm.



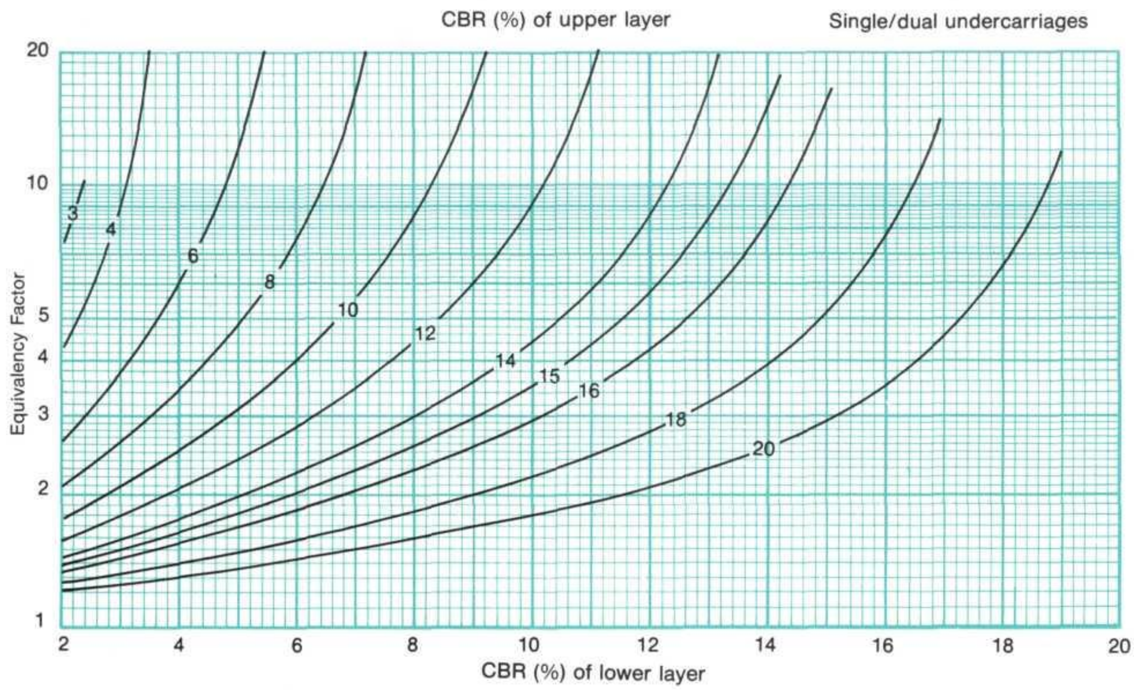


Figure 7 Equivalency factors for the estimation of a design CBR on a layered subgrade

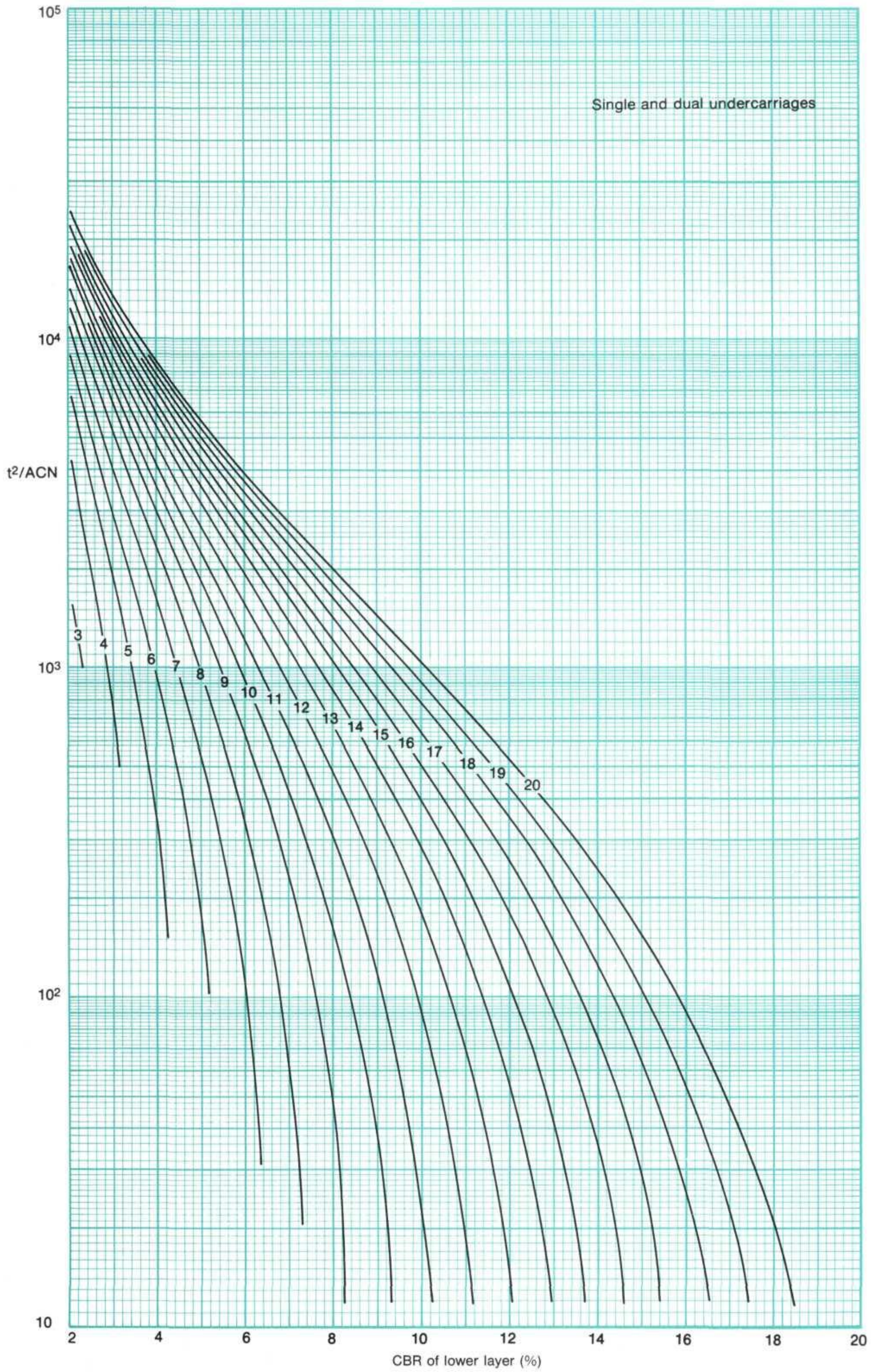


Figure 8 Estimation of a design CBR on a layered subgrade – Single and dual main wheel gears

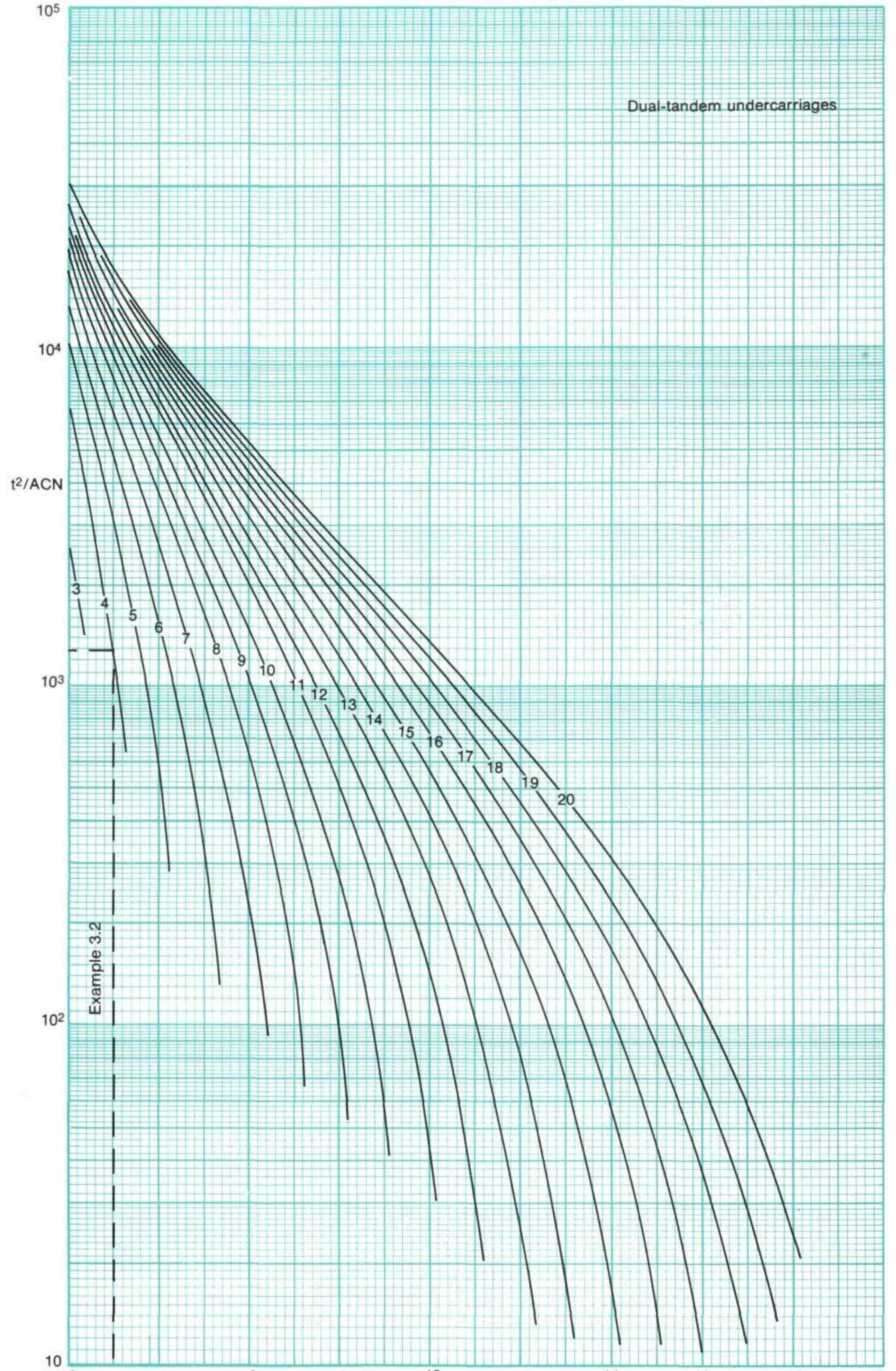


Figure 9 Estimation of a design CBR on a layered subgrade - Dual-tandem and tridem main wheel gears

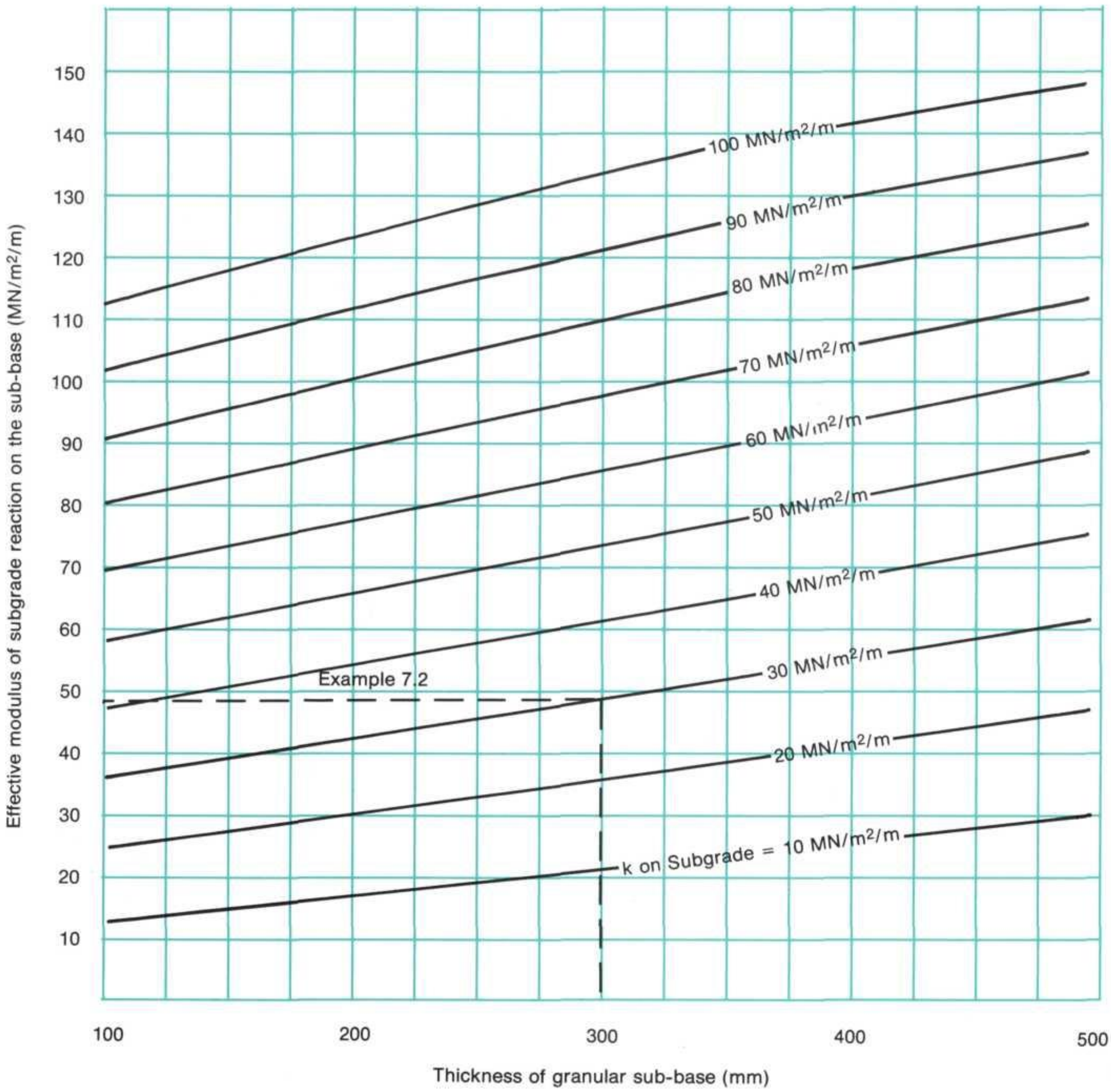


Figure 10 Effect of granular sub-base on the modulus of subgrade reaction (k) for rigid pavements

SUBGRADE EXAMPLES

Example 3.1

A flexible pavement is to be constructed on a sand subgrade with a design CBR of 10%. The Design Aircraft has an ACN of 60, and a dual-tandem main wheel gear. Assess the Relative Compaction requirements.

Using Figure 6

Relative Compaction	Depth below formation (mm)	
100%	0-100	
95%	100-800	
90%	800-1450	(see Example Lines on Figure 6)
85%	1450-2000	

Example 3.2

A subgrade consists of 500mm sand, CBR 10%, overlying a CBR 3% clay. Using Figure 7 and Figure 9 find a design CBR for a flexible pavement for an aircraft with an ACN of 60 on CBR 3% and a dual-tandem main wheel gear. (See para 3.8.3 for a description of the method).

(i) Equivalency Factor = 1.8
(ii) $t = 500/1.8 = 278$
(iii) $t^2/ACN = 278^2/60 = 1286$
(iv) Design CBR for the subgrade is 4%.

4 Design Considerations

4.1 DESIGN PARAMETERS

4.1.1. The design of a new pavement requires information on the following parameters:

(i) Pavement type – rigid or flexible
(ii) Quality of the pavement materials including the flexural strength of concrete
(iii) Subgrade strength
(iv) Design ACN
(v) Frequency of Trafficking. This is derived from a number of factors including
   a. The Design Life
   b. The pattern of trafficking and assessment of passes.
   c. Coverages and Pass-to-Coverage ratio.
   d. Mixed Traffic Analysis if there is more than one significant aircraft.

4.2 TYPES OF PAVEMENT

4.2.1. The design and classification method presented in this document requires a distinction between rigid and flexible pavements as described below.

4.2.2. A rigid pavement comprises either wholly or partly concrete construction which can be plain, reinforced or prestressed and which distributes the aircraft loading to the subgrade by means of its high flexural stiffness. Chapter 5 gives a design method for the preferred new rigid pavement constructions.

4.2.3. A flexible pavement is composed of bound or unbound granular materials. It distributes the aircraft load primarily through the shear strength of the paving material. Cement-bound granular bases beneath bituminous surfacings make pavements quite rigid in their early years. However, for reasons discussed in para 6.3.7 this type of construction is treated as a flexible pavement for design and evaluation purposes. Chapter 6 gives a design method for the preferred new flexible pavement constructions.

4.2.4. Chapter 7 includes procedures for the design or evaluation of the following pavement constructions:

(i) Traditional flexible constructions incorporating unbound granular bases and sub-bases.
(ii) Traditional concrete pavements laid directly on the subgrade or on a granular sub-base.
(iii) Composite pavements – these comprise flexible-on-rigid construction and are generally the result of various strengthening and maintenance overlays.
(iv) Multiple concrete slab construction – like composite pavements they have generally evolved through strengthening overlays.
(v) Overlays and overslabs required for strengthening existing pavements.

4.2.5. The choice of pavement type depends on performance requirements and cost.

4.2.5.1 Performance requirements: In general, concrete is preferred where there is likely to be venting of fuel, spillage of lubricating oils and hydraulic fluids, jet efflux gases from slow moving high performance jet engines, or areas subject to locked wheel turns. Concrete should therefore be used for the following pavement areas:

(i) Runway ends (typically for a distance of at least 150m).
(ii) Sections of taxiways adjacent to runway ends.
(iii) Holding areas.
(iv) Aprons and hard standings.
(v) Hangar floors.
(vi) Engine run-up platforms.
(vii) Compass calibration bases.

4.2.5.2 Cost: For many pavements this will be the main consideration and will depend on such diverse factors as the availability of materials in the locality and the bearing capacity of the natural subgrade on which the pavements are to be constructed. For rigid pavements there is a minimum thickness of concrete below which its use is impractical, and a maximum subgrade strength beyond which further increases in strength result in little saving of construction depth. On soils of good bearing value, flexible construction is likely to be more economical. The opposite is true for weak subgrades.

4.2.5.3 Other considerations:

(i) The absence of joints in flexible pavements gives them better riding qualities for high speed operations than most types of rigid pavement.
(ii) If the only realistic option is to construct a pavement on an unpredictable subgrade which is liable to long-term shrinkage or heave, a flexible pavement will generally be the best option. This is because a flexible pavement can cope with greater movement and remain serviceable; it can also be more cheaply and expediently overlaid to rectify the loss of shape.

4.3 MATERIAL SPECIFICATION

4.3.1. The use of the semi-empirical design methods demands that the quality of material in a pavement is at least as good as those in the pavements upon which the design methods are based. This applies to the material specification and the level of quality control.

4.3.2. Relevant details of the necessary material specification are given in Chapter 5, 6 and 7, and Appendix C.

4.4 SUBGRADE STRENGTH

4.4.1. The determination of subgrade strength, and the other subgrade properties to which consideration should be given during design, is described in Chapter 3. Some specific considerations with respect to rigid and flexible pavements are discussed in Sections 5.5 and 6.4 respectively.

4.5 THE DESIGN ACN

4.5.1. The method of determining the design ACN for a pavement is given in Chapter 2.

4.6 FREQUENCY OF TRAFFICKING

4.6.1. While the magnitude and configuration of the wheel loads are the dominant factors in the design of airfield pavements, the effect of fatigue caused by load repetition is an important secondary consideration for both rigid and flexible pavements. Laboratory and full-scale tests clearly show that pavements subject to high frequencies of trafficking need to be significantly thicker than those subject to low frequencies.[12,27]

4.6.2. The design methods given in this guide cater for 3 frequencies of trafficking: Low, Medium and High, as shown in Table 5.

Table 5 Design Frequency of Trafficking

Frequency of Trafficking	Nominal Number of Coverages* over Design Life of Pavement
Low	10,000
Medium	100,000
High	250,000

*The definition of 'Coverages' is given in Section 4.9

4.6.3. To determine the appropriate frequency of the trafficking, the total number of Coverages during the design life is calculated. This involves consideration of the design life, pattern of trafficking and mixed traffic use.

4.7 DESIGN LIFE

4.7.1. The design method and the frequencies of trafficking in Table 5 assume the aircraft movements are spread fairly evenly over the life of the pavement

4.7.2. In normal circumstances pavement deterioration is gradual, becoming noticeable over a period of a few years. This deterioration can be due to surface weathering or structural fatigue or both. In deciding on an appropriate structural design life, the following considerations should be kept in mind:

(i) The need to keep major maintenance work on airfield pavements to a long term cycle.
(ii) The likelihood of a change in aircraft use after a number of years.
(iii) Durability of pavement construction. Concrete pavements are more durable than blacktop pavements assuming both are constructed in accordance with Defence Estates' Specification. The surface serviceability of concrete should, with the aid of minor maintenance work, be adequate for 25-35 years. On the other hand bituminous surfacings, as a result of surface weathering, generally require maintenance work in the form of slurry sealing, the first coat being required after 7-10 years, and more substantial restoration work after 20-25 years. In the case of friction case resurfacing may be required after approximately 15 years.
(iv) The cost of rehabilitation. Concrete pavements generally cost more to rehabilitate than flexible pavements.

4.7.3. With these factors in mind it is recommended that the structural design life be 20-30 years. The upper end of this range being for concrete pavements and the lower end for flexible pavements.

4.7.4. The design method assumes an increasing degree of minor maintenance (e.g. crack sealing) in the last few years of a pavement's life. Where such maintenance cannot be tolerated, the engineer may wish to project a structural design life beyond the expected life of the surfacing.

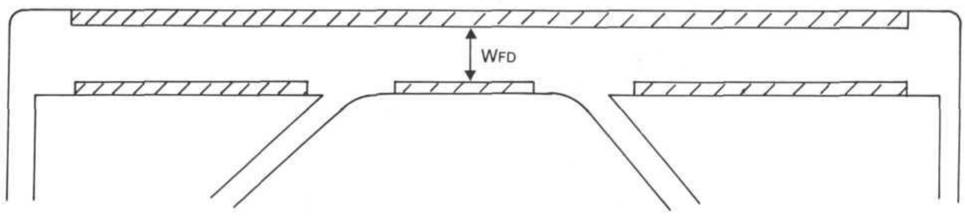
4.8 PATTERN OF TRAFFICKING AND ASSESSMENT OF PASSES

4.8.1. An aircraft movement over a particular section of the pavement normally constitutes a pass. The total number of passes should be taken as the total number of movements and Mixed Traffic Analysis used to consider the effect of aircraft operations at different weights. It is conservative to consider all movements at Maximum Ramp Weight. If it is certain that actual operations (e.g. landings) will always be at weights lower than this figure a more accurate weight can be used (e.g. for fast turn offs, accesses to maintenance areas and where runway length restricts Maximum Take Off Weight).

4.8.2. Runways and main taxiways leading to runway ends are the most heavily loaded pavements as they carry the aircraft at their heaviest, when fuelled for take off. For these pavements the number of passes can be taken as the number of departure movements only; landing movements being accounted for by assuming that all passes are at Maximum Ramp Weight.

4.8.3. Aircraft parking aprons in rigid pavement construction should be designed for the same design ACN and frequency of trafficking as the main taxitrack exit from the apron. This is because it is difficult to predict movement patterns and to construct areas of concrete in varying thicknesses.

4.8.4. The outer portions of runways can be designed to a reduced loading regime as shown in Figure 11. However, where an airfield does not have a parallel or perimeter taxiway, the assessment of the loading regime should include the additional use of the runway for taxiing operations. 'Backtracking' (taxiing) down the runway by departing aircraft will approximately double the Coverages (as defined in Section 4.9) on the runway. In addition, the length of runway used by backtracking aircraft should be provided with the same full depth construction across the width of the pavement to allow for taxiing being offset from the centreline.



Key

Construction required for the Design ACN

Construction depth can be linearly reduced to the edge of the runway where the minimum depth should be that required for two thirds of the Design ACN.

Aircraft wheel track	WFD
Less than 8m	15m
8-12m	20m

Figure 11 Reductions in runway thickness requirement

4.8.5. Reduction in construction thickness on the outer strips of runways, is particularly beneficial when strengthening existing runways which have an inadequate camber. The reduced thickness at the edge will allow improved transverse gradients and surface water drainage.

4.8.6. On helicopter pads and Harrier VTOL pads the dynamic effects of landing aircraft increase the loading factor. For these pavements the passes should be taken as the number of take offs plus the number of landings at the ACN appropriate to the maximum weight; the Pass-to-Coverage Ratio listed in Table 6 and Table 7 should also be adopted.

4.9 Coverages and Pass-to-Coverage Ratio

4.9.1. Coverages represent 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 relationship between passes and Coverages depends on several factors, including the number and spacing of wheels on the aircraft's main wheel gear, the width of the tyre contact area and the lateral distribution of aircraft wheel paths relative to the pavement centre-line or guideline markings. The number of Coverages is calculated using the Pass-to-Coverage Ratio:

$$\text{Coverages} = \frac{\text{Passes}}{\text{Pass-to Coverage Ratio}}$$

4.9.2. Table 6 and Table 7 give Pass-to-Coverage Ratios for various main wheel gear arrangements on runways, taxiways and stands. These ratios assume channelised trafficking consistent with the initial stage of a take off run on runways, very channelised trafficking about a centreline on taxiways, and operations on stands with designated stand centrelines, especially when controlled by docking guidance systems. For aprons without stand centrelines where the actual parking position varies the Pass-to-Coverage Ratios for taxiways should be used.

4.9.3. For the background to the derivation of the Pass-to-Coverage Ratios see Appendix E which also sets out a procedure for calculating Pass-to-Coverage Ratios for non-standard wheel gear arrangements.

Table 6 Pass to Coverage Ratios

Main Wheel Gear Type*	Pass-to-Coverage Ratio		
	Runway	Taxiway	Stand
Single	See Table 7		
Dual	3.2	2.1	1
Dual-Tandem	1.8	1.31	0.5
Tridem	1.44	1	0.33

* Refer to Appendix D for definition of landing gear arrangements.

Table 7 Pass-to-Coverage Ratios for Aircraft with Single Main Wheel Gears

Tyre Pressure MPa	ACN of Aircraft								
	Up to 10		11-20		21-40		Over 40		All
	Runway	Taxiway	Runway	Taxiway	Runway	Taxiway	Runway	Taxiway	Stands
Up to 1.0	8	4	6	3	5	2.5	4	2	1
1.0 to 1.5	10	5	8	4	6	3	5	2.5	1
>1.5	12	6	10	5	7	3.5	6	3	1

4.10 MIXED TRAFFIC USE

4.10.1. At a military airfield the pavements are often designed for operations by a specific type of aircraft, so the calculation of the loading regime is relatively straightforward. However, where traffic forecasts indicate operations by a variety of aircraft, the loading criteria will not be so readily assessed. In allowing for a variety of aircraft types it is necessary to be able to relate the loading severity of each type of aircraft to that of the Design Aircraft and thereby to calculate the number of Equivalent Coverages by the Design Aircraft.

4.10.2. The calculation of the loading regime for pavements subject to mixed traffic is explained with reference to Examples 4.1 and 4.2:

(i) Decide on the required design life of the pavement (see Section 4.7).
(ii) Establish the aircraft types likely to use the pavement.
(iii) Establish the ACNs for each aircraft at the actual subgrade value and the appropriate weight.
(iv) Use Appendix B to identify the main wheel gear type for each aircraft and establish their Pass-to-Coverage Ratios from Table 6 and Table 7 (see Section 4.9).
(v) Establish the number of passes by each aircraft.
(vi) Establish the Design Aircraft.
(vii) For setting out the information refer to Table 8 and Table 9 of Examples 4.1 and 4.2 respectively. The tables show the aircraft (col 1), their ACNs (col 2), Pass to Coverage Ratios (col 3) and annual passes (col 4).

(viii) Calculate the number of Coverages by each aircraft during the design life of the pavement (col 5).

(ix) Calculate the ratio of the ACN of each aircraft to that of the Design Aircraft (col 6).

(x) For rigid pavements, use Figure 12 to obtain rigid mixed traffic factors (RMTF) from the ACN ratios found in step (ix). For each aircraft, select the ACN of the Design Aircraft on the left-hand ordinate and make a horizontal projection until it intersects the curve with the appropriate ACN ratio. Make a vertical projection down the graph and read off the RMTF. See Table 8 col 7. Having established the RMTF for each aircraft the number of Equivalent Coverages by the Design Aircraft is equal to the number of Coverages made by each aircraft divided by its respective RMTF (Table 8 col 8) Hence:

$$\text{Equivalent Coverages by Design Aircraft} = \frac{\text{Coverages by aircraft at less than the design ACN*}}{\text{RMTF}}$$

(xi) For flexible pavements, use Figure 13 to obtain flexible mixed traffic factors (FMTF) from the coverages found in step (viii). For each aircraft, select its respective number of Coverages (Table 9 col 5) on the abscissa of Figure 13. Then make a vertical projection until it intersects the curve. Make a horizontal projection and read off the FMTF from the left-hand ordinate. See Table 9 col 7. Modify the FMTF for each aircraft by multiplying it by the respective ACN ratio (Table 9 col 8). Select the Modified FMTF on the left-hand ordinate of Figure 13. Using the graph in reverse, read off the number of Equivalent Coverages by the Design Aircraft (Table 9 col 9).

(xii) Column 8 in Table 8 and column 9 in Table 9 give the mixed traffic loading in terms of Equivalent Coverages by the Design Aircraft. Calculate the total Coverages at the design ACN from:

Total Coverages at the design ACN = Coverages by the Design Aircraft at the design ACN plus the Equivalent Coverages

(xiii) From Table 5, select a frequency of trafficking to use as an input to the design charts.

* This could include the Design Aircraft at weights other than the maximum considered.

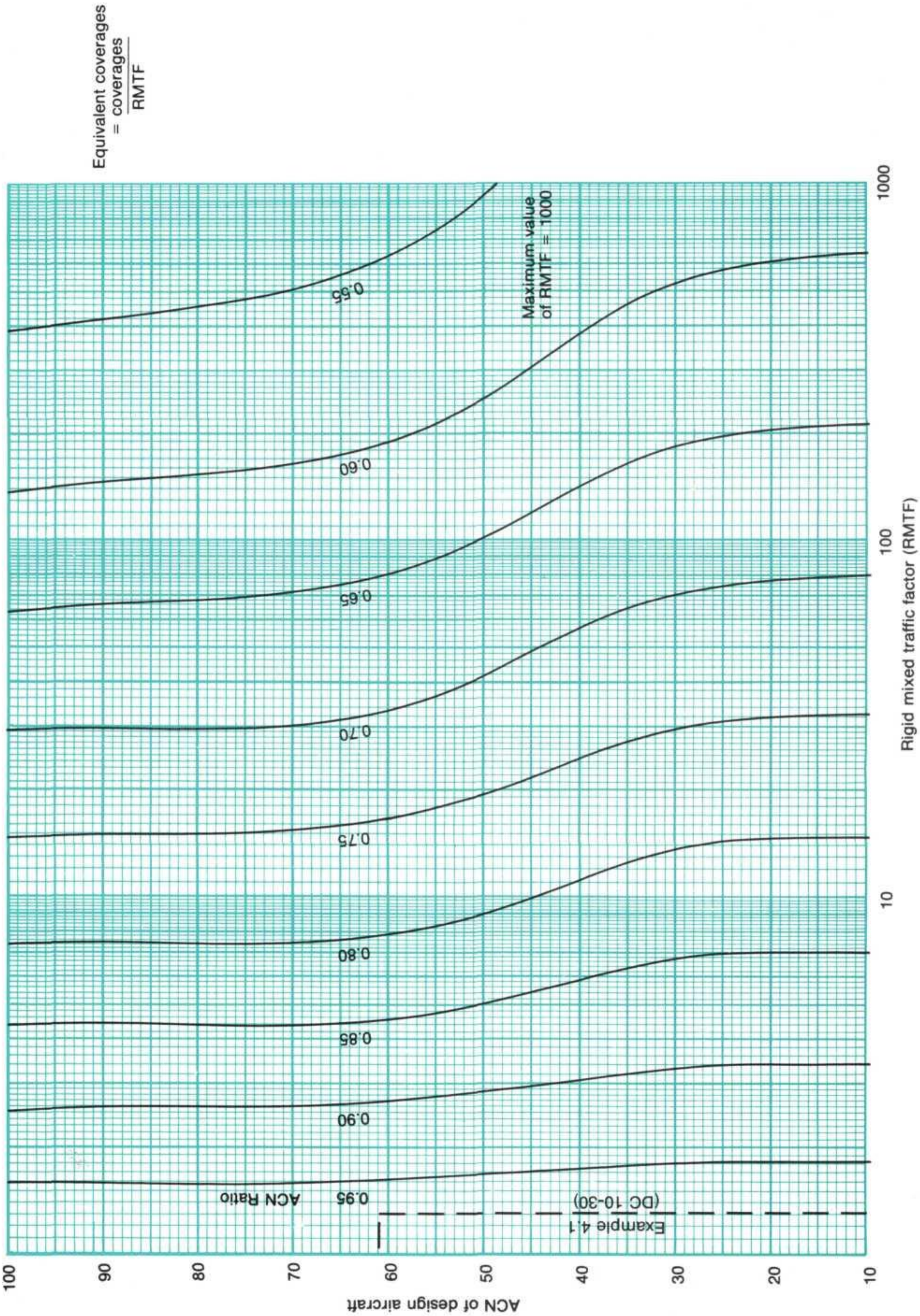


Figure 12 Mixed traffic analysis – rigid pavements

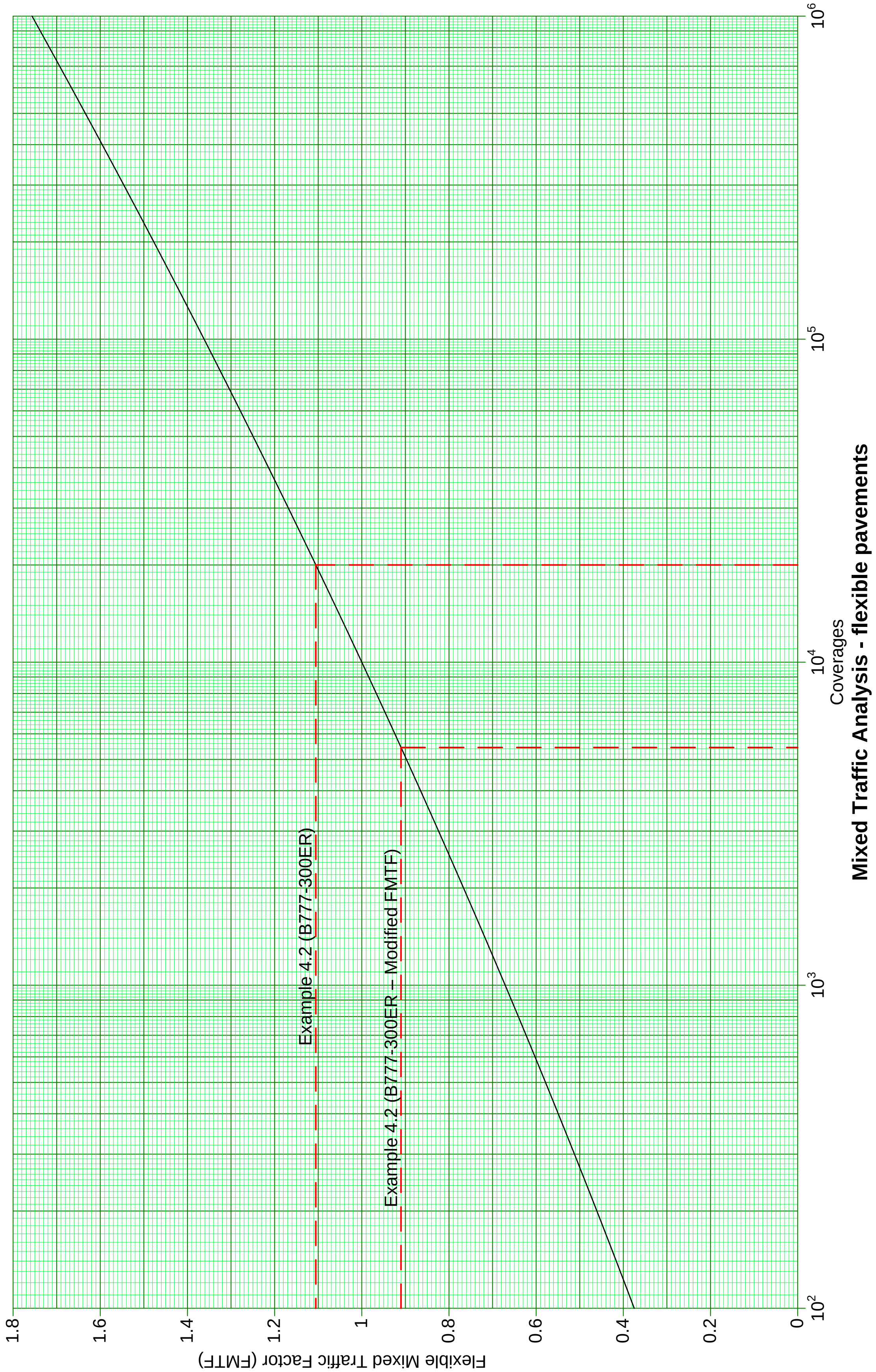


Figure 13 Mixed traffic analysis – flexible pavements

TRAFFIC ANALYSIS EXAMPLES

Example 4.1
Design a new rigid pavement for the main taxiway at an international airport used by a wide range of aircraft.

1. Design Life 30 years (see Section 4.7).
2. Expected Departures:

Aircraft	Departures/Year
A321-200	28600
A340-500	2000
A330-200	1200
B737-800	5000
B747-400	1000
B767-300	3800
B777-300ER	1600

3. Aircraft Data:

Aircraft type	All Up Mass (kg)	RIGID PAVEMENT SUBGRADES - MN//m²/m				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 150	Medium 80	Low 40	Ultra Low 20		
		ACN					
A321-200	89,400	56.5	59.4	62.1	64.3	Dual	2.1
A340-500	369,200	72.8	84.7	100	115.3	Dual Tandem	1.3
A330-200	233,900	53.7	62.4	74.3	86.9	Dual Tandem	1.3
B737-800	79,243	49.3	51.8	54.2	56.1	Dual	2.1
B747-400	397,800	52.4	62.7	74.4	85.1	Dual Tandem	1.3
B767-300	159,665	38.3	45.4	54.1	62.5	Dual Tandem	1.3
B777-300ER	352,441	65.8	85.3	109.3	131.5	Tridem	1

4. Soil Survey: $k = 50\ MN/m^2/m$.
5. Aircraft ACNs at the requisite subgrade value (k = 50) interpolated from Step 3:

Aircraft	ACN @ $k = 50\ MN/m^2/m$
A321-200	61.4
A340-500	96.2
A330-200	71.3
B737-800	53.6
B747-400	71.5
B767-300	51.9
B777-300ER	103.3

The Design Aircraft is the B777-300ER with ACN 103.3.

6. Design Aircraft: Boeing 777-300ER.

7. Total Coverages (see Table 8): 76,775.

Table 8 Rigid Mixed Traffic Analysis Example

1 Aircraft	2 ACN	3 ACN Ratio	4 Passes (Departures / Year x Design Life)	5 Pass-to-Coverage Ratio	6 Coverages (Col 4 / Col 5)	7 Rigid Mixed Traffic Factor (From Figure 12)	8 Equivalent Coverages (Col 6 / Col 7)
A321-200	61.4	0.59	858000	2.1	408571	148.39	2753
A340-500	96.2	0.93	60000	1.3	45802	1.89	24290
A330-200	71.3	0.69	36000	1.3	27481	33.17	828
B737-800	53.6	0.52	150000	2.1	71429	682.42	105
B747-400	71.5	0.69	30000	1.3	22901	32.45	706
B767-300	51.9	0.50	114000	1.3	87023	939.6	93
B777-300ER	103.3	1	48000	1	48000	1	48000
TOTAL COVERAGES							76775

8. Design for Medium Frequency Trafficking (i.e. 100,000 Coverages) by Boeing 777-300ER.

Example 4.2
Design a new flexible pavement for the runway at an international airport used by a wide range of aircraft.

1. Design Life 20 years,

2. Expected Departures:

Aircraft	Departures/Year
A321-200	28600
A340-500	1000
A330-200	2750
B737-800	5000
B747-400	3500
B767-300	3800
B777-300ER	1000

3. Aircraft Data:

Aircraft type	All Up Mass (kg)	FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 15	Medium 10	Low 6	Ultra Low 3		
		ACN					
A321-200	89,400	49.4	52	57.6	63.2	Dual	3.2
A340-500	369,200	75.3	82.2	97.8	129.8	Dual Tandem	1.8
A330-200	233,900	58.5	63.5	73.8	99.8	Dual Tandem	1.8
B737-800	79,243	42.9	45.4	50.4	55.3	Dual	3.2
B747-400	397,800	53	59	72.5	94.1	Dual Tandem	1.8
B767-300	159,665	39.5	43.3	51.1	69.9	Dual Tandem	1.8
B777-300ER	352,441	63.6	71.1	89.1	120.1	Tridem	1.4

4. Soil survey shows CBR 10%.

5. The actual subgrade value is one of the standard values. Therefore ACNs can be taken directly from Step 3 above. The Design Aircraft is the A340-500 with ACN 82.2.

6. Design Aircraft: Airbus A340-500.

7. Total Coverages (See Table 9): 33,442.

Table 9 Flexible Mixed Traffic Analysis Example

1	2	3	4	5	6	7	8	9
Aircraft	ACN	ACN Ratio	Passes (Departures / Year x Design Life)	Pass-to-Coverage Ratio	Coverages (Col 4 / Col 5)	Flexible Mixed Traffic Factor (from Figure 13)	Modified Mixed Traffic Factor (Col 3 x Col 7)	Equivalent Coverages (from Figure 13)
A321-200	52	0.63	572000	3.2	178750	1.46	0.92	5933
A340-500	82.2	1	20000	1.8	11111	1	0	11111
A330-200	63.5	0.77	55000	1.8	30556	1.17	0.9	5261
B737-800	45.4	0.55	100000	3.2	31250	1.17	0.65	849
B747-400	59	0.72	70000	1.8	38889	1.21	0.87	4082
B767-300	43.3	0.53	76000	1.8	42222	1.22	0.64	818
B777-300ER	71.1	0.86	20000	1.4	13889	1.05	0.91	5368
TOTAL COVERAGES								33442

8. Design for Medium Frequency Trafficking (i.e. 100,000 Coverages) by Airbus A340-500.

5 Rigid Pavement Design

5.1 GENERAL

5.1.1. For over 50 years Defence Estates' preferred choice of new rigid construction has comprised unreinforced pavement quality concrete (PQC) without dowels, tie bars or keys, on a rolled drylean concrete (DLC) base. Defence Estates does not consider necessary the use of traditional mechanical load transfer devices; experience has shown that with good base support provided by drylean concrete together with aggregate interlock at the transverse (contraction) joints of the PQC and the omission of regular expansion joints (see para 5.3.1), an adequate level of load transfer is maintained for a considerable number of load repetitions. Furthermore standard unreinforced, undowelled rigid pavement design simplifies construction and gives reliable performance.

5.1.2. Longitudinal joints are simple butt joints without load transfer. In general the absence of load transfer has not caused problems. However, where aircraft regularly traffic across longitudinal joints, e.g. on some aircraft stands where the concrete is laid normal to the stand centre-line, early failures have occurred. In these situations the load transfer should be provided at the longitudinal joint, by dowels or a profiled joint, or the concrete slab thickness should be increased as described in 5.6.3.

5.1.3. Apart from the preferred choice the following types of rigid construction are also considered in this Chapter:

(i) Fully dowelled (unreinforced) – see Section 5.7
(ii) Jointed reinforced (with dowelled expansion, construction and contraction joints) – see Section 5.8.
(iii) Continuously reinforced concrete – see Section 5.9.

5.1.4. In climates with a high seasonal temperature variation, omitting the dowels and regular expansion joints must be considered with caution. Pavements constructed without expansion joints at low temperatures in these climates may be subject to 'blow-ups'. Conversely, those constructed without dowels at the high end of the temperature range would have poor load transfer properties at open transverse joints in winter. Figure 14 gives three zones of annual temperature variation, moderate, high and extreme. The preferred constructions with undowelled PQC slabs without regular expansion joints (see para 5.3.1) on drylean concrete bases apply without restrictions in the zone of moderate temperature variation. For the zone of high annual temperature variation PQC slabs constructed within the centre 80% band of annual temperature do not need dowels or regular expansion joints. However, PQC slabs constructed outside this temperature range and those constructed in the zones of extreme temperature variation, should be fully dowelled; when constructed at low temperatures, regular expansion joints should also be provided.

5.1.5. Also to be considered are the excessive temperature differentials which can develop between the top and bottom of a slab, causing high warping stresses. Figure 15 shows the regions where temperature warping stresses are likely to be significantly greater than allowed for in the rigid pavement design model (see Appendix F). In these regions the PQC thickness requirements obtained from the design charts should be increased by 10% to allow for excessive warping stresses.

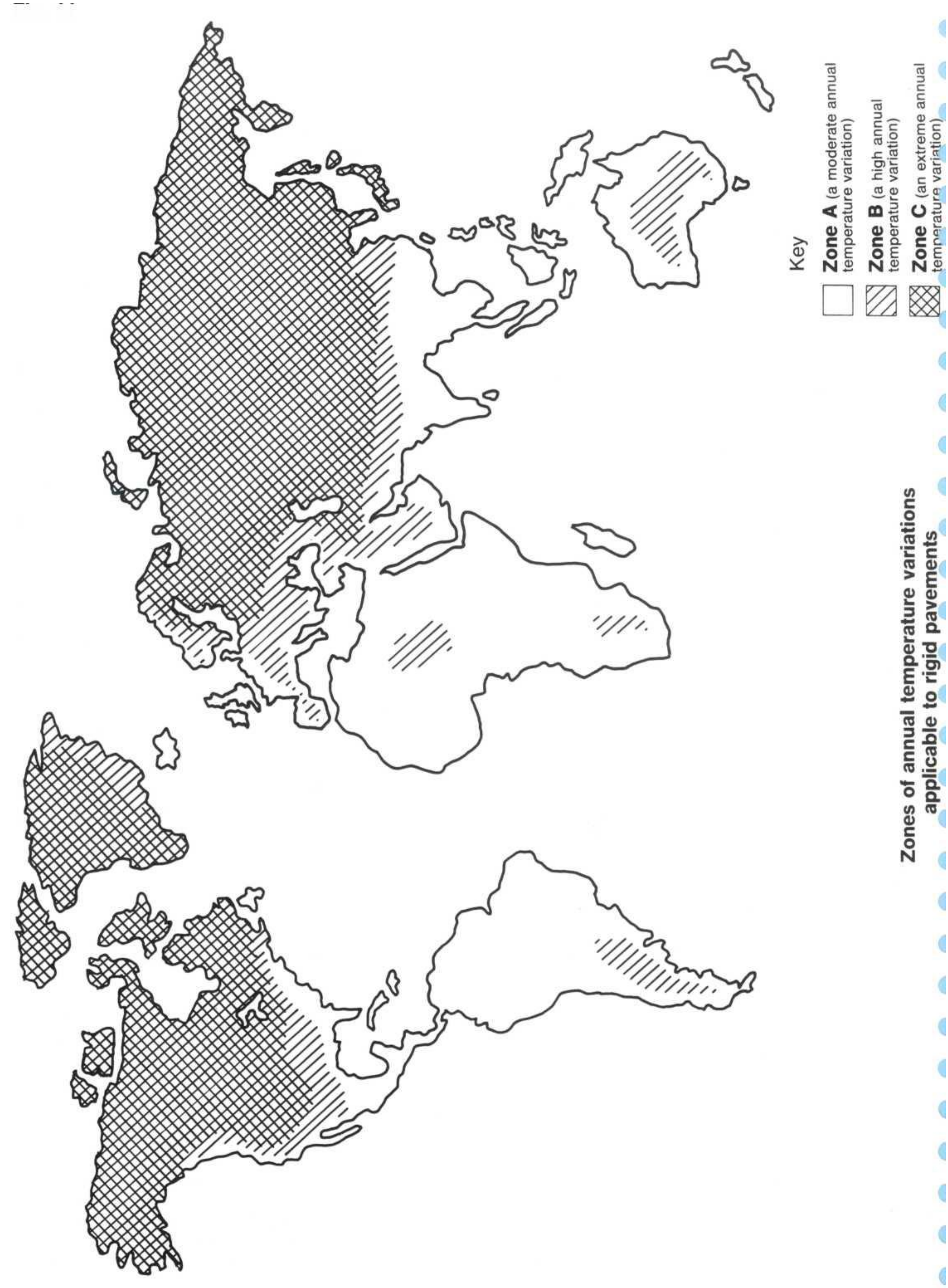


Figure 14 Zones of annual temperature variations applicable to rigid pavements

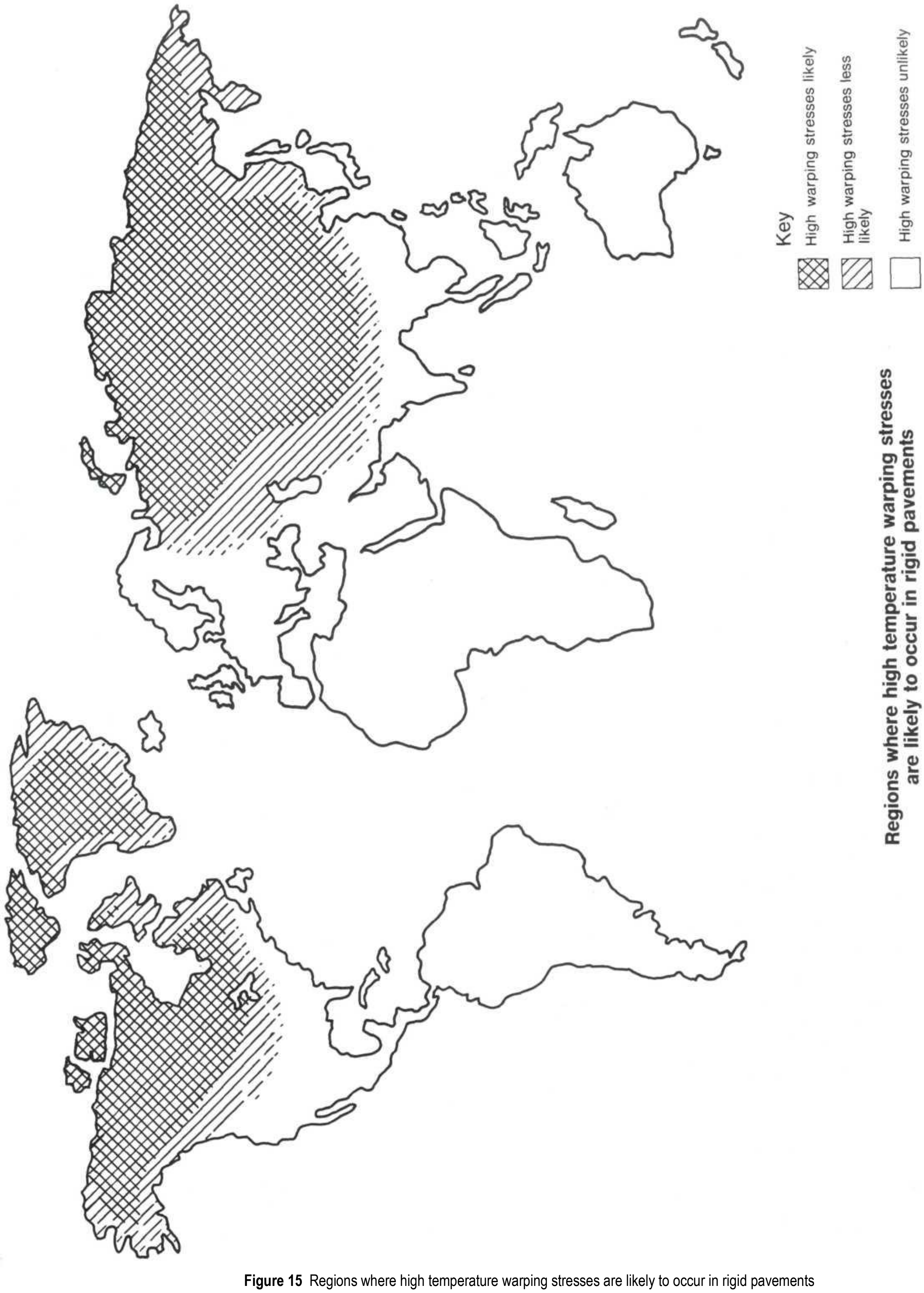


Figure 15 Regions where high temperature warping stresses are likely to occur in rigid pavements

5.2 PAVEMENT QUALITY CONCRETE SLAB

5.2.1. Pavement Quality Concrete (PQC) must be strong enough to provide an economical pavement thickness. The PQC must also provide a durable, hard wearing, weather resistant surface so that the expense and disruption of major maintenance is seldom required. Air entrainment should be used to provide resistance to frost and the action of de-icing chemicals.

5.2.2. The design flexural strength referred to on Charts 1, 2, 3 and 4 is the in situ mean flexural strength of the PQC at 28 days (28 days is assumed to be the minimum time before the pavement is brought into use). The in situ mean flexural strength parameter relates directly to the failure criteria assumed in the design method i.e. 50% of the bays in the trafficked area have developed cracks; as halving cracks initially develop at the underside of the slabs this does not necessarily mean that 50% of bays will be exhibiting surface cracks at failure (see Appendix F). The design flexural strength relates to the concrete in the pavement. Quality control during construction should be based on strength tests on samples with the same degree of compaction and cured in the same regime as the in situ concrete e.g. by using cores from the slab, provided the flexural to compressive strength ratio is known.

5.2.3. Figure 16 provides guidance on relationships between 28 day in situ and laboratory mean flexural strengths, 7 day laboratory mean and characteristic compressive strengths from cubes and 28 day in situ characteristic compressive strengths from cores. The characteristic strength is based on a 5% defective rate. The relationship between 7 day cube strengths and 28 day core strengths is based on the Defence Estate's experience, modified for the method of determining compressive strength described in BS EN 13877-2:2004. It takes into account the differences in both the curing regime and degree of compaction for cast cubes and cores extracted from the pavement. The estimated laboratory mean flexural strength is based on a 20% gain in strength from 7 to 28 days. If evidence suggests that the actual gain in strength is different, the strengths on the axis should be adjusted by the ratio of the actual gain in strength to the assumed 20%.

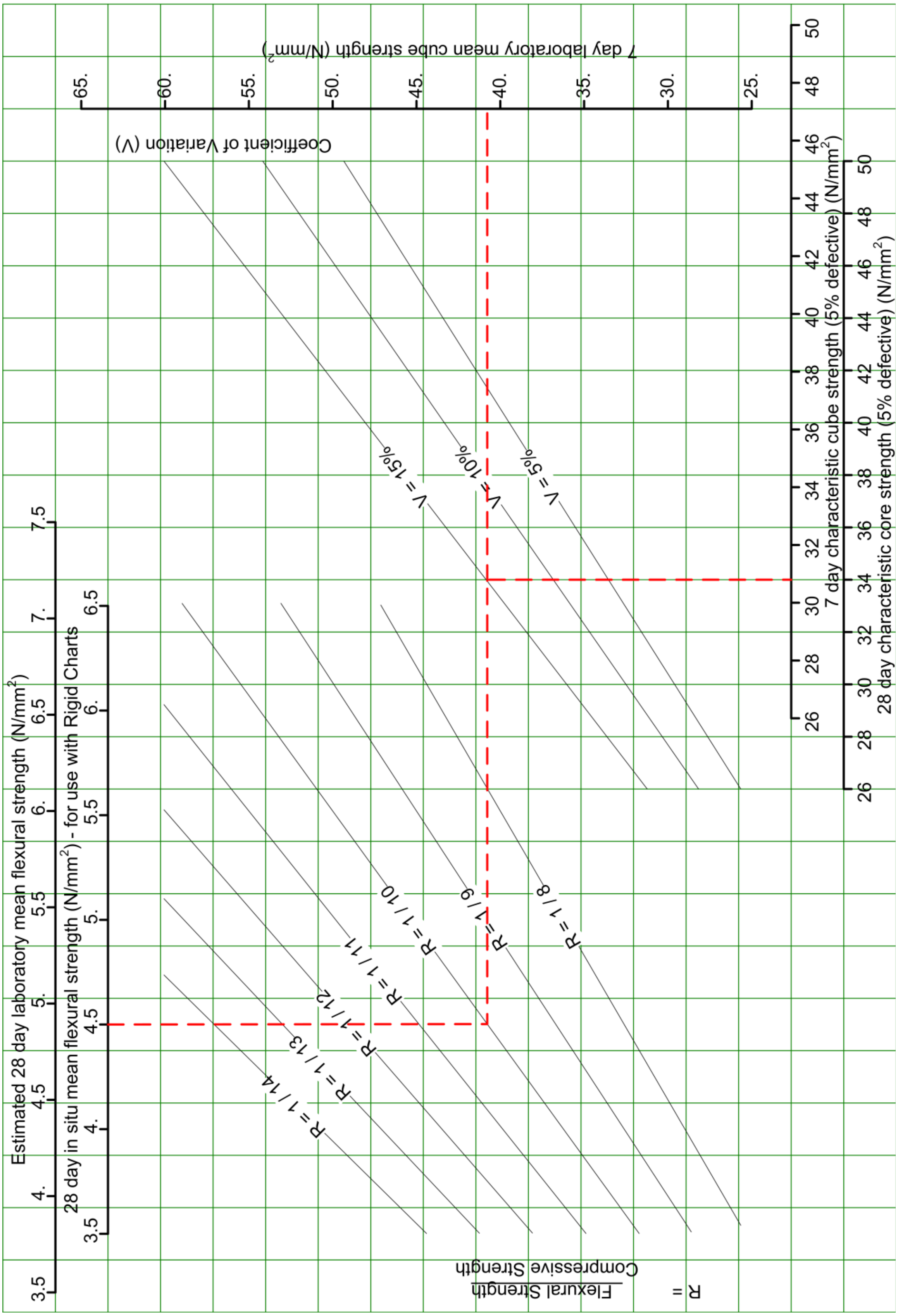


Figure 16 Concrete flexural strengths

5.3 JOINTS IN PAVEMENT QUALITY CONCRETE SLABS

5.3.1. Recommendations on the frequency of expansion joints and the maximum spacing of contraction and construction joints are given in Table 10 for unreinforced PQC. Joint spacings are fundamental to the pavement design so these recommendations should not be exceeded, except as modified in para 5.3.2.

5.3.2. Expansion joints. For PQC slabs 250mm or more in thickness regular expansion joints are not usually required. However, for some situations it is advisable to provide them to limit movement of the pavement, at maximum intervals such that expansion will not cause unacceptable extrusion of the sealant from the joint. For pavements constructed at low temperatures in climates with high or extreme annual variations in temperature as defined by Figure 14 it is wise to provide regular expansion joints for all PQC slab thicknesses. The spacing of expansion joints in these circumstances for slabs 250mm or more in thickness should be similar to that required for 225mm thick slabs (Table 10). The actual spacing will depend on the type of coarse aggregate, the annual range of temperature and the movement accommodation factor of the joint filler and the sealing compounds. For slabs less than 250mm thick the spacing of expansion joints depends on the slab thickness and the type of coarse aggregate used in the PQC. For all PQC thickness expansion joints should be formed between new and existing pavements, at junctions, at tangent points of bends, around box gutters and around other obstructions to the continuity of the slabs. Figure 17 and Figure 18 show details of dowelled and undowelled expansion joints. When constructing undowelled unreinforced PQC, Defence Estates does not normally include any specific design provisions (i.e. dowels or thickened slab edges) for expansion joints. This practice has not led to premature maintenance problems primarily because of the stiff DLC base and the dearth of expansion joints. If dowels are to be provided at expansion joints in an otherwise undowelled pavement, care should be taken in the joint detailing to ensure that the overall movement of the slabs as a result of moisture and temperature changes is not locally impeded in the direction transverse to the dowelled joints.

5.3.3. Contraction grooves. Contraction grooves initially control the development of cracks caused by drying shrinkage or a drop in temperature shortly after laying. Initial cracking due to these factors rarely occurs at every contraction joint. In the long-term the spacing of these grooves together with the construction joints is fundamental to the pavement design as it controls the warping stresses caused by temperature differences between the top and bottom of the slab. An undowelled contraction groove, which is standard Defence Estates practice, relies on aggregate interlock to provide load transfer. Load transfer from aggregate interlock decreases as the crack width increases. For thin slabs with regular expansion joints the effect of joint opening is built into the design, but for thick slabs the assumption is that there are no expansion joints. In situations where there induced cracks in slabs greater then 250 mm thick may open more than expected, e.g. where it is necessary to provide expansion joints, consideration should be given to dowelling the contraction groove. Figure 19, Figure 20 and Figure 21 show typical details for unsealed contraction grooves, sealed contraction grooves, and dowelled and sealed contraction grooves. The grooves can be sawn or wet formed, but Defence Estates prefers the former in conjunction with the use of limestone coarse aggregate which can be easily sawn. This is because wet forming grooves can cause the adjacent concrete to become overworked leading to poor durability and long-term maintenance problems. In arid regions where wind blown sand and dust are prevalent, the engineer may prefer to provide wider contraction grooves which can be sealed to prevent abrasion and 'jamming' of joints (see Figure 20 and Figure 21). For reinforced concrete pavements the spacing of joints can be increased as explained in Section 5.8.

5.3.4 Day work joints. Day work joints are usually simple butt joints. In most circumstances they are infrequent and have little effect on the failure rate of the pavement. In some situations frequent daywork joints become necessary, e.g. laying in winter with short days. In these situations consideration should be given to dowelling the day work joints.

5.3.4. Construction joints. The spacing of these joints in unreinforced PQC should be the same as the contraction groove spacing. This is because experience has shown that the effects of wheel load and warping stresses are much reduced in square bays. Figure 22 shows the standard construction joint detail used by Defence Estates. Figure 23 details a sealed joint which may be considered more appropriate in certain circumstances for the same reason as described in para 5.3.3. Where load transfer is required at construction joints (para 5.1.2) it should be provided by dowels (para. 5.3.5) or a profiled joint (para 5.3.6). Both dowels and profiled joints give greater quality problems than butt joints.

5.3.5. Dowelled Construction joints. The use of dowels, including the diameter and spacing and the potential problems associated with them, is described in Section 5.7. Figure 24 details a dowelled and sealed construction joint.

5.3.6. Profiled Construction joints. An alternative to dowelling construction joints to provide load transfer is a keyed joint. In addition to traditional keys various curved profiles for the faces of construction joints have also been tried, not necessarily for load transfer. The dimensions of keys have been based on the slab thickness rather than a design for the specific loading. Historically keyed joints have not performed well, often suffering premature failure. due to factors such inadequate shear capacity in the key and very high stresses at sharp angles. In addition the geometry of some key designs means that if the joint opens due to shrinkage or movement caused by temperature changes, the faces do not come into contact when loaded and the load transfer is lost.

5.3.7. Figure 25 shows a profiled joint specifically designed to provide load transfer to meet the design requirement while avoiding high stresses at angles and being practicable to construct with a shaped form. A profiled joint may be considered when:

(i) The slab thickness is greater than 250 mm.
(ii) It can be demonstrated that the capacity of the joint, based on its dimensions and the concrete strength, is adequate for the proposed loading.
(iii) The detail can be offered as an option to dowels so that the contractor can confirm that fixed forms and side-forms for slip-form pavers can be formed to the correct dimensions, that the concrete mix allows formation of the profile and will have a standard deviation less than or equal to the design assumption, and that any increase in slab thickness is more economic than dowelling the standard design thickness.

5.3.8. To design a profiled joint:

(i) Design the pavement in accordance with Section 5.6.
(ii) Check the capacity of a profiled joint for the Design Aircraft, using the equation below.
(iii) If an increase the slab thickness is considered a viable option increase the thickness until the Joint Capacity Factor is 1.
(iv) Check the Joint Capacity Factor for any aircraft in the design mix that may have an individual wheel load greater than that of the design aircraft. (NB. Unlike overload of a concrete slab which is unlikely to result in failure under a single load, overload of a profiled joint may result in an immediate failure in shear. If significant overload is foreseeable the Joint Capacity Factor should be checked for likely aircraft.)
(v) Detail the joint in accordance with Figure 25.

The capacity of a profiled joint within a given slab thickness is given by:

(i) $$JCF = \frac{11.199P}{h\sqrt{f_{cm}(1-2.33CV)}\sqrt{\frac{P}{p}}}$$

where:
JCF = Joint Capacity Factor (must be ≥ 1).
h = slab depth (mm)
P = wheel load (kg)
p = tyre pressure (MPa)
$\mathbf{f_{cm}}$ = mean 28 day in situ compressive strength of concrete (N/mm^2)
CV = Coefficient of Variation of concrete (%)

The profiled joint is adequate for the load if the Joint Capacity Factor is greater than or equal to one.

5.3.9. Profiled joints can be formed by manufacturing shaped side forms for fixed-form or slip-form paving. It is vital that the profiled joint is formed without sharp angles or steps. For instance forming the joint by welding a shaped plate to an existing steel shutter, leaving a step along the top of the profile, is guaranteed to cause deep spalling along the joint. When slip-form paving the concrete miz, and the mixing, delivery and laying processes, are critical to accurately forming the profile.

5.3.10. Joint Rotation. When construction joints are loaded by wheels trafficking directly along joint the deflection causes the bay edge to rotate towards the adjacent bay. Contact between the two faces can cause deep spalling. Providing load transfer does not mitigate the problem as transferring the load to the adjacent bay results in the same total rotation. Figure 25 shows a solution to the problem, eliminating the contact between the faces. This solution may be applied to any of the other construction joint details. It may also be applied to daywork joints where the same problem can occur.

Table 10 Maximum Joint Spacing for Unreinforced PQC

Type of joint or groove	Coarse aggregate	Nominal thickness of slab (mm)				
		150	200	225	250	275 or over
Expansion	Limestone	36 m	48 m	54.0 m	None	None
	Other rocks and gravels	18 m	22.5 m	31.5 m	None	None
Contraction or Construction	Limestone	4 m	4 m	6.75 m	6.75 m	7.5 m
	Other rocks and gravels	3 m	3 m	5.25 m	5.26 m	6 m

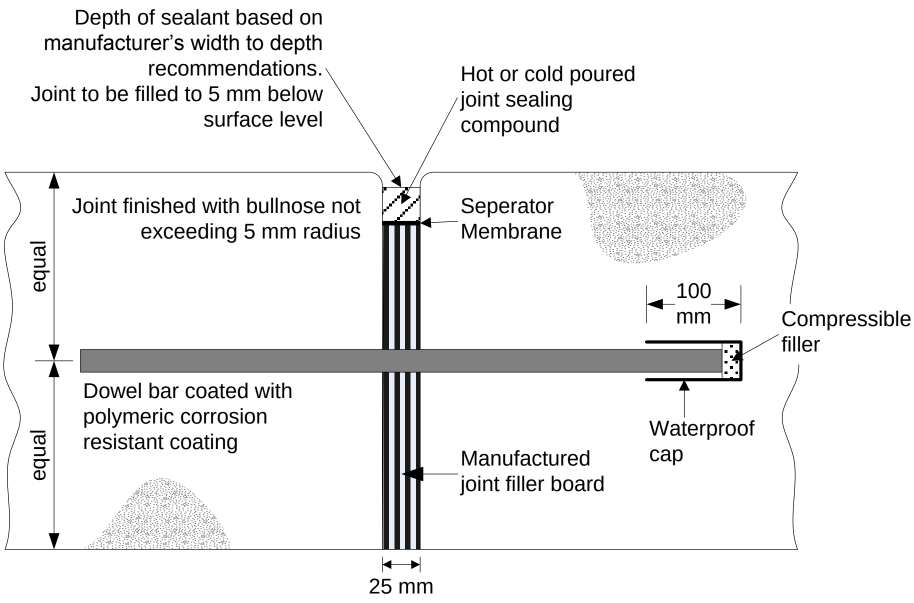


Figure 17 Dowelled expansion joint

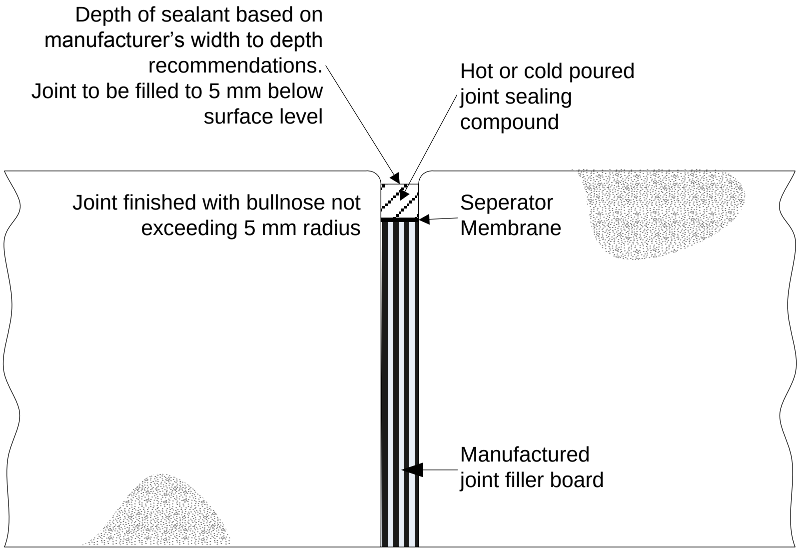


Figure 18 Undowelled expansion joint with hot or cold poured sealant

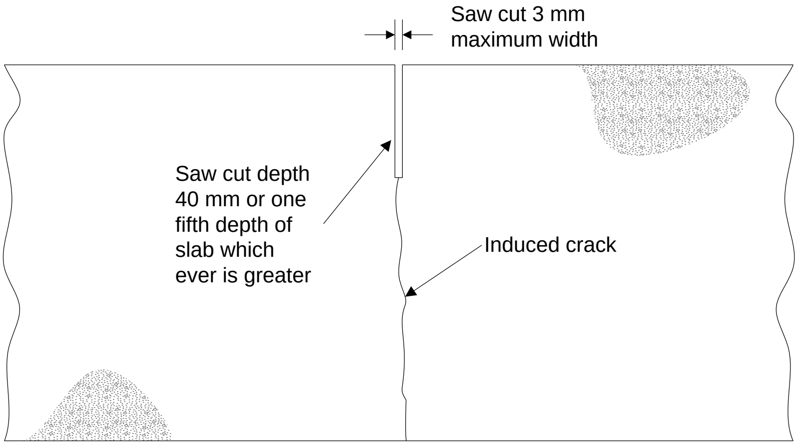


Figure 19 Sawn contraction groove (not to be used for flint gravel aggregates)

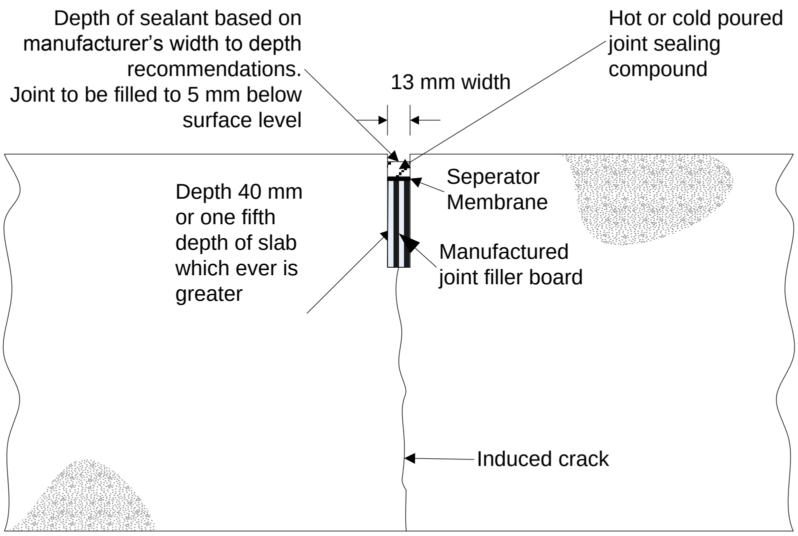


Figure 20 Formed contraction groove

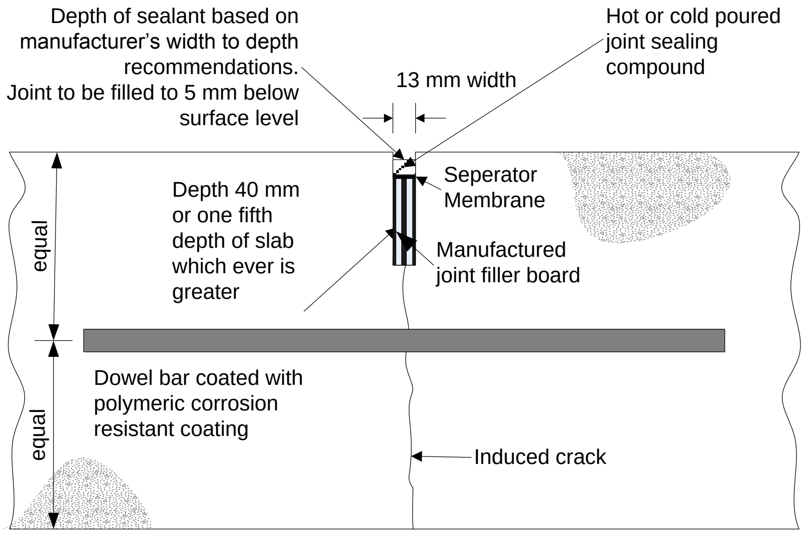


Figure 21 Dowelled contraction joint with formed groove

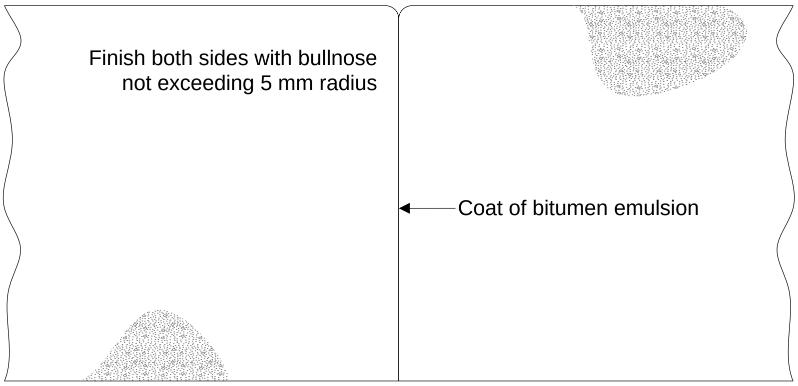


Figure 22 Undowelled construction joint

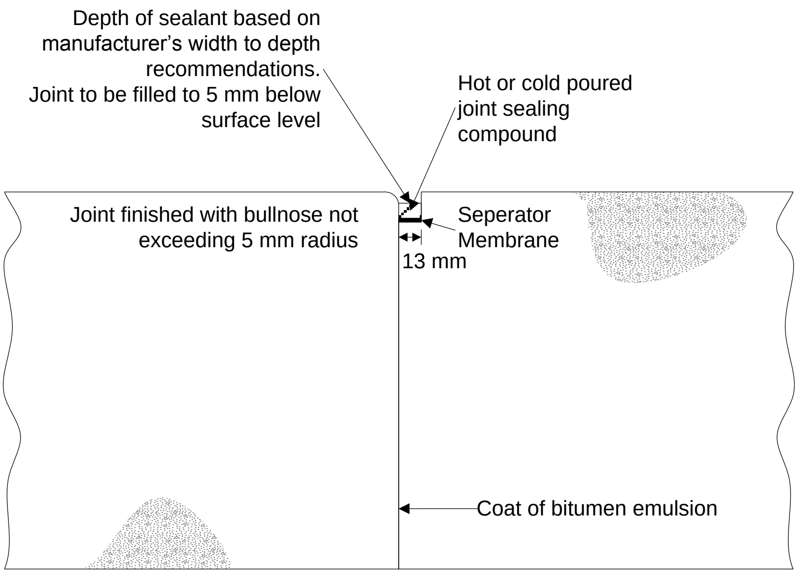


Figure 23 Undowelled sealed construction joint

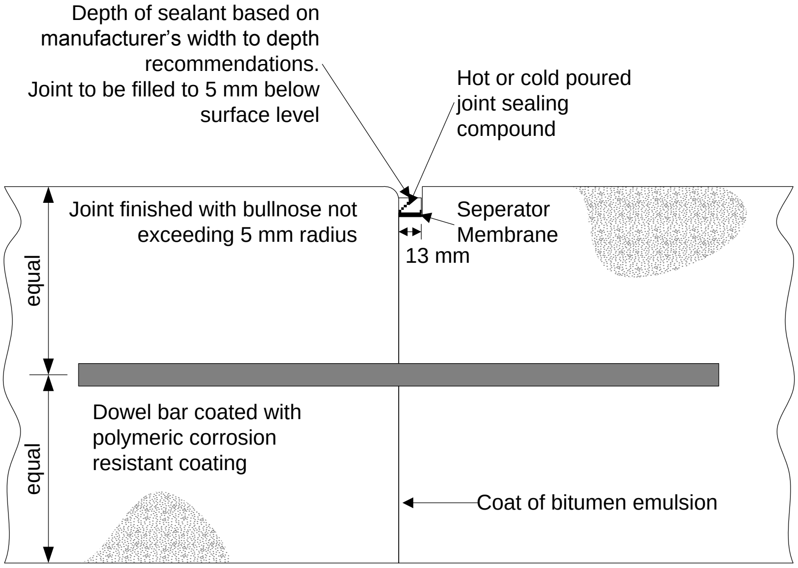


Figure 24 Dowelled sealed construction joint

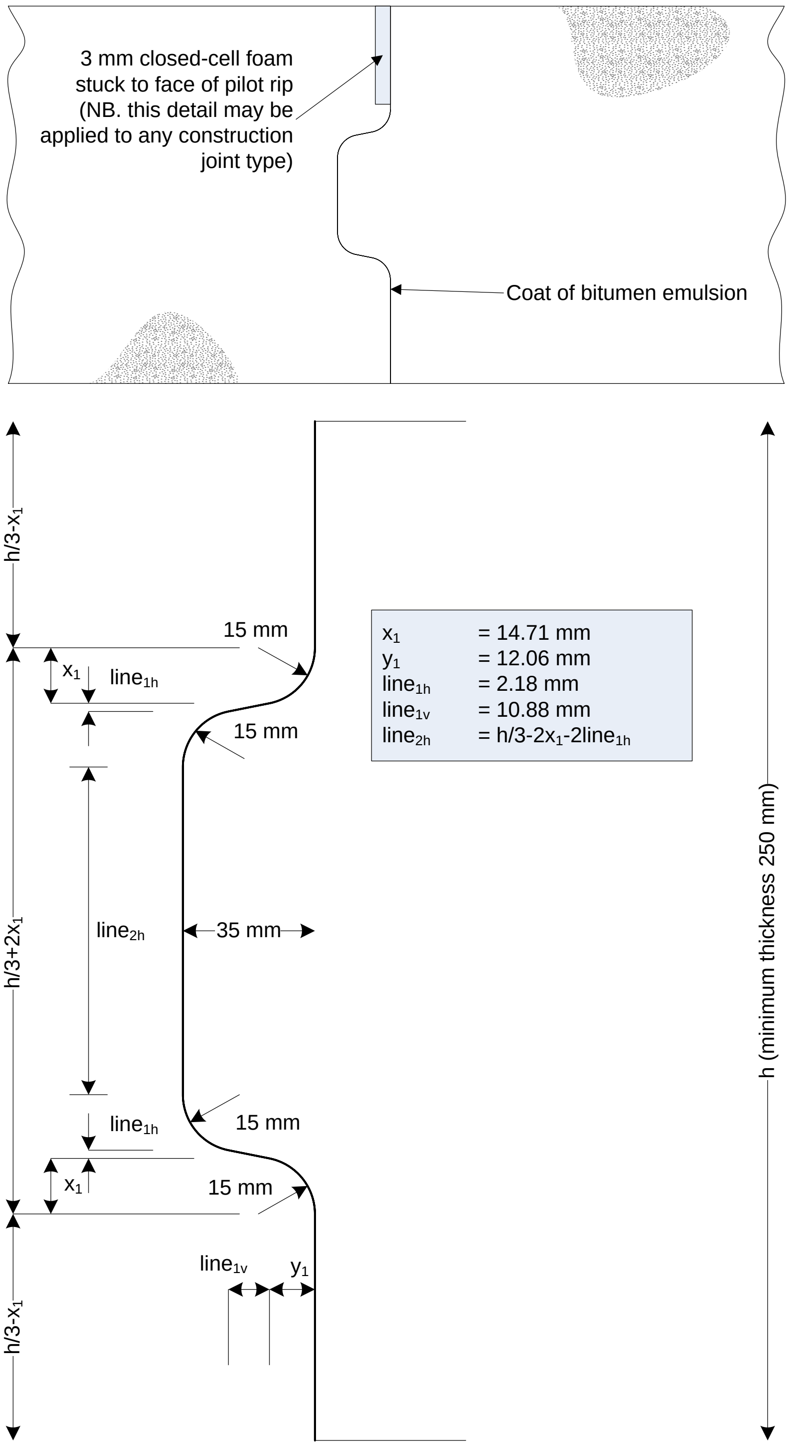


Figure 25 Profiled construction joint, with former to mitigate joint rotation

5.4 BASE

5.4.1. The standard base material used by Defence Estates is drylean concrete (see Appendix C).

5.4.2. The purpose of the DLC base is:

(i) to provide a uniform and substantially improved support to the PQC, particularly at slab joints,

(ii) to reduce the deflection at slab joints, caused by wheel loading, and thereby help to preserve aggregate interlock at transverse joints so that a high level of load transfer is maintained,

(iii) to act as a protective layer to moisture sensitive soils while PQC works is in hand and also to form a level and firm working course on which to lay the PQC,

(iv) to prevent mud pumping,

(v) to reduce the rate of deterioration if cracking of the PQC occurs,

(vi) in the case of PQC pavements for high ACN values on poor subgrades, to reduce the required PQC thickness.

5.4.3. Chart 3 gives an increased thickness of DLC for pavements on poor subgrades subjected to heavy multiple wheel loads. This is to allow for the additional wheel load interaction at depth and to prevent over-stressing or poor subgrades.

5.4.4. The DLC thickness shown on Charts 1-3 is the minimum thickness required under the PQC slab. On poor subgrades it may not be possible to achieve adequate compaction and the necessary finished level tolerances if this thickness is laid on one layer directly on the natural formation. In this case the DLC should be place in two layers, the first one forming a working course on which the second can be compacted. The working course should be laid by hand with only very light compaction. As a guide, the working course thickness should be 100mm for k = 20-30MN/m^2/m and 75mm for k = 30-40 MN/m^2/m. As the top layer should not be less than 75 mm thick the minimum total thickness of DLC which can be practically laid directly on a poor natural formation will be 175mm for k = 20-30 MN/m^2/m and 150mm for k = 30-40 MN/m^2/m. Alternatively the subgrade may be improved by using a granular sub-base (see para 3.8.4).

5.4.5. The use of a cement-stabilised material may be considered instead of drylean concrete. Figure 26 gives equivalency factors for cement-stabilised material in relation to drylean concrete which depends on the 7-day characteristic cube strength (5% defective). Note that Figure 26 gives reduced equivalency factors for cement-stabilised fine-grained materials. This is because of the greater reduction in stability that occurs in fine-grained materials after cracks eventually propagate in the cement-bound layer; there being substantially less aggregate interlock. Cement-stabilised material may be produced by plant mixing or in situ stabilisation as long as the required strengths are met. In other respects, such as surface tolerances and densities, the specification for cement-stabilised material should be the same as that for drylean concrete.

5.4.6. Rigid pavement designs for PQC slabs on granular sub-bases, or for PQC slabs laid directly on the subgrade are not included in this Chapter. Chapter 7 includes a procedure using Chart 5 for evaluating these types of construction. This procedure can also be used for assessing strengthening requirements (see Chapter 7).

5.5 SUBGRADE

5.5.1. For details of subgrade characteristics, the test method for determining k, subsoil drainage and sub-grade compaction requirements, see Chapter 3.

5.5.2. In assessing k, the presence of work underlying layers in the soil must be carefully considered. Heavy multiple wheel loads induce large deflection basins in a rigid pavement giving rise to significant stress levels in the subsoil at depth. Therefore pavements for aircraft with heavy multiple wheel gears should be designed for a conservative k which reflects the strength of the underlying weak soils.

5.5.3. Figure 10 sets out a method of assessing subgrade improvement provided by a granular sub-base (see para 3.8.4).

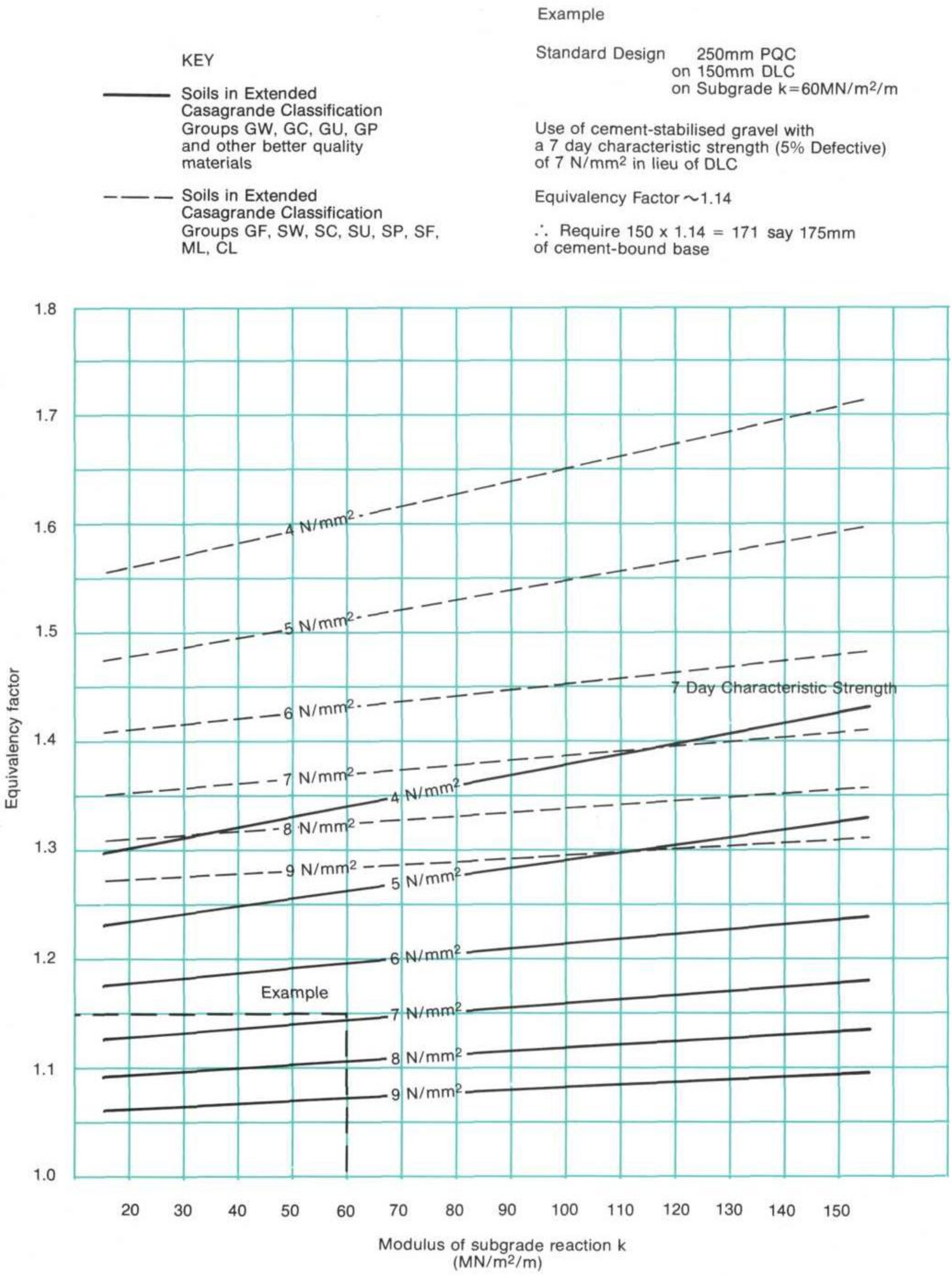


Figure 26 Cement-bound sub-bases for rigid construction

5.6 DESIGN OF UNDOWELLED AND UNREINFORCED CONCRETE PAVEMENTS

5.6.1. Separate design charts have been prepared for single, dual dual-tandem and tridem main wheel gears, Charts 1, 2, 3 and 4 respectively; see Appendix D for the definition of these gear types. The use of the charts requires four design parameters:

(i) Flexural strength of the concrete (see section 5.2).

(ii) The Modulus of Subgrade Reaction k. See Section 5.5 and Chapter 3 for details of subgrade characteristics. If subgrade improvement is to be carried out as detailed in Section 3.8 the increased k value will be appropriate for design.

(iii) The design ACN (see Section 2.6).

(iv) The frequency of trafficking – either Low, Medium or High. Chapter 4 defines these traffic levels in terms of Coverages by the Design Aircraft. For calculating the number of Coverages for different areas of pavement and equating the loading effects of different aircraft see Chapter 4.

5.6.2. Having established the above parameters, Charts 1-3 are used as follows:

(i) Select the frequency of trafficking (i.e. Low, Medium, High); for High Frequency Trafficking see Section 5.10.

(ii) Make a horizontal projection until it intersects with the appropriate design flexural strength.

(iii) Make a vertical projection from the intersection point to the design ACN.

(iv) From this intersection point make a horizontal projection to the k = 20 line. Trace a line parallel to the curves until it intersects the vertical projection of the appropriate k. At this stage read off the DLC base thickness required.

(v) From the last intersection point make a horizontal projection to the right hand ordinate. Read off the PQC thickness and round it to the nearest practical construction increment (Defence Estates works in 25mm increments). The minimum PQC slab thickness is 150mm. This is because thinner slabs constructed to the minimum practical by size in the Specification would crack prematurely due to warping effects.

See Examples 5.1 and 5.2.

5.6.3. Where aircraft regularly traffic across the longitudinal joints and load transfer is not provided the slab should be thickened (see paragraph 5.1.2). To provide the equivalent of good load transfer the thickness of the slab should be increased by 25%.

See Examples 5.1 and 5.2.

5.7 FULLY DOWELLED CONCRETE PAVEMENTS

5.7.1. Generally Defence Estates does not specify fully dowelled concrete pavements (i.e. with all joints dowelled). If dowelled concrete designs are being considered the following points should be borne in mind.

(i) Dowels induce high localised stresses in the concrete. This can lead to crushing and/or cracking of the concrete around the dowel, particularly if the concrete has not been properly compacted in this area.

(ii) Long-term effectiveness of dowels depends largely on their accurate alignment which reduces their tendency to seize up. If the movement at the joint is impeded this can lead to ‘blow-ups’ in hot weather or the development of cracks parallel to the joints in unreinforced concrete in cold weather. In addition excessive differential shrinkage between newly constructed adjacent lanes of concrete can cause the dowels across the construction joints to become jammed, so that any subsequent contraction of the bays would induce tensile stresses in the concrete.

(iii) Experience has shown that the load transfer effectiveness of dowels lessens with load repetition so in many cases it is considered that to incorporate dowels does not reduce the thickness given by the standard undowelled/unreinforced designs (see para 5.7.2). In climates with high annual variations in temperature it may be necessary to provide a dowelled PQC slab to maintain load transfer effectiveness (see para 5.1.4).

5.7.2. With the following exception, the design procedure for fully dowelled concrete pavements is the same as that set out in Section 5.6 for undowelled/unreinforced concrete using Charts 1,2 and 3. Table 11 gives reductions in PQC slab thickness (determined from Chart 1, 2 and 3) for dowelled concrete pavements less than 250mm thick.

5.7.3. For a fully dowelled concrete pavement dowels should be provided at all construction, contraction and expansion joints to the requirements set out in Table 12. They should be installed at mid-depth of the slab. See Section 5.3 for joint layout requirements and details.

5.8 JOINTED REINFORCED CONCRETE PAVEMENTS WITH DOWELS

5.8.1. Generally Defence Estates does not specify this type of construction which provides little or no gain in structural performance and is more likely to present long term maintenance problems than the standard undowelled unreinforced rigid pavement designs.

5.8.2. Jointed reinforced concrete pavement is usually constructed in long bays giving fewer transverse joints. The long bays will tend to develop one or more transverse cracks due to shrinkage and differential temperature stresses. The function of the reinforcement is to hold the cracks tight and minimise deterioration. Failure of jointed reinforced concrete pavement is generally by spalling of the transverse cracks. Monitoring and maintenance of the cracks can be problematic, including increased disruption to aircraft operations. The risk of premature failure is greater in this type of pavement than in jointed unreinforced concrete constructed in square bays.

Table 11 Design Thicknesses for Dowelled Constructions

PQC slab thickness requirement from Charts 1, 2 and 3 (mm)	Allowable reduction in PQC slab thickness for dowelled construction (mm)
equal to or greater than 250	0
225	15
200	25
175	25

Table 12 Dowel Size Requirements

PQC Slab Thickness (mm)	Dowel Diameter (mm)	Total Dowel Length (mm)	Spacing (mm)
150	20	400	300
175-200	25	450	300
225-275	30/32	450	300
300-400	40	500	375
425-450	50	600	450

5.8.3. Incorporating light reinforcement into a concrete slab to control shrinkage and warping cracks does allow a considerable increase in the spacing of transverse contraction joints. The quantity of steel required varies from 0.05% to 0.3% of the cross sectional area of the slab and should be placed in the upper part of the slab with at least 50mm cover. This does not improve the flexural strength of the slab and therefore the design thickness requirements are the same as those for dowelled PQC slabs (see para 5.7.2).

5.8.4. The areas of reinforcement required in both the longitudinal and transverse directions should be calculated from the following formula.

$$A_s = \frac{LC_f}{2F_s}\frac{Wh}{}$$

Where A_s = area of steel in mm^2/m width of slab

L = distance between contraction joints (longitudinal direction) or construction joints (transverse direction) in metres. The spacing of contraction joints should not exceed 23 metres.

W = weight of concrete in kN/m^3

h = slab thickness in mm

F_s = working stress in reinforcement in N/mm^2 (F_s = 0.75 yield stress)

C_f = coefficient of subgrade resistance to slab movement. This is dependent on the base material and the slab dimension. For construction with a DLC base and a polythene separation layer a value of 1.5 can be taken.

'A_s' should not be less than 0.05% of the cross sectional area of the concrete. Reinforcement is usually in the form of mesh. Longitudinal laps should be at least 30 times the diameter of the wire. Transverse laps should be not less than 150 mm or 20 times the transverse wire diameter, whichever is the greater.

5.8.5. Dowels should be provided at construction, contraction and expansion joints in accordance with the requirements set out in Table 11. Figure 27 shows a typical longitudinal section through a jointed reinforced concrete pavement. See Section 5.3 for joint details.

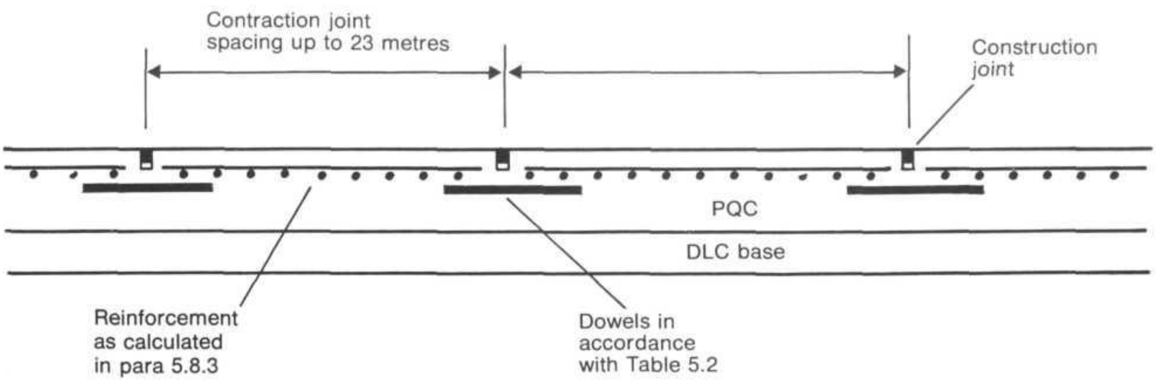


Figure 27 Typical longitudinal section through jointed reinforced concrete pavement

5.9 CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

5.9.1. In Defence Estates' experience the use of continuously reinforced slabs has not resulted in more cost-effective pavements. Continuously reinforced slabs of reduced thickness generally suffer early spalling at shrinkage/warping cracks. The spalling is caused by a combination of frost damage, excessive working of cracks from repetitive wheel loading, jet blast and high tyre pressure.

5.9.2. Defence Estates does not have an established procedure for this type of construction. However, where it is being considered the following points need to be remembered:

(i) To achieve a consistent and controlled development of cracks the amount of longitudinal reinforcement required is likely to be between 0.5-0.7% of the cross-sectional area of the concrete.

(ii) To avoid excessive deflections and consequent working of cracks due to trafficking by heavy wheel loads, only a modest saving in PQC thickness should be considered. A reduction in excess of 15% on the dowelled concrete designs may be unwise.

(iii) Special attention needs to be given to compaction around and under the reinforcement particularly with the thicker constructions and their correspondingly higher steel contents.

(iv) The advantage of improved ridability is perhaps not significant in relation to unreinforced PQC with sawn contraction grooves.

5.10 DESIGN FOR HIGH FREQUENCY OF TRAFFICKING

5.10.1. The High Frequency design level is nominally 250,000 Coverages by the Design Aircraft (see para 4.6.2). As Defence Estates lacks both experience and research data on pavement performance at this level of use, the construction thickness requirements have been extrapolated beyond proven designs. On this basis the PQC slab thickness for the High Frequency design is 10% greater than that required for the Medium Frequency design.

RIGID DESIGN EXAMPLES

Example 5.1

A rigid pavement is required for nre stands at a small municipal airport, used principally for charter traffic. The majority of departures are Boeing 737-800s.

Guide Reference

1. SUBGRADE: Soil Survey shows k = 30 $MN/m^2/m$

2. AIRCRAFT DATA:

Appendix B a) ACN

Aircraft type	All Up Mass (kg)	RIGID PAVEMENT SUBGRADES - $MN//m^2/m$				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 150	Medium 80	Low 40	Ultra Low 20		
		ACN					
B737-800	79,243	49.3	51.8	54.2	56.1	Dual	1

Appendix B — b) Main Wheel Gear: Dual

Section 4.9, Table 6 — c) Pass–to-Coverage Ratio: 3.2

3. AIRCRAFT USE: Expected departures are 3 Boeing 737-200s per day.

Para 4.7.3 — 4. DESIGN LIFE: 30 years.

5. FREQUENCY OF TRAFFICKING

Section 4.9

$$\text{No of Coverages} = \frac{(30\times365\times3)}{1} = 32850$$

6. DESIGN CRITERIA

a) ACN: from 2a above, using linear interpolation between subgrade values and rounding to the nearest integer, ACN = 55

Para 4.6.2, Table 5 — b) Frequency of Trafficking: Low

Para 5.2.3 — 7. CONCRETE STRENGTH: 4.5 N/mm^2 mean flexural at 28 days

Chart 2 — 8. REQUIRED CONSTRUCTION: 345 mm Pavement Quality Concrete
150 mm Rolled Drylean Concrete

Para. 5.6.3 — Note: if the stands are constructed so that the butt longitudinal joints are normal to the stand centreline, the joints should be provided with load transfer (e.g. dowels) or the slab thickness increased by 25%. i.e.
430 mm Pavement Quality Concrete
150 mm Rolled Drylean Concrete

Para 2.4.2 — 9. CLASSIFICATION:

a) Subgrade Category: Low (C)

b) PCN:ACN of the Boeing 737-800 on a Rigid Low Subgrade =54.2

c) Pavement Type: Rigid (R)

d) Tyre Pressure Limitations: No limitations on a concrete surface (W)

e) PCN 55/R/C/W/T

Example 5.2

Design a rigid pavement for a new parallel taxiway at an international airport used by a wide range of aircraft.

Guide reference

1. SUBGRADE: Soil Survey shows k = 50 $MN/m^2/m$

2. AIRCRAFT DATA:

Appendix B
Section 4.9 Table 6

a) ACNs, Main Wheel Gears and Pass-to-Coverage Ratios.

Aircraft type	All Up Mass (kg)	RIGID PAVEMENT SUBGRADES - $MN//m^2/m$				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 150	Medium 80	Low 40	Ultra Low 20		
		ACN					
A321-200	89,400	56.5	59.4	62.1	64.3	Dual	2.1
A340-500	369,200	72.8	84.7	100	115.3	Dual Tandem	1.3
A330-200	233,900	53.7	62.4	74.3	86.9	Dual Tandem	1.3
B737-800	79,243	49.3	51.8	54.2	56.1	Dual	2.1
B747-400	397,800	52.4	62.7	74.4	85.1	Dual Tandem	1.3
B767-300	159,665	38.3	45.4	54.1	62.5	Dual Tandem	1.3
B777-300ER	352,441	65.8	85.3	109.3	131.5	Tridem	1

3. AIRCRAFT USE: Proposed aircraft use shown in Table 8 (Page 38).

Para 4.7.3

4. DESIGN LIFE: 30 years.

5. DESIGN CRITERIA:

a) ACNs of user aircraft calculated at k = 50 are shown in Table 8.

Section 2.6

The Design Aircraft is the Boeing 777-300ER. Design ACN = 103.

Para 4.6.2 Table 5

b) The Mixed Traffic Analysis is shown in Table 8. The total coverage is 76,775 and therefore Medium Frequency Trafficking is used.

6. CONCRETE STRENGTH: 5 N/mm^2 mean flexural strength at 28 days.

Chart 4

7. REQUIRED CONSTRUCTION: 390 mm Pavement Quality Concrete
175 mm Rolled Drylean Concrete

Para 2.4.2

8. CLASSIFICATION:

a) Subgrade Category: Low (C)

b) PCN is the ACN of the Boeing 777-300ER on a Rigid Low Subgrade = 109.3

c) Pavement Type: Rigid (R)

d) Tyre Pressure Limitations: No limitations on a concrete surface (W).

e) PCN 110/R/C/W/T.

6 Flexible Pavement Design

6.1 GENERAL

6.1.1. For over 50 years Defence Estates' policy has been to construct 'flexible' pavements with either cement or bitumen-bound bases. While an unbound base or sub-base can provide the desired performance, the strict grading requirement together with the need for a high a consistent level of compaction throughout can result in construction problems and unreliable performance. This is particularly true on wet sub-grades, common in the UK. These disadvantages are worsened in the case of pavements subject to regular trafficking by heavy and high tyre pressure aircraft. On the other hand pavements with bound bases permit the use of less stringent specification and give structural benefits over conventional flexible pavements, allowing a saving in thickness over the granular base and sub-base requirements. The bound base designs provide an economic and practical solution and most significantly give reliable performance.

6.1.2. Sometimes, the availability of good quality materials, with or without self-cementing properties, can make convention granular base and sub-base construction a practical and economic choice. However, this type of construction is only recommended on good dry subgrades, where it is possible to achieve the necessary high level of compaction. As the aircraft weight increases there is an increasing possibility that failure to achieve uniform compaction over the whole pavement area will lead to premature rutting due to consolidation of the granular materials. Thus for heavy aircraft (ACN > 50) the granular materials should have a well proven record of performance an the designer may consider proof rolling the base course before laying the bituminous surfacing. The required subgrade conditions are more likely to be found in certain overseas locations than in the UK. For these reasons flexible constructions with unbound bases/sub-bases are not included on the design chart for this Chapter (Chart 4). Chapter 7 includes a design and evaluation chart for these pavements.

6.2 SURFACING

6.2.1. The standard flexible pavement designs to Chart 5 require a minimum surfacing thickness of 100mm. This will usually be made up to a 40mm surface course on a 60mm binder course. A 20mm thick open macadam friction course is not considered to be a structural layer and therefore should not be counted as part of the 100mm surfacing.

6.2.2. The principal bituminous surfacing materials used by Defence Estates are Marshall asphalt surface and base course or hot rolled asphalt surface course on macadam binder course. These materials should meet the specialist performance required of airfield pavements:

(i) High stability to withstand the shear stresses induced by heavy wheel loads and high tyre pressures.
(ii) Good ridability.
(iii) A durable hard-wearing weatherproof surface free from loose material and sharp edges which might endanger aircraft.

6.2.3. To provide good wet weather braking characteristics on a runway an additional surface treatment is usually specified. The standard surface treatments used by Defence Estates are friction course, coarse slurry seal and grooving; the choice of treatment depends on the availability of materials, site geography and performance requirement. A surfacing of porous macadam friction course with cross-falls of 1.5%, gives runways an excellent all-weather friction characteristic. The friction course must have an underlay of at least 100mm of high quality asphalt. A friction course is not recommended in areas where the pores are liable to silt up (e.g. by wind blown sand) or to freeze in extreme winter conditions (e.g. where temperatures of less than -10ºC can be expected to last for periods o greater than 24 hours). Surface dressing should not be use for jet aircraft operations because of its tendency to loose stones which present a FOD (foreign object damage) hazard. High speed taxiways may also need treatment, either with a course slurry seal or by grooving.

6.2.4. For temperate climates, Table 13 gives guidance on the suitability of various surfacing materials. The stability of hot rolled asphalt on a macadam base course is adequate for the frequencies of trafficking and tyre pressures given in Table 13. The frequencies of trafficking assumed in the table apply to a single user aircraft. Where mixed traffic use is envisaged the aircraft with the highest category tyre pressure, not necessarily the Design Aircraft, should be considered at the frequency of trafficking appropriate to that tyre pressure category. For pavements in hot climates the Marshall asphalt specification should be used. For guidance on high tyre pressure aircraft operations on blacktop surfacings see Chapter 8.

Table 13 Suitability of Surfacing Materials (Temperate Climates)

Tyre Pressure	Frequency of Trafficking		
	Low	Medium	High
W (> 1.5 Mpa)	MA	MA	MA
X (up to 1.5 MPa)	HRA/MB	MA	MA
Y (up to 1.0 MPa)	HRA[1]/MB	HRA/MB	MA
Z (up to 0.5 MPa)	HRA[1]/MB	HRA/MB	HRA/MB

MA – Marshall asphalt or alternatively Marshall Dense Tar Surfacing surface course on Marshall asphalt or Marshall DTS binder course.
HRA/MB – Hot rolled asphalt on Macadam binder course.
Note 1 – Dense Tar Surfacing is acceptable as an alternative.

6.2.5. Marshall asphalt is a more highly controlled and consistent material than hot rolled asphalt and is to be preferred wherever a contract is large enough to cover the enhanced level of supervision and testing effort involved. To aid proper control and make sure that the performance criteria will be met, Marshall asphalt should, wherever physically and economically possible, be mixed on site.

6.2.6. If there is a requirement for a fuel resistant pavement surface such as for a runway end or apron a tar-based slurry seal can be used to provide some resistance, although it should be noted that they are susceptible to mechanical damage, especially early in their life, and are not resistant to hydraulic fluid spillage.

6.2.7. Other surfacing materials which have been use to a limited extent by Defence Estates include grouted macadam, concrete blocks and Stone Mastic Asphalt. Grouted macadam and concrete blocks are fuel resistant and can be used on aprons, although fuel can penetrate the joints between blocks to reach lower layers. In many cases grouted macadam has not given good long-term performance, and the performance of Concrete Block Surfacing has been variable. Concrete Block Surfacing should not be used on runways. Research shows that Concrete Block Surfacing has little structural capacity until the blocks rotate sufficiently to come into contact and develop interlock. Because of the surface tolerances required for airfield pavements enough movement to create interlock is unlikely, and a Concrete Block Surfacing will add little to the strength of a pavement. Concrete Block Surfacing should be treated as being equivalent to 50mm of Marshall Asphalt or less. Stone Mastic Asphalt has shown considerable promise, although long-term durability has not been fully proven. Structurally it should be treated as equivalent to Marshall Asphalt.

6.3 BASE

6.3.1. The standard designs in Chart 5 require a bound base construction from the underside of the surfacing down to the subgrade or improved subgrade (see Section 3.8). Therefore a conventional unbound sub-base is not needed.

6.3.2. The bound base materials normally specified by Defence Estates are high strength drylean concrete (Type FH DLC), and Marshall Asphalt. The thickness requirements on the design chart can be made up of any one or a combination of these materials. However having regard to stringent compaction and laying requirements for Marshall asphalt, DLC should normally be used as the first layer of bound base material on the formation / subgrade.

6.3.3. The design model for Chart 5 is based on that used in Reference 11 for the design of standard flexible pavements comprising bituminous surfacing materials on bound bases. However, Chart 5 incorporates higher equivalency factors than those used in Reference 11 to take account of experience gained since that time and more recent full scale testing of high strength drylean concrete bases (Type FH DLC). Further details on materials and design rationale are provided in Appendices C and F.

6.3.4. When laid on low strength subgrades (CBR less than 6%) it may be difficult if the initial layers of Type FH DLC are laid directly on the subgrade to compact the layers sufficiently to obtain the required minimum strength and density. In this situation a working course should be provided before laying the initial layer, either an unbound capping or a sacrificial working course of drylean concrete.

6.3.5. Chapter 7 deals with the evaluation and strengthening of existing pavements incorporating DLC bases laid to the Defence Estates' specification prior to 1989 and now designated Type F DLC, and also of pavements incorporating asphalts, including Hot Rolled Asphalt and Macadam Base Course. Details of these materials are provided at Appendix C. Chart 5 does not apply to these materials.

6.3.6. Chapter 7 also deals with pavements incorporating unbound granular base and sub-base layers; Chart 5 does not apply to them.

6.3.7. The use of DLC as the principal base material results in a pavement with a rigid mode of behaviour which gradually changes to a flexible mode as the stiffness of the pavement reduces after cracks form in the cement-bound layer. Experience shows that DLC cracks into irregular shaped bays, and the cracks eventually reflect through the surfacing. Where the DLC is thick the resultant irregular bays tend to be large giving rise to wide cracks subject to considerable movement. A requisite thickness of blacktop overlay should be provided; it will substantially delay reflective cracking in the surfacing and postpone the need for widespread maintenance with consequent loss in rideability, drainage and friction characteristics. For minimisation of reflection cracking the thickness of bituminous material over the DLC should be in accordance with Defence Estates Design & Maintenance Guide 33[52].

6.3.8. The use of cement stabilised material instead of DLC may be considered as follows:

(i) Cement-stabilised fine or medium-grained material is unlikely to provide long-term load-spreading characteristics comparable to DLC. Following the eventual propagation of cracks in a cement-stabilised layer a fine-grained material will provide minimal aggregate interlock with a consequent loss of stability and load distribution properties. Therefore, cement-stabilised fine or medium-grained material of which more than 60% passes the 5mm sieve should only be considered as a sub-base. See Chapter 7, para 7.4.2.3 for cement-stabilised sub-bases.

(ii) To give performance comparable to a DLC base, strength characteristics of the cement-stabilised material should comply with the minimum requirements for either Type FH DLC or Type F DLC , as described in Appendix C, for use with Chart 5 or Chart 7 respectively. Cement-stabilised materials are more likely to meet Type F DLC requirements, in which case para 6.3.6 should be referred to. The strength requirements for DLC are unlikely to be achieved with any degree of consistency with an in situ stabilised soil.

6.4 SUBGRADE

6.4.1. For details of subgrade characteristics, the CBR test method, subsoil drainage and subgrade compaction requirements, see Chapter 3.

6.4.2. Flexible pavements are more sensitive to sub-grade characteristics than rigid pavements, making the assessment of a representative design CBR more critical than that of a design k. On most sites the soil types at the formation levels are likely to vary. Where the variation occurs in distinct and large areas of the site it may be feasible to consider separate flexible pavement designs. However, if such variation occurs randomly, then a single design based on the limiting soil type (i.e. lowest CBR) may be the only realistic solution. If the change in subgrade support characteristics is considerable, the possibility of differential settlement and densification, particularly in the transitional areas, may need to be considered.

6.4.3. The presence of weak layers in the subsoil must be carefully considered in assessing the design CBR. This is particularly important when the pavement is to be designed for heavy aircraft with multiple wheel main gears which induce significant stress levels at considerable depths below the pavement surface, as reflected in the ACNs for poor subgrades. Para 3.4.7 sets out a procedure for assessing the design CBR when there is a weak underlying layer in the subsoil.

6.4.4. For assessing the subgrade improvement provided by a granular fill see Section 3.8.

6.4.5. The maximum CBR value on Chart 7 is 20%. This is intended to limit the stresses and strains in the bound base materials by imposing a minimum pavement thickness for a given aircraft loading, i.e. the pavement thickness required for CBR 20%. The same subgrade scale is shown on Chart 8, but in this case it is also possible to design for CBR 30% by using the Y-axis only.

6.5 DESIGN OF FLEXIBLE PAVEMENTS WITH BOUND BASES

6.5.1. Chart 5 has been prepared for single, dual, dual-tandem and tridem main wheel gears; see Appendix D for the definition of these gear types. The use of the Chart requires three design parameters:

(iii) The CBR of the subgrade – see Section 6.4 and Chapter 3 for details of subgrade characteristics. If subgrade improvement is to be carried out as detailed in Section 3.8 the increased CBR value will be the appropriate design value.

(iv) The design ACN (see Section 2.6).

(v) The frequency of trafficking – either Low, Medium or High. Chapter 4 defines these traffic levels in terms of Coverages by the Design Aircraft. For calculating the number of Coverages for different areas of pavement and equating the loading effects of different aircraft, see Chapter 4.

6.5.2. Having established the above parameters the Chart is used as follows:

(i) Select the frequency of trafficking (Low, Medium or High); for High Frequency Trafficking see Section 6.6.

(ii) Select the ACN scale appropriate to the Design Aircraft’s main wheel gear type. Enter the Chart with the design ACN and make a horizontal projection until it intersects the vertical projection of the appropriate CBR.

(iii) From the intersection, trace a line parallel to the curves until it intersects the right hand ordinate. Read off the thickness of bound base material required. The minimum surfacing thickness required on top of the base is 100mm. See Section 6.2 for details of surfacing and Section 6.3 for details of bound base construction.

See Examples 6.1 and 6.2.

6.6 HIGH FREQUENCY OF TRAFFICKING

6.6.1. The High Frequency design level is nominally 250,000 Coverages by the Design Aircraft (see para 4.6.2). As Defence Estates lacks both experience and research data on pavement performance at this level of use, the construction thickness requirements have extrapolated beyond proven designs. On this basis the required thickness of bound base material for the High Frequency design is increased to provide a total pavement thickness which is 8% greater than that required for the Medium Frequency design.

FLEXIBLE DESIGN EXAMPLES

Example 6.1

Design a flexible pavement using a Type FH Bound Base Material for a taxiway at small municipal airport used principally for charter traffic. The majority of departures are Boeing 737-800s.

Guide reference

1. SUBGRADE: Soil Survey shows CBR 5%.

2. AIRCRAFT DATA:

Appendix B — a) ACN

Aircraft type	All Up Mass (kg)	FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 15	Medium 10	Low 6	Ultra Low 3		
		ACN					
B737-800	79,243	42.9	45.4	50.4	55.3	Dual	2.1

Appendix B — b) Main Wheel Gear: Dual

Section 4.9 Table 6 — c) Pass-to-Coverage Ratio: 2.1

3. AIRCRAFT USE: Expected Departures are 3 Boeing 737-800s per day.

Para 4.7.3 — 4. DESIGN LIFE: 20 years.

Section 4.9 — 5. FREQUENCY OF TRAFFICKING

No of Coverages = $\dfrac{(20\times365\times3)}{2.1}=10429$

6. DESIGN CRITERIA

a) Design ACN: from 2a above, using linear interpolation between subgrade values and rounding to the nearest integer ACN = 52

Para 4.6.2,Table 5 — b) Frequency of Trafficking: Low

Chart 5 — 7. REQUIRED CONSTRUCTION: 100mm Surfacing
375mm High Strength Bound Base Material

Para 6.3.7 — If most of the Bound Base Material is to be Type FH Drylean Concrete, a minimum thickness of bitumonous material should be provided to restrict reflective cracking. Defence Estates Design & Maintenance Guide 33 suggests that 220mm is required for a long-life pavement.

e.g. 40mm Marshall Asphalt Surface Course
60mm Marshall Asphalt Binder Course
60mm Marshall Asphalt Base Course
60mm Marshall Asphalt Base Course
255mm TypeFH Drylean Concrete.

Para 2.4.2 — 8. CLASSIFICATION:

a) Subgrade Category: Low (C).

b) PCN is the ACN of the Boeing 737-200 on a Flexible Low Subgrade =51.

c) Pavement Type: Flexible (F)

Table 13 — d) Tyre Pressure Limitations: No limitations for Marshall Asphalt surfacings (W).

e) PCN 51/F/C/W/T.

Example 6.2

Design a flexible pavement using a Type F Bound Base Material for a runway at an international airport used by a wide range of aircraft.

Guide Reference

1. SUBGRADE: Soil Survey shows CBR 10%

Appendix B, Section 4.9 Table 6

2. AIRCRAFT DATA:

a) ACNs, Main Wheel Gears and Pass-to-Coverage Ratios

Aircraft type	All Up Mass (kg)	FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design	Pass-to-Coverage Ratio (Table 6)
		High 15	Medium 10	Low 6	Ultra Low 3		
		ACN					
A321-200	89,400	49.4	52	57.6	63.2	Dual	3.2
A340-500	369,200	75.3	82.2	97.8	129.8	Dual Tandem	1.8
A330-200	233,900	58.5	63.5	73.8	99.8	Dual Tandem	1.8
B737-800	79,243	42.9	45.4	50.4	55.3	Dual	3.2
B747-400	397,800	53	59	72.5	94.1	Dual Tandem	1.8
B767-300	159,665	39.5	43.3	51.1	69.9	Dual Tandem	1.8
B777-300ER	352,441	63.6	71.1	89.1	120.1	Tridem	1.4

3. AIRCRAFT USE: Proposed aircraft use shown in Table 9 (page 40).

Para 4.7.3.

4. DESIGN LIFE: 20 years.

5. DESIGN CRITERIA.

a) ACNs of the user aircraft calculated at CBR 10% are shown in Table 9.

The Design Aircraft is the Airbus 340-500. Design ACN = 82

Para 4.6.2 Table 5

b) The Mixed Traffic Analysis is shown in Table 9. The total coverages are 33,442, therefore Medium Frequency Trafficking is used.

Chart 6

6. REQUIRED CONSTRUCTION: 100mm Surfacing
525mm High Strength Bound Base Material

Para 6.3.7

If most of the Bound Base Material is to be Type F Drylean Concrete, a minimum thickness of bitumonous material should be provided to restrict reflective cracking. Defence Estates Design & Maintenance Guide 33 suggests that 220 mm is required for a long-life pavement, if the Surface Course is grooved.

e.g. 40mm Marshall Asphalt Surface Course
60mm Marshall Asphalt Binder Course
60mm Marshall Asphalt Base Course
60mm Marshall Asphalt Base Course
305mm TypeFH Drylean Concrete.

Para 2.4.2

7. CLASSIFICATION

a) Subgrade Category: Medium (B)

b) PCN is the ACN of the Airbus A340-500 on a Flexible Medium Subgrade = 82

c) Pavement Type: Flexible (F)

Table 13

d) Tyre Pressure Limitations: None (W)

e) PCN 82/F/B/W/T

7 Pavement Evaluation and Strengthening

7.1 METHODS OF EVALUATION

7.1.1. For various reasons it may be necessary or desirable to reappraise the bearing capacity of a pavement. This would apply in any of the following circumstances:

(i) A mid/end of life reassessment of the pavement to plan future maintenance work and/or rehabilitation.
(ii) The pavement has been disused for some time and is to be rehabilitated.
(iii) The pavement is to be strengthened for regular use by heavier aircraft.
(iv) After several years service it has become apparent that the pavement's strength has been reduced and it is showing signs of premature fatigue.
(v) There has been a change in the classification system.

7.1.2. Evaluation is carried out by 'reverse design', with pavement inputs determined by a site investigation. There are no in situ test methods that directly measure pavement strength.

7.1.3. Reverse design works best when used with a pavement management system which includes periodic maintenance inspections and records of construction, subgrade characteristics and aircraft movements. Defence Estates has used reverse design extensively for over 35 years. The method, as presented in this guide, requires the existing pavement to be structurally equated to one of the standard constructions included in the design and evaluation Charts 1-8. Using the Charts in reverse, the strength of the pavement can be determined.

7.1.4. Any evaluation must be weighted by consideration of factors such as the pavement condition, records of its operational use, future operational requirements and an estimate of the pavement's residual fatigue life which must necessarily be subject to engineering judgement.

7.1.5. If strengthening is required an overlay thickness can be calculated using the procedures set out in Sections 7.5 to 7.10

7.2 INVESTIGATIONS FOR EVALUATION AND STRENGTHENING

7.2.1. Reverse design requires details of pavement construction and condition, and possibly a record of its use. Section 7.3 provides an overview on assessment of pavement condition and residual fatigue life. An evaluation is then made using the methods described in Section 7.4.

7.2.2. Construction records are not always reliable and rarely give any indication of material condition. A site investigation to determine the pavement inputs for reverse design is therefore usually necessary. The investigation will:

(i) ascertain the existing construction,
(ii) ascertain the condition of the pavement,
(iii) ascertain material condition.

7.2.3. A detailed description of site investigation and interpretation methods for airfield pavements in given in Appendix I.

7.3 PAVEMENT CONDITION AND RESIDUAL LIFE

7.3.1. Accounting for the Pavement Condition

7.3.1.1 Structural deterioration of pavement layers will reduce their load bearing capacity and suitable allowances may have to be made in evaluation or overlay design. The formulae for design and evaluation of composite and multiple slab pavements (Sections 7.9 and 7.10) include specific Condition Factors which take account of cracking of underlying concrete slabs. In some other cases the effective pavement thickness can be reduced by an analysis of fatigue consumption (para 7.3.2.2). If the pavement is showing signs of serious structural distress materials may be downgraded to ones of a lower structural value (paras 7.3.1.5 and 7.3.1.6). The following paragraphs describe how to determine Condition Factors and when they should be used; advice on assessing Condition Factors by in situ testing is given in Appendix I.

7.3.1.2 Normal deterioration caused by live loading or climatic effects is built into the methods presented in this document. An airfield operator expects a pavement with a given strength when new, and that the pavement will have the same strength until the end of its life before major maintenance. The operator does not expect to change aircraft use in the middle of the pavement life because normal deterioration has occurred. Unless deterioration is excessive the pavement classification should not be altered during the design life, and future performance must be defined by considering together the initial strength, the residual fatigue life and the expected life of the pavement materials. If a mid/end of life evaluation is being made to obtain a classification or to estimate a pavement's residual fatigue life, condition factors should not be applied to the most recent layers of construction. The condition factors for other layers should not generally be changed from those used in the design of the last strengthening, unless it has been agreed with the operator that excessive deterioration is best expressed by a decrease in PCN rather than by a decreased pavement life.

7.3.1.3 If a pavement is being strengthened, the overlay thickness requirement should be calculated on the basis that it provides a renewed design life (see Section 4.6). The evaluation of the existing pavement should therefore reflect its current condition. This will generally necessitate making due allowance for the deterioration of all layers in the existing construction.

7.3.1.4 If deterioration is excessive the likely causes are overloading, fatigue, poor quality construction or reduction in subgrade strength due to a change in moisture content. The reason for the failure should be established and an evaluation of the various layers of construction made. The pavement should be overlaid or reconstructed to restore serviceability at the current classification. Where adjacent level constraints are critical reconstruction may be the only choice.

7.3.1.5 Structural failure of concrete pavements is indicated by cracking of bays in the wheel path (see Appendix F). The following points should be considered:

(i) Condition Factors related to cracking of concrete are given in Table 18 and should be applied to slabs that are part of a composite construction, or to concrete slabs that are to be overlaid, including all layers of existing multiple slab constructions (see Sections 7.9 and 7.10). The Condition Factors should not be applied to the top layer of an existing multiple slab construction for a mid/end of life evaluation.

(ii) If the state of the pavement is significantly worse than the failure condition described in Appendix F the concrete is down-graded to an equal thickness of drylean concrete. Alternatively localised areas of severe failure could be reconstructed and the pavement as a whole assigned an appropriate condition factor.

7.3.1.6 Structural failure of a flexible pavement is indicated by rutting with associated heaving and/or cracking in the wheel paths (see Appendix F). (Note: Rutting is sometimes due to compaction of the pavement layers or subgrade by aircraft operations which while giving rise to a serviceability problem, does not cause a loss of bearing capacity). Provided a pavement is not showing signs of a severe failure it should still possess enough residual strength to form an integral part of a strengthened construction. In developing a design concept for rehabilitation of a failed flexible pavement the following points need to be considered:

(i) If the failure has occurred suddenly or unexpectedly or there is uncertainty in assessing the condition of the various layers of construction the engineer may consider it wise to do deflection tests to reassess the behaviour of the pavement. Tests in failed areas and in areas adjacent to the wheel tracks may indicate whether pavement strength has been significantly reduced.

(ii) Severe shear failure of a pavement (i.e. rutting and heave in excess of twice that given as the failure criterion in Appendix F) is likely to result in loosening of the pavement construction and/or subgrade. In these circumstances either reconstruction, or recompaction and reappraisal of material strengths should be done.

(iii) If small areas of severe rutting have occurred, the pavement may be reconstructed locally and assigned the same residual strength as the sections which are deteriorating normally.

(iv) A surface showing appreciable cracking but with little or no ravelling of the cracks should not be considered as being any better than a granular base (see para 7.8.3). Again, if the crack pattern along the wheel tracks is extensive and well defined with ravelling along the cracks, the surfacing should not be considered as being any better than a granular sub base (see para 7.8.4), or it should be removed before overlaying.

(v) Structural failure resulting from shear failure within unbound base and sub-base layers or within the subgrade can lead to substantial decreases in the in situ CBR values of the layers. Ideally a failed base should be regraded and recompacted but this will be difficult if there is a thick layer of bituminous materials overlying it. Recompaction of sub-bases and the subgrade will only be possible if the pavement is completely reconstructed. The alternative to recompaction is to do in situ CBR tests in the failed and unfailed areas and then use the lowest results for overlay design. This would normally lead to the downgrading of granular bases and sub-base to sub-base and capping layer respectively and to a reduced CBR value being taken for the subgrade.

(vi) If subgrade shear failure is due to the reduced load bearing characteristics of a degraded bound base the structural value of the bound base will also need to be reconsidered. Depending on its density and grading, an aged and embrittled bituminous bound base will be reassessed as a granular base course or granular sub-base. An extensively cracked drylean concrete base will be worth little more then a granular sub-base.

(vii) When an underlying concrete slab has undergone extensive multiple cracking with subsequent shear failure of the subgrade the broken slabs should be equated to an equivalent thickness of lean concrete base. (See Table 18). As in (vi) above, the subgrade should also be reassessed.

7.3.2. Estimating Residual Fatigue Life

7.3.2.1 Records of aircraft use are essential for residual fatigue life calculations, and are very useful when designing pavement overlays where adjustments can be made to existing pavements thicknesses to allow for past fatigue. They are also helpful in assessing the classification of a pavement, since in conjunction with the pavement condition they provide a good indication of the integrity and strength of the pavement. The greater the previous use the more significant this factor becomes in the evaluation. Care should be taken to ensure that too great a reliance is not put on the evidence of a few movements.

7.3.2.2 If a pavement has been regularly used for several years by aircraft at or approaching its PCN it must be considered that a portion of the structural fatigue life has been used up. A mid/end of life appraisal of a pavement will indicate its remaining structural life. The first step is to evaluate the pavement. As explained in paragraph 7.3.1.2 condition factors for the most recent layers of construction need not be incorporated into a reverse design and generally condition factors for other layers should not be altered from those used for the previous strengthening design. The next step is to use the records of aircraft movements to assess the past fatigue, in terms of an aircraft with an ACN equal to the PCN of the pavement. The procedures for mixed traffic analysis detailed in Chapter 4 should be used for this purpose. The residual fatigue life is the difference between the design Coverage level for the evaluation and the equivalent number of Coverages by the aircraft to date (see Example 7.9).

7.4 EVALUATION BY REVERSE DESIGN

7.4.1. Procedure

7.4.1.1 Where an existing pavement can be equated to one of the standard constructions in Chapters 5 and 6 reverse design is carried out using the design Charts 1-8. For existing flexible pavements equating to the standard construction described in Chapter 6 but with Type F DLC Chart 7 can be used in the same way as Chart 5 (para 7.4.2.8). In other cases use Charts 5, 7 and 8 as described in Section 7.6 and 7.7. The subgrade strength, pavement thickness, material properties and the relevant frequency of trafficking are determined and then entered on the charts to give the ACN of the Design Aircraft. The ACN is then modified to allow for the difference between the actual subgrade strength and one of the standard subgrade categories in the ACN-PCN method.

7.4.1.2 If the pavement construction is not similar to one of those shown on the relevant Charts then equivalency factors are used to convert materials in the actual pavement to an equivalent thickness of one of the standard materials assumed in the Charts. The methods of equating various types and combinations of construction to an appropriate type covered by the Charts are set out in and, Figure 29 and Figure 30. The procedures for converting composite and multiple slab constructions to equivalent standard ones are set out in Sections 7.9 and 7.10.

7.4.1.3 Pavement condition is dealt with either by using residual life calculations to allow for past fatigue and give an equivalent pavement thickness (see Example 7.9) or, in the case of concrete slabs, by using ‘Condition Factors’ as set out in Table 18. An explanation of determining Condition Factors and when Condition Factors should be used is given in section 7.3.

7.4.2. Pavement Constructions

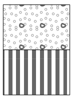
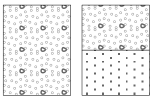
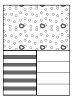
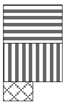
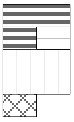
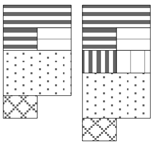
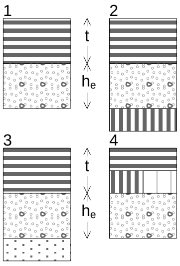
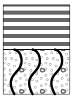
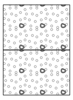
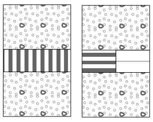
7.4.2.1 Many existing pavements may not directly correspond with the construction assumed for the design charts. Techniques for dealing with them are described below.

7.4.2.2 Subgrade improvement: Capping layers under flexible pavements and granular sub-bases under rigid pavements are allowed for by calculating an effective subgrade strength using the techniques described in Chapter 3. For capping layers, evaluation will be an iterative process as it is first necessary to estimate the ACN of the Design Aircraft from which an effective CBR at the formation is calculated. The effective subgrade strength is the value entered on the charts.

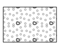
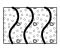
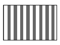
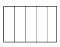
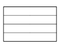
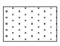
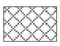
7.4.2.3 Cement-stabilised soils: Cement-stabilised soils under rigid pavements should be converted to an equivalent thickness of drylean concrete (see para 5.4.5 and Figure 26). In flexible pavements cement-stabilised gravels and crushed rock can be treated as a bound base material (see para 6.3.4), otherwise cement-stabilised soils should be converted to an equivalent thickness of granular sub-base, using the equivalency factors given in Table 17, and Chart 8 for the evaluation.

7.4.2.4 Excess or deficiency of Type R DLC under rigid pavements: Reverse design using Charts 1-4 implies that a certain thickness of Type R DLC base exists for a given concrete slab thickness and subgrade strength. If this thickness does not exist in practice the drylean concrete thickness can be altered by converting PQC thickness to drylean concrete thickness or vice versa, using an equivalency factor for PQC to drylean concrete of 3. The maximum deficiency of drylean concrete is 100mm (i.e. not more than 33mm of PQC should be converted to drylean concrete) and the maximum excess is 50mm (i.e. not more than 50mm drylean concrete should be converted to PQC). Any additional thickness of drylean concrete should be ignored. If the PQC-drylean concrete conversion is insufficient to give the required thickness of drylean concrete the existing drylean concrete should be ignored.

Table 14 Reverse design and overlay design procedures

Construction	**Pavement Type**	**Procedure**		**Chart (Example)**
		Reverse Design	**Overlay Design**	
	Rigid (PQC on Type R DLC)	If base thickness greater or less than the design chart requirement para 7.4.2.4	See Composite or Multiple Slab	Chart 1, 2, 3, 4 (7.1)
	Rigid (PQC on the subgrade or on an unbound sub-base)	Para 7.4.2.2 Section 7.6 Effective k on a granular sub-base para 3.8.4	See Composite or Multiple Slab	Chart 6 (7.2)
	Rigid (PQC on a bituminous base)	Para 7.4.2.5.	Para 7.5.4	Chart 1, 2, 3, 4
	Flexible (100 mm Marshall Asphalt surfacing on Type FH DLC or Marshall Asphalt Bound Base Material, on an optional capping layer)	Para 7.4.1.1 *Capping layer* Para 7.4.2.2 Effective CBR on a capping layer para 3.8.2	Para 7.5.2	Chart 5 (7.4, 7.9)
	Flexible (100 mm Marshall Asphalt surfacing on Type F DLC or Bituminous Bound Base Material, on an optional capping layer)	Para 7.4.2.8 Section 7.7 *Capping layer* Para 7.4.2.2 Effective CBR on a capping layer para 3.8.2	Para 7.5.2 Section 7.7	Chart 7
	Flexible (100 mm Marshall Asphalt surfacing on unbound granular base and sub-base, or a combination of Bound Base Material and unbound materials, on an optional capping layer)	Para 7.4.2.9, Para 7.4.2.10 Section 7.8 *Capping layer* Para 7.4.2.2 Effective CBR on a capping layer para 3.8.2	Para 7.5.2 Section 7.8	Chart 8
Case  $\beta = t / h_e$	Composite $\beta \leq 0.5$ Type 1 $\beta \geq 1$ Type 2 $0.5 < \beta < 1$ Type 3	Para 7.4.2.11 Section 7.9	Para 7.5.3 Section 7.9	*Type 1* Case 1, 3, 4 Chart 6 Case 2 Chart 1, 2, 3, 4 (7.3) *Type 2* Case 1, 2, 4 Chart 5 or 7 Case 3 Chart 8 (7.5) *Type 3* Case 1, 4 Chart 6, and 5 or 7 Case 2 Chart 1, 2, 3, 4, and 5 or 7 Case 3 Chart 6 and 8 (7.8)
	Composite (Crack and Seat)	Para 7.4.2.12	Para 7.5.3.3	Chart 5
	Rigid (Multiple Slab)	Para 7.4.2.13 Section 7.10	Para 7.5.5 Section 7.10	Chart 1, 2, 3, 4, 6 (7.10)
	Rigid (Multiple Slab with cement-bound or bitumen-bound inter-layer)	Para 7.4.2.13 Section 7.10	Para 7.5.5 Section 7.10	Chart 1, 2, 3, 4, 6 (7.2)

Key

	Pavement Quality Concrete		Crack and Seat		Type R or FH DLC or equivalent cement-bound base		Type F DLC or equivalent cement-bound base
	Marshall Asphalt		Bituminous Bound Materials other then Marshall Asphalt		Granular base and / or sub-base		Granular fill beneath flexible pavements

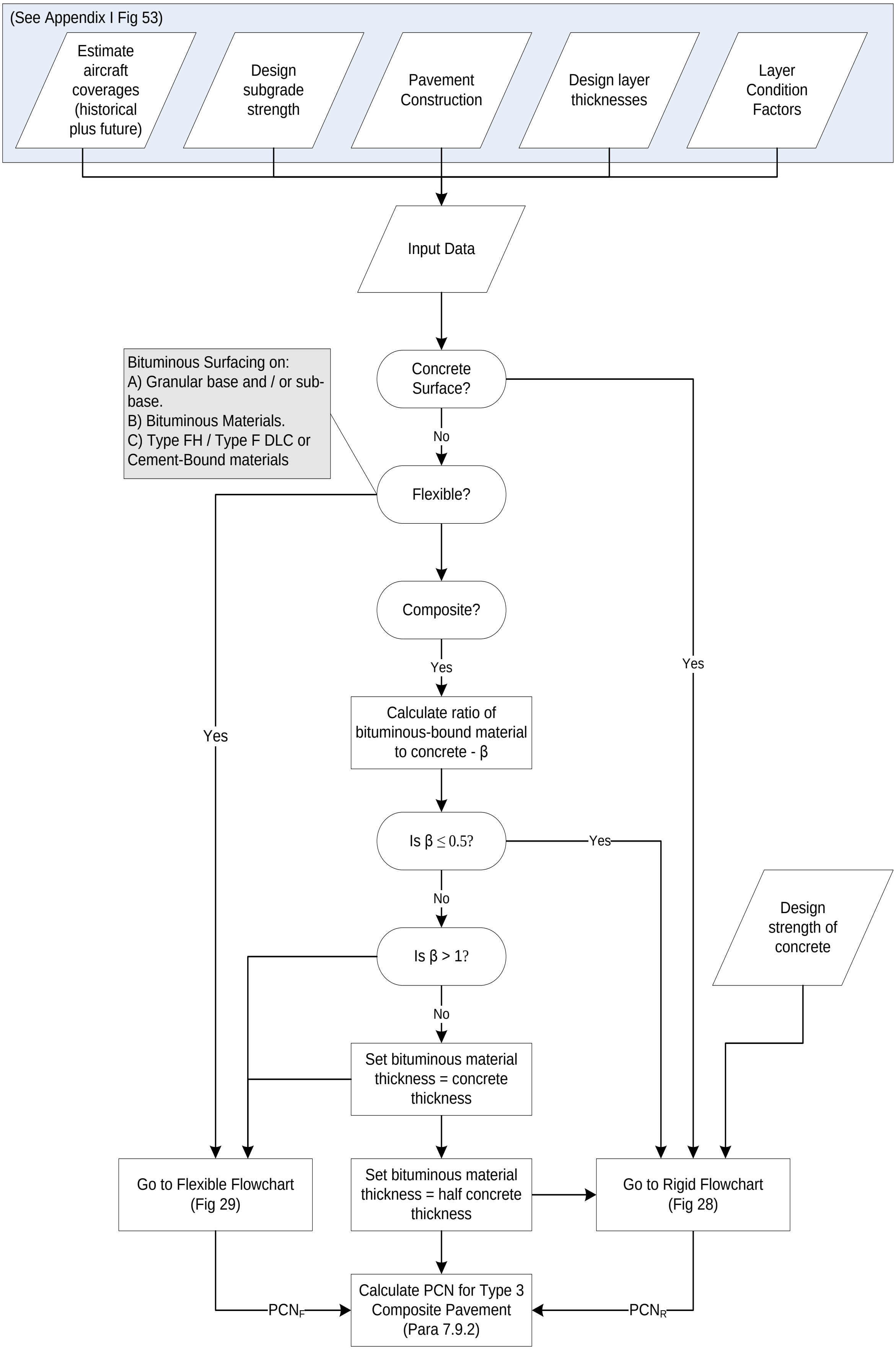


Figure 28 Flow charts for the evaluation of airfield pavements

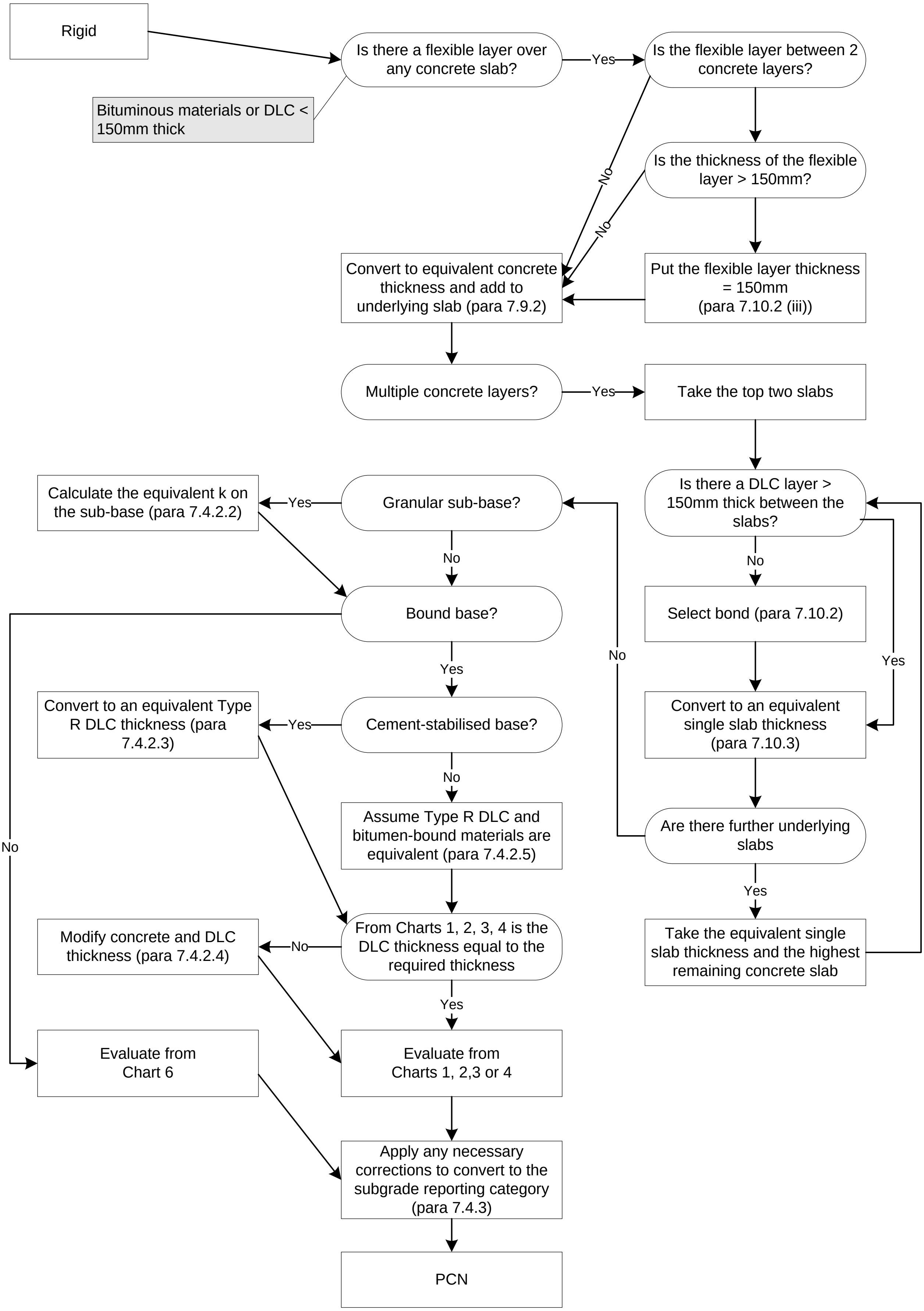


Figure 29 Reverse design for rigid pavements

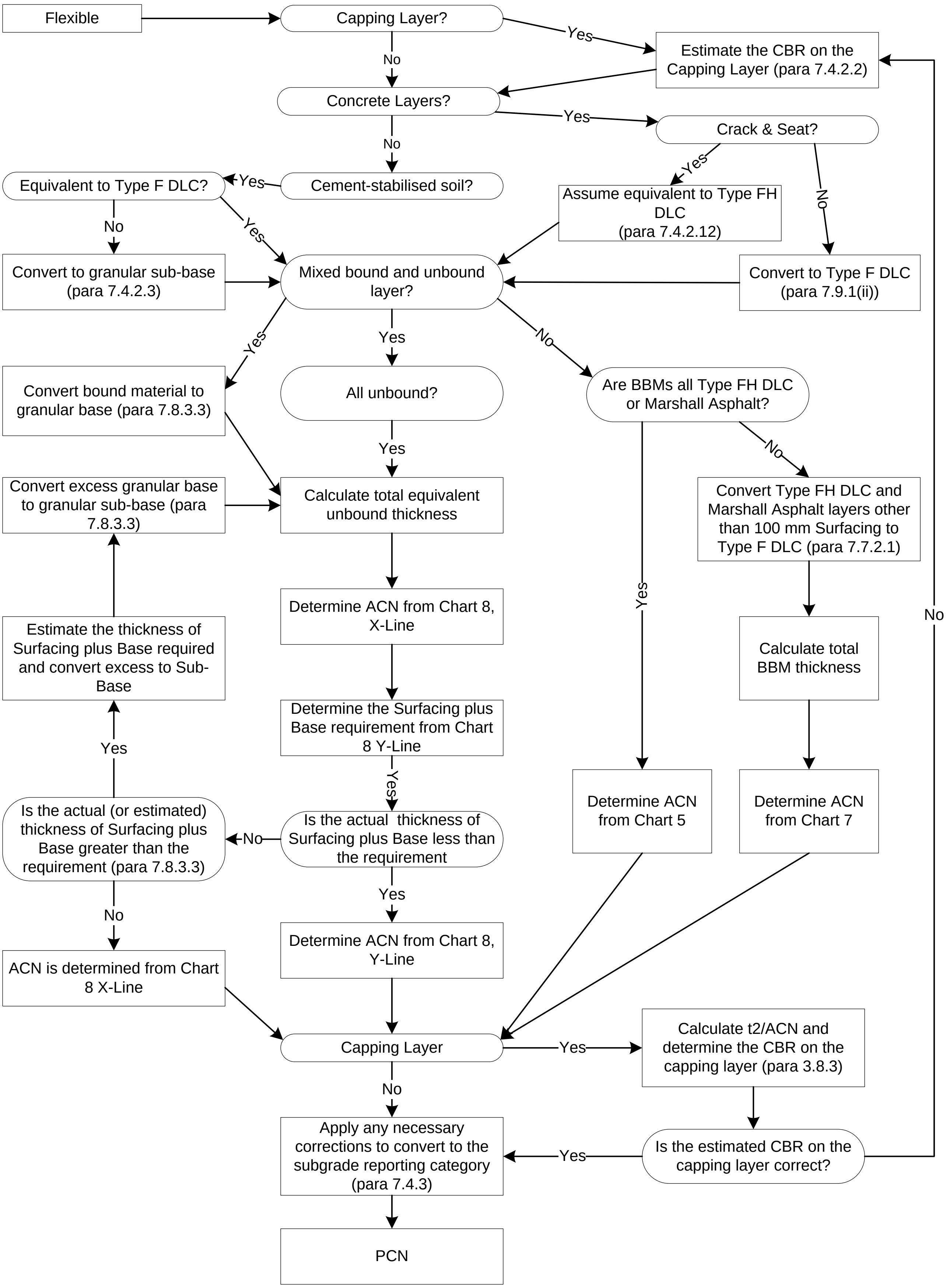


Figure 30 Reverse design for flexible pavements

7.4.2.5 Rigid pavements with bituminous base layers: The bituminous base layers should be treated as Type R DLC.

7.4.2.6 Rigid pavements without a bound base: These pavements are evaluated using Chart 6, as described in Section 7.6.

7.4.2.7 Dowelled and/or reinforced concrete: For dowelled concrete pavements the slab should be treated as described in Sections 5.7 or 7.6 for rigid pavements with and without bound bases respectively. Jointed reinforced concrete pavements (see Section 5.8) should be taken as plain concrete. Continuously reinforced concrete pavements (see Section 5.9) are outside the scope of this guide and should be evaluated using the method by which they were originally designed.

7.4.2.8 Flexible pavements incorporating Type F DLC, as described in Appendix C: These should be evaluated using Chart 7 as described in Section 7.7.

7.4.2.9 Flexible pavements incorporating unbound granular bases and sub-bases: These pavements are evaluated using Chart 8, as described in Section 7.8.

7.4.2.10Mixed bound and unbound flexible constructions: The procedure for dealing with these constructions is set out in Section 7.8. Bound layers should be converted to unbound layers so that the strength at the top of the unbound layers is properly checked in the evaluation. Converting an unbound layer to a bound layer risks missing the possibility that the surface of the unbound layer is the critical point for the pavement strength.

7.4.2.11 Bituminous layers on concrete (including bituminous layers on thin drylean concrete regulating courses on concrete): These pavements are defined as composite and the techniques for evaluating them are described in Section 7.9. Thin drylean concrete regulating courses will crack and behave as a flexible material; they should therefore be included as part of the bituminous material thickness.

7.4.2.12 When an existing concrete slab has been or is to be treated by Crack and Seat techniques before overlaying the cracked concrete should be treated as Type FH DLC as described in Appendix C, and the pavement designed or evaluated using Chart 5.

7.4.2.13 Multiple slabs: Concrete slab on concrete slab, concrete on bituminous layers on concrete and concrete on drylean concrete on concrete are evaluated using the techniques described in Section 7.10.

7.4.2.14 Concrete Block Surfacing: The structural contribution of Concrete Block Surfacing is discussed in Section 6.2. Concrete Block Surfacing provides very little structural strength, in particular on strong bases, it should be considered as equivalent to no more than 50mm bituminous surfacing.

7.4.3. Determination of PCN

The allowable ACN obtained from the Charts corresponds to the actual subgrade value. To establish a PCN several aircraft should be selected with ACNs at or near this value so that the ACN corresponding to the reported subgrade category can be interpolated. This is the reverse procedure to that described in Section 2.6 and Examples 2.1 and 2.2; it is shown in Example 7.3. If the allowable ACN is greater than that for any existing aircraft the ACN adjustment (to correspond with the reported subgrade category) will have to be based on a percentage increase or decrease determined from the aircraft with the closest ACN. If the subgrade strength is greater than that represented by CBR 15% for flexible pavements or k = 150MN/m^2/m for rigid pavements then the allowable ACN should be modified as described in Section 2.6 to give the PCN. The choice of the pavement type, i.e. rigid (R) of flexible (F), should be based on the chart used for the evaluation. If Charts 1 to 3 or 5 are used the pavement is classified as rigid; if Charts 4 or 6 are used the pavement is classified as flexible. Composite Pavements Type 3 (see para 0) should be categorised as flexible (F).

7.5 PAVEMENT STRENGTHENING (DESIGN OF OVERLAYS)

7.5.1. Procedure

7.5.1.1 The procedure for establishing the strengthening requirements is as follows:

(i) The existing pavement is evaluated by reverse design (see Section 7.4).

(ii) The likely composition and mode of behaviour (i.e. rigid or flexible pavement incorporating a bound or unbound base) of the strengthened pavement is addressed and the appropriate design/evaluation chart is used to establish the full depth pavement required for the Design Aircraft; Table 14 can be used as a guide for selecting the appropriate design method and chart together with the procedures set out in paras 7.5.2 to 7.5.5.

(iii) The existing pavement evaluated in (i) above is assigned an equivalent structural value in terms of the new construction calculated in (ii) above. For composite pavements this necessitates using the semi-empirical equivalency factors given in Section 7.9. When an existing flexible pavement is to be overslabbed it can only be structurally equated to a rigid pavement base (see para 7.5.4).

(iv) If a flexible overlay is being provided the required thickness is given by the difference between (ii) and (iii) above. For a concrete overslab the thickness can be established using the method set out In para 7.5.4, or in the case of multiple slab construction from the empirical design method set out in para 7.5.5 and Section 7.10.

7.5.2. Flexible Overlays on Existing Flexible Pavements

7.5.2.1 The existing pavement is evaluated using the procedures described in Section 7.4.

7.5.2.2 If a renewed design life (see Section 4.5) is to be provided by the strengthening overlay an additional thickness of overlay is needed to allow for the reduced effective value of the existing pavement (see Example 7.9).

7.5.2.3 Where the existing construction is showing signs of impending failure or deterioration of any layer, its structural value is appropriately reduced below its original design value and a strengthening overlay provided to give a renewed design life. See para 7.3.1.6 for assessment of flexible pavement constructions which are showing signs of fatigue.

7.5.2.4 If the pavement is to be used by a Design Aircraft with an ACN greater than the PCN of the pavement the required construction is obtained from Chart 5 (see Chapter 6), Chart 7 or Chart 8 (see Sections 7.7 and 7.8), whichever is appropriate. The overlay requirement is then the shortfall in construction between the new requirement and the existing.

7.5.3. Flexible Overlays on Existing Rigid Pavements

7.5.3.1 Unless it is necessary to retain a surface with a high resistance to fuel spillage and jet blast strengthening of a rigid pavement will generally be more expediently and economically achieved by a flexible overlay, in spite of the problems of cracking of bituminous layers laid over unreinforced concrete. The thickness of the bituminous surfacing over existing unreinforced concrete should not in any case be less than 100mm. If the existing concrete is jointed early reflective cracking will occur at the transverse joints. For minimisation of reflection cracking the thickness of bituminous material over the DLC should be in accordance with Defence Estates Design & Maintenance Guide 33[52]. In apron areas, either concrete blocks or grouted macadam will give adequate resistance to fuel spillage and jet blast. However, as stated in Section 6.2 these surfacing materials are not yet fully proven.

7.5.3.2 Flexible overlays on existing rigid pavements are defined as composite pavement and are designed using the methods described in Section 7.9.

7.5.3.3 Numerous methods have been tried in attempts to control reflective cracking of existing joints and cracks in concrete pavements through flexible overlays[52]. The most effective technique has been found to be crack and seat, where the existing concrete slab is cracked at regular intervals to minimise movement and cracks and joints. Overlay design should be based on the recommendations of para 7.4.2.12. The specification and construction of crack and seat overlays is described in Reference 53. The discussion of design in the reference is superceded by this document.

7.5.4. Overslabbing Existing Flexible Pavements

7.5.4.1 The overslab design method considers the existing flexible pavement either as a bound base or simply an improved subgrade. When the existing pavement includes a good quality blacktop surfacing and bound base of adequate thickness and is in sound condition, it may be considered equivalent to a drylean concrete base; Chart 1, 2, 3 or 4 (see Chapter 5) may be used to design the overslab. Otherwise, the existing pavement should be equated to a granular sub-base.

7.5.4.2 If there is a difference in thickness between the actual bitumen/cement-bound flexible construction and the base requirements of Charts 1, 2, 3 or 4, the overslab thickness may be modified as described in para 7.4.2.3.

7.5.5. Overslabbing Existing Rigid Pavements

7.5.5.1 The basis of the design method is the multiple slab empirical design formula developed by the US Army Corps of Engineers[35] and described in Section 7.10.

7.5.5.2 The joint layout of the overslab should as far as possible correspond with that of existing slab unless the overslab is at least 1.25 x the thickness of the existing slab, or the existing slab is showing multiple cracking.

7.5.5.3 To allow for differential temperature effects the minimum top slab thickness for an unreinforced and undowelled slab should not be less than that given in Table 15.

Table 15 Minimum Top Slab Thickness for a Multiple Slab Construction

	Minimum top slab thickness (mm)	
ACN for k = 150 $MN/m^2/m$	Low Frequency Trafficking	Medium Frequency Trafficking
>50	275	300
41-50	250	275
31-40	225	250
21-30	200	225
15-20	175	200
15	150	175

7.6 CONCRETE SLABS LAID ON THE SUBGRADE OR ON A GRANULAR SUB-BASE (DESIGN, REVERSE DESIGN AND OVERLAY DESIGN)

7.6.1. General

7.6.1.1 Chart 6 has been developed for the design or evaluation of PQC slabs founded on either a granular sub-base or directly onto the subgrade.

7.6.1.2 The same design model as that described in Appendix F for new reinforced, rigid pavement designs was used to produce Chart 6, except that the structural contribution of the lean concrete base was not included (i.e. no enhancement of subgrade support taken) and a reduced value of load transfer at transverse joints is adopted for slabs less than 300mm thick. The pavement designs have been linked directly to ACNs as described in Appendix F. The omission of a cement-bound base layer allows the three standard main wheel gear types (i.e. single, dual and dual-tandem (see Appendix D) to be included on one chart.

7.6.1.3 The improvement in the subgrade support provided by a granular sub-base can be assessed using Figure 10 (see Section 3.8).

7.6.2. Use of Chart 6

7.6.2.1 The use of design/evaluation Chart 6 requires the following parameters:

(i) Flexural strength of the concrete. This can either be established from construction records (28 day core strengths) or by tests on samples taken from the pavement (see para I8.5).

(ii) If the Chart is being used for evaluation purposes, the thickness of the concrete slab and granular sub-base (if any).

(iii) If Chart 6 is being used for design purposes the design ACN (see Section 2.6).

(iv) The Modulus of Subgrade Reaction k. Chapter 3 and Section 5.5 give details of subgrade characteristics. If subgrade improvement in accordance with Section 3.8 is to be allowed for, the increased k will be appropriate for design.

(v) The frequency of trafficking; either Low, Medium or High. Chapter 4 defines these traffic levels in terms of Coverages by the Design Aircraft. For equating the loading effects of different aircraft, see Chapter 4.

7.6.2.2 In addition to the above parameters the bay layout and the load transfer effectiveness of transverse joints also have a significant bearing on the future performance of the pavement. If it is suspected that load transfer is substantially below that assumed in the rigid pavement design model (see Appendix F) the evaluation will need to be done conservatively. For details of joints and spacing requirements see Section 5.3.

7.6.2.3 For slabs less than 300mm thick a fully dowelled pavement should provide a significantly greater load transfer than that assumed in the design model for Chart 6. Table 17 gives allowable reductions in the PQC thickness requirements of Chart 6 for fully dowelled slabs (i.e. dowelled expansion, construction and contraction joints).

Table 16 Dowelled PQC Pavements on the Subgrade or on a Granular Sub-Base

Chart 6 Design thickness of PQC (mm)	Allowable reduction in PQC thickness for fully dowelled slabs (mm)
300	0
265	15
230	30
185	35
The minimum slab thickness is 150mm	

7.6.2.2 Having established the design parameters, Chart 6 is used in the same way as Charts 1 to 4 for design. The procedure for evaluation is as follows:

(i) Select the appropriate PQC thickness on the right hand ordinate.
(ii) Make a horizontal projection until it intersects the vertical projection of the appropriate k. From this intersection point trace a line parallel to the curves until it intersects the left-hand ordinate which is also the k = 20 line.
(iii) At the k = 20 line make a horizontal projection; this projection must be maintained.
(iv) Select the design frequency of trafficking (i.e. Low, Medium, High); for High Frequency of Trafficking see Section 5.10. Make a horizontal projection until it intersects the appropriate flexural strength.
(v) Make a vertical projection until it intersect the horizontal projection maintained from (iii) above. Read off the design ACN.

7.7 FLEXIBLE PAVEMENTS INCORPORATING TYPE F DLC (DESIGN, REVERSE DESIGN AND OVERLAY DESIGN)

7.7.1. General

7.7.1.1 Chart 7 deals with the Department's pre 1989 specification for drylean concrete now designated Type F DLC as well as Hot rolled asphalt and Macadam bases and equivalent materials. Further details on material types is provided at Appendix C. Higher quality materials can be converted to Type F DLC using equivalency factors given in Table 17: this is for the purposes of evaluation of composite pavements (see Section 7.9) and multi-layer pavements incorporating combinations bituminous and/or Type F and HF DLC.

7.7.1.2 When used for design, the thickness of bituminous material (including the 100mm surfacing) over the DLC should not be less than one third of the total thickness of the bound pavement materials, as described in para 6.3.7, unless special measures are taken to control reflective cracking, e.g. the use of geotextiles or crack and seat techniques, as described in Defence Estates Design & Maintenance Guide 33[52].

7.7.2. Use of Chart 7

7.7.2.1 Chart 7 has been prepared for single, dual, dual-tandem and tridem main wheel gears; see Appendix D for the definition of these gear types. The use of the Chart requires three design parameters:

(i) If Chart 7 is being used for evaluation purposes – the thickness of the surfacing, and BBM. The thickness of BBM entered into the chart should be the total thickness of surfacing plus BBM minus a standard 100 mm allowance for surfacing.

(ii) If Chart 7 is being used for design purposes, the design ACN (see Section 2.6).

(iii) The CBR of the subgrade. See Chapter 3 and Section 6.4 for details of subgrade characteristics. If subgrade improvement in accordance with Section 3.8 is to be allowed for, the increased CBR value will be the appropriate design value.

(iv) The frequency of trafficking – either Low, Medium or High. Chapter 4 defines these traffic levels in terms of Coverages by the Design Aircraft. For equating the loading effects of different aircraft see Chapter 4.

7.7.2.2 Having established the above parameters, the following sets out the procedure for use of Chart 7 for pavement evaluation:

(i) Select the appropriate BBM thickness and trace a line parallel to the curves until it intersects the vertical projection of the appropriate subgrade CBR.

(ii) From the intersection point make a horizontal projection. Read off the design ACN at the relevant trafficking level.

When designing pavements use Chart 7 in the same way as Chart 5

7.8 FLEXIBLE PAVEMENTS INCORPORATING A GRANULAR BASE AND/OR SUB-BASE DESIGN, REVERSE DESIGN AND OVERLAY DESIGN)

7.8.1. General

7.8.1.1 Chart 8 has been developed for the design or evaluation of flexible pavements incorporating granular bases and/or sub-bases.

7.8.1.2 The same design model as that described in Appendix F for new flexible pavements was used to produce Chart 8, except that the Equivalency Factors for bound base materials were omitted. Figure 31 sets out the pavement construction for use with Chart 8. To enable various other combinations of construction to be considered in the design evaluation, Table 17 sets out Equivalency Factors relating the structural value of a granular base and sub-base as given by Chart 8 to that of cement and bitumen-bound materials; see para 7.8.3.3 for the application of Equivalency Factors. The Equivalency Factors for materials are related to a number of parameters, including the quality of materials, the subgrade strength, the thickness of construction and the magnitude of the loading. Consequently they vary particularly with regard to the sub-base. They have largely developed from studies of full scale tests and are set out in Table 18 in relation to subgrade CBR. For practical design purposes the Equivalency Factors can be linearly interpolated for intermediate subgrade CBRs.

7.8.2. Bituminous Surfacing

7.8.2.1 A minimum thickness of 100mm is necessary to comply with the requirements set out in Section 6.2. Pavements designed for regular use by aircraft with an ACN greater than 50 should have a minimum surfacing thickness of 125mm if constructed on a granular base. This is to prevent early fatigue cracking being developed in the surfacing by high wheel load deflections on a granular base. The additional 25mm thickness can, in these circumstances, be subtracted from the granular base requirement.

Table 17 Equivalency Factors for Base and Sub-base Materials

Material	Structural Equivalency Factor to apply to Chart 8 requirements			
	Base	Sub-Base		
		Subgrade CBR		
		3%	10%	20%
Granular Sub-base	-	1.0	1.0	1.0
Granular Base	1.0	2.0	1.5	1.0
Marshall Asphalt Surface and Base Course	1.5	3.0	2.3	1.5
Other Bituminous Materials	1.15	2.3	1.75	1.15
Type FH DLC (see Appendix C)	1.5	3.0	2.3	1.5
Type F DLC (see Appendix C)	1.15	2.3	1.75	1.15
Cement stabilised fine grained material with a minimum compressive cube strength of 4 N/mm^2 at 7 days	-	1.0	1.0	1.0

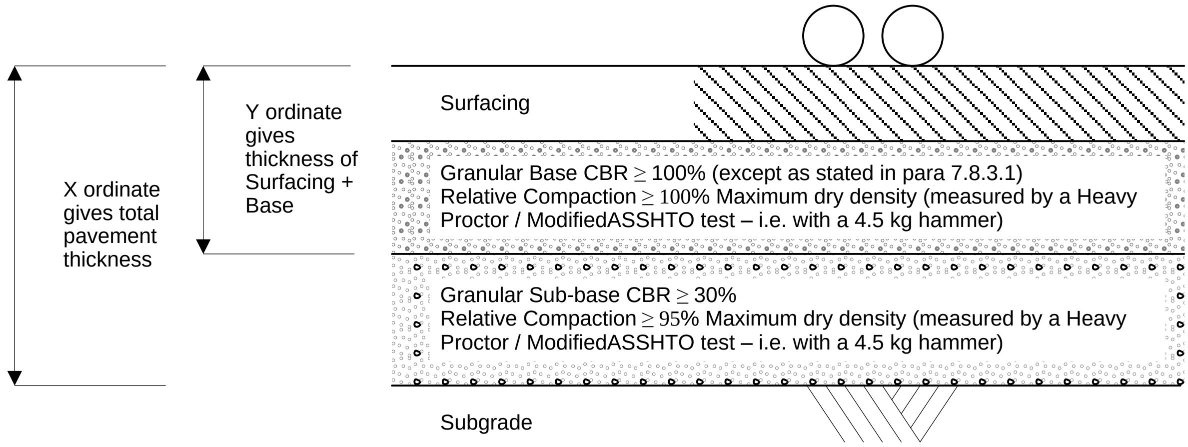


Figure 31 Pavement design and thickness requirements for Chart 8

7.8.3. Granular Base

7.8.3.1 The base thickness requirement on Chart 8 relates to the granular base material shown in Figure 31. For medium/low severity loading (i.e. nominally less than ACN 30) the granular base requirements can be reduced to CBR 80%.

7.8.3.2 In circumstances where long-term use has shown good performance of a different type of base material to those listed in Table 17, the engineer may assess it as being structurally equivalent to the standard granular base. Some base materials in overseas locations have good self-cementing properties including limestone, coral and certain laterites and may well give adequate performance particularly in respect of medium/low severity loading (i.e. nominally less than ACN 30). Field performance of cement-stabilised fine-grained material may also indicate structural equivalency as a pavement base at this level of loading. However, as explained in Section 6.3 it is unlikely that this type of construction will provide long-term load-spreading characteristics equivalent to the base construction listed in Table 17.

7.8.3.3 If, in addition to granular base and/or sub-base, the pavement contains a bound base material (see Sections 6.3 and 7.7) the material should be converted to an equivalent thickness of granular base using the equivalency factors given in Table 17. If when the first stage of the evaluation is complete there is found to be an excess of granular base, some of the excess can be converted to granular sub-base, using the equivalency factors given in Table 17; this will give a new total thickness and surfacing plus base course thickness. This process is repeated until the surfacing plus base thickness is equal to that required for the critical ACN obtained from the total thickness (see Example 7.7).

7.8.4. Granular Sub-base

7.8.4.1 The sub-base thickness requirement on Chart 8 relates to the granular sub-base material shown in Figure 31. For granular sub-bases with CBRs between 20% and 30% see para 7.8.5.2. vi. Materials less than CBR 20% should not be considered as a sub-base.

7.8.5. Use of Chart 8

7.8.5.1 Chart 8 has been prepared for the three standard main wheel gear types i.e. single, dual and dual-tandem (see Appendix D) and the constructions are linked directly to ACNs as described in Appendix F. The use of Chart 8 requires the following parameters:

(i) If Chart 8 is being used for evaluation purposes – the thickness of the surfacing, base and sub-base.

(ii) If Chart 8 is being used for design purposes, the design ACN (see Section 2.6).

(iii) The CBR of the subgrade. See Chapter 3 and Section 6.4 for details of subgrade characteristics. If subgrade improvement in accordance with Section 3.8 is to be allowed for, the increased CBR value will be the appropriate design value.

(iv) The frequency of trafficking – either Low, Medium or High. Chapter 4 defines these traffic levels in terms of Coverages by the Design Aircraft. For equating the loading effects of different aircraft see Chapter 4.

7.8.5.2 Having established the above parameters, the following sets out the procedure for use of Chart 8 for pavement evaluation:

(i) Select the appropriate total pavement thickness on the X ordinate and trace a line parallel to the unbroken line curves until it intersects the vertical projection of the appropriate subgrade CBR.

(ii) From the intersection point make a horizontal projection. Read off the design ACN at the relevant trafficking level.

(iii) Check the base thickness required for this classification. Retrace the horizontal projection to the ACN in (ii) until it again intersects the vertical projection of the appropriate subgrade CBR.

(iv) From the intersection point trace a line parallel to the *broken line* curves until it intersects the Y ordinate. The minimum combined thickness of surfacing and base required can then be read off.

(v) If the required thickness of base an surfacing is greater than the actual thickness in the pavement then the PCN will be limited by the thickness of the base. If the actual thickness of the base and surfacing is greater than the required thickness, the excess thickness can be converted to granular sub-base using the equivalency factors given in Table 17. This will give an equivalent total pavement thickness which can be re-entered on the X-Axis.

(vi) If the sub-base is between CBR 20% and 30% the base thickness requirement is greater than that determined in (iv) and can be derived from the Chart by the following method. From (ii) retrace the horizontal projection to the ACN and project it across until it intersects the X ordinate. The X ordinate then represents the combined thickness of surfacing and base required above the sub-base.

7.8.5.3 When designing pavements use Chart 8 in the same way as Chart 5; except that, having drawn a horizontal line through the design ACN to meet the subgrade CBR line, follow both the continuous and dotted curves to obtain the total thickness and surfacing plus base thickness from the X and Y axes respectively. Obtain the sub-base thickness by subtracting one from the other.

7.9 COMPOSITE PAVEMENTS – REVERSE DESIGN AND STRENGTHENING

7.9.1. General

7.9.1.1 Pavements comprised of flexible overlays on concrete slabs are termed composite. The methods for designing and evaluating these pavements depend on how they behave, and in particular the form of their failure. When viewed in this way composite pavements, other than those using crack and seat techniques to minimise reflective cracking (para 7.5.3.3), can be divided into three types:

(i) Type 1: Composite pavements with relatively thin flexible overlays.

In this case reflective cracking of structural cracks in the underlying concrete will lead to the failure mechanism described in Appendix F. From long-term performance of pavements, Equivalency Factors have been obtained to convert the thickness of flexible overlay to an equivalent concrete thickness. The pavement is then treated as rigid.

(ii) Type 2: Composite pavements with relatively thick flexible overlays.

In this case reflective cracking is delayed sufficiently for a considerable amount of structural cracking to occur in the concrete slab, leading to a transfer of the load to the subgrade and eventual failure by subgrade shear, as described in Appendix F. From long-term performance of pavements, Equivalency Factors have been obtained to convert the thickness of the concrete slab to an equivalent thickness of bound base material which is added to the thickness of the overlying flexible overlay. The pavement is then treated as flexible and Charts 7 or 8 should be used for reverse design and strengthening.

(iii) Type 3: Composite pavement with overlays which fall between Types 1 and 2 above.

In this situation the pavement cannot be defined as rigid or flexible and there is no clear cut failure criterion. The evaluated strength of these pavements is found by interpolating between rigid and flexible strengths calculated for nominal constructions conforming to Types 1 and 2 above.

7.9.1.2 Type 1 and 3 composite pavements are likely to suffer from reflective cracking from the joints in the concrete slab before structural cracking of the slab occurs. Techniques for controlling reflective cracking when designing flexible overlays of concrete slabs are described in Defence Estates Design & Maintenance Guide 33[52]

7.9.1.3 Reverse design and overlay design for composite pavements using crack and seat techniques to minimise reflective cracking is described in para 7.5.3.3.

7.9.2. Reverse Design of Composite Pavements

The following formulae should be used for evaluating composite pavements:

(i) Type 1 If $\beta \leq 0.5$

$$h_c = C_t h_e + \frac{t}{1.8}$$

(ii) Type 2 If $\beta \geq 1$

$$h_f = t + 1.8 C_t h_e + b_e$$

(iii) Type 3 If $0.5 < \beta < 1$

$$PCN = PCN_R + (PCN_F - PCN_R)(2\beta - 1)$$

where β = **t**/**h**$_e$
h$_e$ = thickness of existing concrete slab
h$_c$ = equivalent concrete thickness
h$_f$ = equivalent thickness of flexible pavement (surfacing plus bound base material)
t = thickness of bituminous surfacing
b$_e$ = thickness of existing bound base, if any
PCN$_R$ = PCN of nominal Type 1 Composite pavement with $\beta = 0.5$.
PCN$_F$ = PCN of a nominal Type 2 Composite Pavement with $\beta = 1.0$
C$_t$ = Condition Factor (see Table 18)

NB: If **C**$_t$<0.85 and β<1 then reliable performance of the pavement cannot be guaranteed and measures should be taken to increase the overlay thickness.

7.9.3. Overlay Designs for Composite Pavements

The following formulae should be used for designing flexible overlays to concrete pavements, producing a composite pavement.

(i) If the overlay thickness is less than or equal to half the concrete thickness

$$t = 1.8(h_c - C_t h_e)$$

(ii) If the overlay thickness is greater than or equal to the concrete thickness

$$t = h_f - 1.8 C_t h_e - b_e$$

(iii) If the overlay thickness is greater than half and less than one times the concrete thickness

$$\beta = \left(\frac{PCN_P - PCN_R}{PCN_F - PCN_R} + 1 \right) \times \frac{1}{2}$$

and $$t = h_e \beta$$

where **t** = bituminous overlay thickness required

h_e = thickness of existing concrete slab

b_e = thickness of existing bound base, if any

h_c = concrete slab thickness required for the design

h_f =total flexible pavement thickness (surfacing plus bound base material) required for the design

PCN_R = PCN of a nominal Type 1 composite pavement with an overlay thickness equal to half the concrete thickness

PCN_F = PCN of a nominal Type 2 composite pavement with an overlay thickness equal to the concrete thickness

PCN_P = Design ACN for the strengthened pavement

C_t = Condition Factor (see Table 18)

NB: If the value of C_t is less than 0.85 then the concrete should be converted to an equal thickness of drylean concrete, and an overlay at least equal to the concrete thickness applied to ensure reliable performance.

7.10 MULTIPLE SLAB PAVEMENTS – REVERSE DESIGN AND STRENGTHENING

7.10.1. Pavements comprised of two or more successive concrete slabs are termed multiple slab pavements. These pavements are designed or evaluated using the empirical formula developed from full scale testing by the US Army Corps of Engineers[28]. The formula can be expressed in the form

$$h_r = \sqrt[n]{C_1 h_1^{\,n} + C_2 h_2^{\,n}}$$

where **h_r** is an equivalent single slab thickness
h_1,**h_2** are the component slab thicknesses
n is a factor depending upon the bond between the layers
C_1,**C_2** are condition factors

7.10.2. Three conditions of bond are used:

(i) Fully bonded: by very careful preparation of the existing surface the two concrete layers are bonded together and behave as a monolithic slab. This form of construction should only be used if the existing surface is in good condition.

(ii) Partially bonded: the two slabs are place on top of each other with no attempt at producing a bond between layers, although some shear transfer is achieved at the interface through friction and mechanical interlock. The Defence Estates normally places a membrane between the layers; the value of **n** given below for partially bonded slabs is based on analysis of the long-term performance of this type of construction.

(iii) Unbonded: in some situations it may be necessary to use a layer of regulating material between the two slabs. When separated in this way the slabs will act more independently of each other than in the partially bonded case. Regulating courses of bituminous material or bituminous materials on thin cement-bound or drylean concrete layers less than 150mm thick can be accounted for the evaluation by converting them to an equivalent concrete thickness. This thickness should be added to the thickness of the underlying slab using the composite pavement equation shown in para 7.9.2 (I). The maximum amount of material assessed in this way should be 150mm; anything in excess of this is to be ignored. Techniques for dealing with thick drylean concrete regulating courses are given in para 7.10.3. As the thickness of a bituminous regulating course increases, the behaviour of the top slab will be increasingly governed by elastic deflections of the bituminous materials, reducing the effect of the lower slab. Therefore as much as possible of a thick regulating course should be in cement-bound materials.

7.10.3. Reverse Design of Multiple Slab Pavements

The following formulae should be used for evaluating multiple slab pavements.

(i) $$h_r = \sqrt[n]{C_1 h_1^{\,n} + C_2 h_2^n}$$

(ii) for multiple slab construction containing a drylean concrete regulating course greater than 150mm thick.

$$h_r = \sqrt[1.6]{C_1 h_1^{1.6} + \left(\frac{h_d}{1.8}\right)^{1.6} + C_2 h_2^{1.6}}$$

where **h**$_r$ = equivalent slab thickness

h_1, h_2 = top and bottom slab thickness

C$_1$, **C**$_2$ are the condition factors (see Table 18)

NB **C**$_1$ will only be required if the evaluation is being carried out as part of an overlay design (see para 7.3.1.3)

n = 1.0 for a fully bonded pavement

n = 1.6 for a partially bonded pavement

n = 2.0 for an unbonded pavement

h$_d$ = thickness of drylean concrete.

7.10.4. Overlay Design for Multiple Slab Pavements

The following formulae should be used for the design of an overslab to an existing concrete pavement.

(i) $$h_o = \sqrt[n]{h_c^n - C_2 h_e^n}$$

(ii) for a multiple slab construction containing a drylean concrete regulating course greater than 150mm thick

$$h_o = \sqrt[1.6]{h_c^{1.6} - \left(\frac{h_d}{1.8}\right)^{1.6} - C_2 h_e^{1.6}}$$

where **h**$_0$ = thickness of overslab required

h$_c$ = thickness of a single concrete slab required for the design

h$_e$ = thickness of the existing concrete slab

h$_d$ = thickness of a drylean concrete regulating course

C$_2$ = condition factor (see Table 18)

n = 1.0 for a fully bonded pavement

n = 1.6 for a partially bonded pavement

n = 2.0 for an unbonded pavement.

7.10.5. If the top slab is thicker than the underlying one, the formulae given above may be conservative and evaluations or overlay designs should be checked by assuming k = 150MN/m^2/m on the surface of the bottom slab and then using Chart 8.

Table 18 Condition Factors for Concrete Slabs

Conditions of concrete bays in the wheel track area	Condition Factors	
	C_t	C_i
a) No more than a few cracks	1.0	1.0
b) 30%-50% contain halving, quartering or delta cracks	0.85	0.75
c) Virtually all cracked with 30%-50% containing corner cracks or having cracked into 5 or more pieces	DLC N/A	DLC N/A
d) Many with multiple cracking and some deformation	Gbc N/A	Gbc N/A

Abbreviations: DLC = equate to drylean concrete
Gbc = equate to granular base course
N/A = formulae in Section 7.9 are not applicable
NB: i = 1, 2 etc see para 7.10.1

7.11 EVALUATION OF HANGAR FLOORS

7.11.1.1The rigid design model and the Charts include a factor for the effects of temperature-induced stresses. However, in hangar floors, the full effects of the temperature range are not often experienced, and thus evaluation based on Charts 1, 2, 3, 4 or 6 will underestimate the bearing strength of the floors. For existing hangar floors where the normal daily temperature range is 50% (or less) of that for the external pavements, the PCN obtained from the charts may be factored by:

(i) 1.5 for Low Frequency Trafficking
(ii) 1.2 for Medium Frequency Trafficking

if the aircraft use suggests that the evaluation without these factors is over-conservative.

EVALUATION EXAMPLES

Example 7.1 CONCRETE SLAB ON A ROLLED DRYLEAN CONCRETE BASE

Guide Reference

1. CONSTRUCTION:

   275mm Pavement Quality Concrete
   150mm Rolled Drylean Concrete.

2. USE: The runway end on a busy military airfield.

   MAIN USER AIRCRAFT: ACN – 16
   Main Wheel Gear – Single
   Tyre Pressure – 1.3 MPa
   Pass-to-Coverage Ratio – 8 (Para 4.9.2, Table 7)

   MOVEMENTS: 75 Departures a day for about 350 days of the year.

   DESIGN LIFE: 30 years. (Para 4.7.3)

   FREQUENCY OF TRAFFICKING: Coverages = $\frac{75x30x350}{8} = 98438$

   Take Medium Frequency Trafficking (Para 4.6.2, Table 5)

3. SUBGRADE: A clay with k = 30MN/m^2/m, obtained from in situ testing.

4. MATERIAL QUALITY: Concrete produced to Defence Estates' Specification. Assume the concrete flexural strength = 4.5N/mm^2 at 28 days (Para 5.2.3)

5. PAVEMENT TYPE: Rigid.

6. EVALUATION: PCN 33. (Chart 1)

   k = 30 MN/m^2/m is not a standard subgrade reporting value and it may therefore be necessary to correct the PCN to an appropriate value for k.

   In this case the reporting category is Low (k = 40MN/m^2/m) but the ACN for a single main wheel gear does not change with the subgrade support. (NB. Examination of published ACN data may show a variation of ACN with subgrade support for single wheels, but this is due to the difference between the actual tyre pressure and the standard ACN-PCN tyre pressure). (Para 2.4.2, Table 1)

7. CLASSIFICATION: (Para 2.4.2)

   a) Subgrade Category: Low (C).

   b) PCN: 33.

   c) Pavement Type: Rigid (R).

   d) Tyre Pressure Limitations: No limitations on a concrete surface (W).

   e) PCN 33/R/C/W/T.

Example 7.2 MULTIPLE CONCRETE SLAB CONSTRUCTION

Guide Reference

1. CONSTRUCTION:

200mm Pavement Quality Concrete} Twin slab construction
200mm Pavement Quality Concrete} with a building paper separating membrane.
150mm average Rolled Drylean Concrete
150mm Concrete
300mm Granular Fill.

2. USE: A parallel taxiway on a busy military airfield with an expected use of 50,000 Coverages by dual and dual-tandem aircraft during a 30 year life i.e. Medium Frequency Trafficking.

3. SUBGRADE: Clay. k = 30 MN/m^2/m obtained from in situ testing.

4. MATERIAL QUALITY: The construction records for the twin slab show that the concrete strength was low. The 150mm concrete slab is wartime construction. Cores show that the mean flexural strength of the top slab is 4.7 N/mm^2. There are no visable cracks in the pavement surface.

5. PAVEMENT TYPE: Rigid.

6. EVALUATION:

Para 7.4.2.13 — (i) Convert the twin slab construction to an equivalent single slab.

Para 7.10.4

$$h_r = \sqrt[n]{C_1 h_1^n + C_2 h_2^n}$$

n = 1.6 for a twin slab with a separating membrane.
C$_1$ = 1.0 as the surface layer is considered as new.
C$_2$ = 1.0 for a slab in good condition.

Table 18

$$\sqrt[1.6]{1.0 \times 200^{1.6} + 1.0 \times 200^{1.6}} = 308\text{mm}$$

(ii) Convert the equivalent single top slab, DLC regulating course and bottom slab to an equivalent single thickness pavement.

From Para 7.10.4

$$h_r = \sqrt[1.6]{C_1 h_1^{1.6} + \left(\frac{h_d}{1.8}\right)^{1.6} + C_2 h_2^{1.6}}$$

It is conservative to assume that the wartime concrete slab has suffered some degree of cracking; take C$_2$ = 0.75.

Table 18

$$\sqrt[1.6]{1.0 \times 308^{1.6} + \left(\frac{150}{1.8}\right)^{1.6} + 0.75 \times 150^{1.6}} = 375\text{mm}$$

Para 7.4.2.2 — (iii) Calculate an effective k on the granular sub-base.

Para 3.8.4, Figure 10 — 300mm granular sub-base on k = 30MN/m^2/m gives 48MN/m^2/m.

(iv) Equivalent construction 375mm PQC on k = 48MN/m^2/m.

Chart 6 — (v) PCN 41.

Para 2.4.2 Table 2.1
Para 7.4.3

The standard subgrade category is Low (k = 40MN/m^2/m). Examination of a number of aircraft with an ACN close to 40 on a Rigid Low Subgrade (see below) shows that the change in ACN between k = 48MN/m^2/m and k = 40MN/m^2/m is less than 1.

Aircraft type	All Up Mass (kg)	RIGID PAVEMENT SUBGRADES - $MN/m^2/m$			
		High 150	Medium 80	Low 40	Ultra Low 20
		ACN			
A318	68,400	36	38.4	40.6	42.5
B737-500	60,800	36.4	38.4	40.2	41.7
B757-200	116,100	30.7	36.8	43.4	49.3
C130H	79,379	35.8	38.6	41.6	44.5
Embraer 190	47,790	41.8	42.2	42.6	42.9
Nimrod MR Mk 2	83,461	31.9	36.2	40.4	44.1

Para 2.4.2

7. CLASSIFICATION

a) Subgrade Category: Low (C).

b) PCN: 41.

c) Pavement Type: Rigid (R).

d) Tyre Pressure Limitations: No limitations on a concrete surface (W).

e) PCN 41/R/C/W/T.

Example 7.3 THIN FLEXIBLE OVERLAY ON A CONCRETE SLAB ON A THIN ROLLED DRYLEAN CONCRETE BASE

Guide Reference

1. CONSTRUCTION:

   40mm Hot Rolled Asphalt Surface Course
   60mm Macadam Binder Course
   300mm Pavement Quality Concrete
   100mm Rolled Drylean Concrete.

2. USE: A taxiway for dual-tandem aircraft. Expected use is less than 10,000 Coverages in a 20-year life, i.e. Low Frequency Trafficking.

3. SUBGRADE: Sand. k = 60MN/m^2/m from in situ testing.

4. MATERIAL QUALITY: Cores in the Pavement Quality Concrete at 20 years give a mean compressive strength of 56N/mm^2, and show a crushed rock aggregate. A conservative estimate of the flexural strength is 56/10 = 5.6N/mm^2. Take a 4.5N/mm^2 mean flexural strength at 28 days to allow for gain in strength with age. Concrete bay joints have reflected through the surfacing, but there is no sign of structural cracking.

Para 7.4.2.10
Para 7.9.2

5. PAVEMENT TYPE: Composite pavement where the ratio of the flexible overlay to the concrete thickness is less than 0.5 i.e Type 1.

6. EVALUATION:

Para 7.9.2

(i) $$h_c = C_t h_e + \frac{t}{1.8}$$

$$h_c = 1 \times 300 + \frac{100}{1.8} = 355$$

Chart 3

(ii) 355mm PQC on subgrade k = 60 MN/m^2/m requires 150mm DLC.

Para 7.4.2.4

(iii) Convert some of the PQC slab to DLC to make up the deficiency.

$$\frac{50}{3} = 17\text{mm}$$

Modified PQC thickness: 355-17 = 338mm.

(iv) Equivalent construction is 340mm PQC
150mm DLC.

Chart 3

PCN 70.

Para 2.4.2, Table 1

(v) Subgrade Category – Medium (k = 80 MN/m^2/m). For aircraft with dual-tandem main wheel gears the ACN varied considerably with the sub-grade strength. Consider a range of aircraft with an ACN close to 70 on k = 60.

Appendix B

Aircraft	k80 (1)	k60 (2)	k40 (3)	$\frac{(1)}{(2)}$
B747-200	56	61	67	0.92
Concorde	71	76	82	0.93
DC10-30	53	58	64	0.91
L1011-500	59	65	72	0.91
				0.92

The ACN on k = 80 is approximately 0.92 times the ACN on k = 60

70 x 0.92 = 64

Para 2.4.2

7. CLASSIFICATION

Table 2.1

a) Subgrade Category: Medium (B).

b) PCN: 64.

c) Pavement Type: Rigid (R).

Para 6.2.4, Table 13

d) Tyre Pressure Limitations: X.

e) PCN 64/R/B/X/T.

Example 7.4 ASPHALT ON TYPE F DRYLEAN CONCRETE

Guide Reference

1. CONSTRUCTION:

   40mm Marshall Asphalt Surface Course
   60mm Marshall Asphalt Binder Course
   37mm Hot Rolled Asphalt Surface Course
   63mm Macadam Base Course
   450mm Drylean Concrete.

2. USE: The main runway on a busy military airfleld, with the majority of movements by aircraft with single main wheel gears. 200,000 Coverages expected in a 20-year life, i.e. High Frequency Trafficking.

3. SUBGRADE: Clay. CBR 3%.

4. MATERIAL QUALITY: All materials to Defence Estates' Specification or one of its predecessors. Cores show the Drylean Concrete has a compressive strength of 11 N/mm^2; therefore take as Type F.

5. PAVEMENT TYPE: FLEXIBLE.

6. EVALUATION:

   (i) Correct for High Frequency Trafficking (and use the Medium Frequency Trafficking line on the Chart).

Para 6.6.1

$$\text{Equivalent Thickness} = \frac{TotalThickness}{1.08} = \frac{650}{1.08} = 600$$

   (ii) Equivalent Construction 100mm Surfacing
   500mm Bound Base Material.

Chart 7

PCN 30.

CBR 3% is a standard subgrade category, so no correction is required to the PCN.

Para 2.4.2, Table 1

7. CLASSIFICATION:

   a) Subgrade Category: Ultra Low (D).

   b) PCN 30.

   c) Pavement Type: Flexible (F).

Para 6.2.4, Table 13

   d) Tyre Pressure Limitations: No limitations for Marshall Asphalt (W).

   e) PCN 30/F/D/W/T.

Example 7.5 FLEXIBLE OVERLAY ON A THIN CONCRETE SLAB

Guide Reference

1. CONSTRUCTION:
   40mm Hot Rolled Asphalt Surface Course
   60mm Macadam Base Course
   40mm Hot Rolled Asphalt Surface Course
   60mm Macadam Base Course
   25mm Asphalt Surface Course
   65mm Tarmacadam
   20mm Sand Asphalt.
   150mm Concrete

2. USE: Main taxiway of a provincial airport with an expected use of 10,000 Coverages by dual-tandem aircraft in a 20-year life, i.e. Low Frequency Trafficking.

3. SUBGRADE: A dense silty sand. CBR 10%.

4. MATERIAL QUALITY: Cores show that all the materials are in good condition.

   Inspection Reports produced before the first Hot Rolled Asphalt/Macadam Base Course overlay was placed indicate that considerable reflection cracking was present showing longitudinal, quartering and corner cracking in the underlying concrete.

Para 7.4.2.11
Para 7.9.2

5. TYPE: Composite pavement where the ratio of the flexible overlay to the concrete thickness is greater than 1 i.e. Type 2.

6. EVALUATION

Para 7.9.2

   (i) $h_f = t + 1.8 C_t h_e + b_e$

Table 18

   Take $\mathbf{C}_t$ = 0.85 because of evidence of structural cracking in the concrete.

   $$h_f = 310 + 1.8 \times 0.85 \times 150 + 0 = 540\text{mm}$$

   (ii) Equivalent Construction: 100mm Surfacing
   450mm Bound Base Material.

Chart 7

   PCN 68.

   CBR 10% is a standard subgrade category.

Para 2.4.2

7. CLASSIFICATION:

Table 1

   a) Subgrade Category: Medium (B).

   b) PCN 68.

   c) Pavement Type: Flexible (F).

Para 6.2.4 Table 13

   d) Tyre Pressure Limitations: X.

   e) PCN 68/F/B/X/T.

Example 7.6 ASPHALT SURFACING ON A GRANULAR BASE AND SUB-BASE

Guide Reference

1. CONSTRUCTION:

   40mm Marshall Asphalt Surface Course
   60mm Marshall Asphalt Base Course
   250mm Granular Base
   650mm Granular Sub-base.

2. USE: Taxiway on a provincial airport. Expected use 10,000 Coverages by dual aircraft in a 20-year life, i.e. Low Frequency Trafficking.

3. SUBGRADE: Clay. CBR 4%.

4. MATERIAL QUALITY: Site investigation confirms that the granular materials have a grading and in situ density compatible with Defence Estates' Specifications for base and sub-base materials. There are no signs of rutting.

5. TYPE: FLEXIBLE.

Section 7.7, Chart 8

6. EVALUATION:

(i) Evaluate PCN from the total pavement thickness.
Total Pavement Thickness = 1000mm.
From X-Axis PCN = 48.

(ii) Check thickness of base plus surfacing.
From Y-Axis, PCN48 on CBR 4% requires 275mm of base plus surfacing, which compares with 350mm in the actual pavement.

(iii) Convert excess granular base to granular sub-base.
Estimate base + surfacing requirement as 275mm.
Excess Granular Base = $(250-(275-100)=75\text{mm}$.

Para 7.8.5.2v
Table 17

Equivalency Factor (by interpolation between the published values)

$$= 2.0 - \frac{(2.0-1.5)}{7} \times 1 = 1.93$$

Equivalent Thickness of Granular Sub-Base = $75 \times 1.93 = 145\text{mm}$.

Equivalent Construction
100mm Surfacing
175mm Granular Base
800mm Granular Sub-Base

Chart 8
Para 7.8.2.1

(iv) If we round up to the nearest 100mm, i.e. total thickness of 1100mm, Chart 6 indicates a PCN greater than 50 and a base plus surfacing requirement of 275mm. However for a PCN greater than 50, 125mm of surfacing is required, therefore the PCN of the actual pavement is restricted to 50.

Para 2.4.2, Table 1

(v) The Standard Subgrade Category is Ultra Low (CBR 3%).
Consider a range of aircraft:

Aircraft	ACN CBR 6% (1)	CBR 4% (2)	CBR 3% (3)	$\frac{(3)}{(2)}$
A321-200	46.8	50.7	52.6	1.038
B727-100	49.2	52.6	54.3	1.032
B737-900	50.4	53.7	55.3	1.030
MD90-30	49.5	51.6	52.6	1.020
				1.03

The PCN on CBR 3% is approximately 1.04 times the PCN on CBR 4%.
50 x 1.03 = 51.

Para 2.4.2

7. CLASSIFICATION:

a) Subgrade Category: Ultra Low (D).

b) PCN: 51.

c) Pavement Type: Flexible (F).

Para 6.2.4, Table 13

d) Tyre Pressure Limitations: W.

e) PCN 51/F/D/W/T.

Example 7.7 MIXED BOUND AND UNBOUND FLEXIBLE CONSTRUCTION (INCLUDING CAPPING LAYER)

Guide Reference

1. CONSTRUCTION:
   20mm Friction Course
   40mm Marshall Asphalt Surface Course
   60mm Marshall Asphalt Base Course
   650mm Type F Rolled Drylean Concrete
   300mm Granular Sub-base
   550mm Capping Layer.

2. AIRCRAFT USE: Main runway for an international airport. Expected use is equivalent to 100,000 Coverages by Boeing 767-200 in 20 years i.e. Medium Frequency Trafficking.

3. SUBGRADE: Silty Clay CBR 2%.

4. MATERIAL QUALITY: Asphalt, Rolled Drylean Concrete and Granular Sub-base are compatible with Defence Estates' Specification. The capping layer is a granular material with a minimum CBR of 15%.

5. PAVEMENT TYPE: FLEXIBLE

6. EVALUATION:

Para 6.2.1 — (i) The Friction Course is ignored.

Figure 7 Dual-Tandems — Determine Equivalency Factor for the Capping Layer. 1.3

Para 7.4.2.2 — (ii)

			1st Estimate	2nd Estimate			
		Determine Design ACN					
	a)	CBR on Capping	Estimate 3%	3.5	3	3.5	
Para 7.4.2.9 Para 7.8.1.2, Table 17	b)	Calculate Equivalency Factor of BBM to Granular Base.	1.15				
		Calculate Equivalency Factor of Granular Base Course to Granular Sub-base	2	((2-1.5)/(3-10))*(3.5-3)+2=1.96	2	1.96	
Para 7.8.2.1	d)	Surfacing Requirement.	100				
Para 7.8.3.3	e)	Convert BBM to Granular Base.	650x1.15=750				
		Convert Excess Granular Base Course to Granular Sub-base	0	(750-275)*1.96=930	(750-375)*2=750	(750-325)*1.96=830	
	f)	Calculate Total Thickness (X).	100+750+0+300=1150	100+275+930+300	100+375+750+300	100+325+830+	

				=1605	=1525	300= 1555	
Chart 8 X-line	g)	Determine Design ACN	32	71	55	67	
Chart 8 Y-line	h)	Determine required Surfacing and Base Thickness required.	275	375	325		
		Check CBR on Capping Layer					
		Calculate Equivalent Thickness of Capping Layer as Granular Sub-base	550/1.3=423				
		Calculate t^2/ACN for the Capping Layer	423^2/32=5591	423^2/ 71=2520	423^2/55=3253		
		Determine the CBR on the Capping Layer	3.5	3	3.5		
	i)	Estimate thickness of Surfacing plus Base required	350				
	j)	Convert Excess Base to Sub-base.	1000				

Para 7.4.2.9 Para 7.8.1.2, Table 17	b) Calculate Equivalency Factor of BBM to Granular Base.	1.15	
Para 7.8.1.2, Table 17	c) Calculate Equivalency Factor of Granular Base to Sub-base.	2	
Para 7.8.2.1	d) Surfacing Requirement.	100	
Para 7.8.3.3	e) Convert BBM to Granular Base.	750	
		1st Estimate	2nd Estimate
	f) Calculate Total Thickness.	1150	1650
Chart 8 X-line	g) Determine PCN.	37	70
Chart 8 Y-line	h) Determine Surfacing and Base Thickness required.	275	350
	i) Estimate thickness of Surfacing plus Base required.	350	
	j) Convert Excess Base to Sub-base.	1000	
	k) Return to step (f).		

(iii)Check the CBR on the Capping Layer.

Calculate Equivalent Thickness of Capping Layer as Granular Sub-base. 423

Figure 9	Calculate t^2/ACN for the Capping Layer.	2557
	Determine the CBR on the Capping Layer.	3%

Para 2.4.2

7. CLASSIFICATION:

a) Subgrade Category: Ultra Low (D).

b) PCN: 70.

c) Pavement Type: Flexible (F).

Para 6.2.4, Table 13

d) Tyre Pressure Limitations: W.

e) PCN 70/F/D/W/T

(This pavement is likely to suffer from early and extensive reflection cracking form shrinkage cracks in the DLC base, and is not recommended for a new pavement. Further details are given in Defence Estates Design & Maintenance Guide 33[52])

Example 7.8	FLEXIBLE OVERLAY ON CONCRETE WITH $0.5<\beta<1$
Guide Reference	1. CONSTRUCTION: 40mm Hot Rolled Asphalt 60mm Macadam Base Course 37mm Hot Rolled Asphalt 63mm Macadam Base Course 225mm Pavement Quality Concrete.
	2. USE: A Taxiway. The use is expected to be less than 10,000 Coverages by dual-tandem aircraft in a 20-year life, i.e. Low Frequency Trafficking.
Figure 32	3. SUBGRADE: A silty sand. CBR 10%/k = 50 MN/m^2/m.
	4. MATERIAL QUALITY: All materials complied with Defence Estates' Specification. Concrete strength is 5.3 N/mm^2.
Para 7.4.2.10 Para 7.9.2	5. PAVEMENT TYPE: Composite pavement where the ratio of the flexible overlay to concrete thickness lies between 0.5 and 1 i.e. Type 3.
Para 7.9.2	6. EVALUATION:
	(i) Evaluate an imaginary pavement with β =1
	Construction is 225mm Asphalt 225mm Concrete.
Para 7.9.2(ii)	$\mathbf{h_f} = t + 1.8\,\mathbf{C_t h_e} + \mathbf{b_e}$ $(\mathbf{b_e = 0})$ $= 225 + 1.8 \times 225 = 630$
	Equivalent Construction 100mm Surfacing 525mm Bound Base Material.
Chart 7	$PCN_F = 95$
	(ii) Evaluate an imaginary pavement with β = 0.5
	Construction is 112mm Asphalt 225mm Concrete.
Para 7.9.2(i)	$\mathbf{h_c} = \mathbf{C_t h_e} + \frac{t}{1.8}$ $= 1.0 \times 225 + 112/1.8 = 287$
	Equivalent Construction = 285mm PQC
Chart 6	$PCN_{(R)} = 30$
	(iii)Evaluate the final PCN of the actual construction.
Para 7.9.2	$\beta = \frac{200}{225} = 0.89$
Para 7.9.2(iii)	$PCN = PCN_{(R)} + (PCN_f - PCN_{(R)})\,(2\beta-1)$ $= 30 + (95-30) \times 0.78$ $= 81.$

Para 2.4.2

7. CLASSIFICATION:

a) Subgrade Category: Medium (B).
b) PCN: 81.
c) Pavement Type: Flexible (F). — Para 2.4.2
d) Tyre Pressure Limitations: X. — Para 6.2.4, Table 13
e) PCN 81 /F/B/X/T

(In determining the final overlay requirement the critical factor for the overlay requirement may be reflection cracking rather than structural strength. Further details are given in Defence Estates Design & Maintenance Guide 33[52])

OVERLAY DESIGN EXAMPLES

Example 7.9 FLEXIBLE OVERLAY ON AN EXISTING PAVEMENT

Guide Reference

1. REQUIREMENT: A taxiway currently carrying Boeing 727-200 and lighter aircraft is to be strengthened to take Boeing 747-100.

2. AIRCRAFT DATA:

Appendix B
Para 4.9.2, Table 6

a) ACN, Undercarriage Type and Pass-to Coverage Ratio.

Aircraft Type	FLEXIBLE PAVEMENT SUBGRADES				Main Wheel Gear	Pass-to-Coverage Ratio
	High	Medium	Low	Ultra Low		
747-100	44	48	58	78	D-T	1.6
727-200	40	42	48	53	Dual	3.2

3. AIRCRAFT USE: The existing pavement is 15 years old and had had an average use of 1200 departures per year by Boeing 727-200.

Para 4.9.1

Use to date = $\frac{15x1200}{3.2} = 5625$ coverages.

The expected future use is 500 departures a year by Boeing 747-100.

Para 4.7.3

4. DESIGN LIFE: 20 YEARS.

5. FREQUENCY OF TRAFFICKING:

Para 4.6.2, Table 5

a) For the evaluation of the existing pavement use Low Frequency Trafficking based on the use to date.

Para 4.7.3

b) For future use the optional coverages in a 20 year life are

$$\frac{20x500}{1.6} = 6250$$

Para 4.6.2, Table 5

i.e. Low Frequency Trafficking

6. EXISTING CONSTRUCTION:
40mm Hot Rolled Asphalt Surface Course
60mm Macadam Base Course
425mm Type F Rolled Drylean Concrete.

7. SUBGRADE: CBR 6%.

8. MATERIAL QUALITY: The pavement materials are compatible with Defence Estates' Specification. Some rutting is present in small areas.

9. EVALUATION OF THE EXISTING PAVEMENT:
100mm Surfacing
425mm Bound Base Material.

Chart 7

PCN 50.

Para 7.3.2.2

The aircraft use to date equates to 5625 coverages by Boeing 727-200, which have an ACN of 48 on CBR 6%. The Equivalent Coverages on the actual pavement can be calculated as follows:

Figure 13

	ACN/ PCN	ACN Ratio	FMTF	Modified FMTF	Coverages
Pavement	50				
Boeing 727-200	48	0.96	0.92	0.88	4400

Since the evaluation is for Low Frequency Trafficking – nominally 10,000 coverages – the remaining life is 10,000-4,400 = 5,600 Coverages. if the pavement is to be overlaid to give a further 20 year life for a heavier aircraft the existing construction should be re-evaluated to find a lighter load which will allow a new 10,000 Coverage life. Therefore find a PCN which will allow a further 10,000 Coverages with an equivalent damaging effect to 5,600 Coverages at ACN 50.

Mixed Traffic Factor 10,000 Coverages = 1.0
Mixed Traffic Factor 5,600 Coverages = 0.91.

ACN Ratio = $\dfrac{0.91}{1.0} = 0.91$

PCN = 50 x 0.91 = 45

Chart 7

Equivalent Construction is 100mm Surfacing
400mm Bound Base Material

10. DESIGN REQUIREMENT FOR NEW LOADING

The design requirement is for Low Frequency Trafficking by Boeing 747-100.

(see also Example 4.2)

The ACN of the aircraft on a Flexible Low Subgrade is 58.

Chart 7

100mm Surfacing
500mm Bound Base Material

11. OVERLAY DESIGN

(i) The existing pavement is equivalent to a total of 500mm.

(ii) The new design requirement is for a total of 600mm.

Overlay requirement = 600-500 = 100mm.

Overlay with 100mm asphalt or equivalent materials.

(In determining the final overlay requirement the critical factor for the overlay requirement may be reflection cracking rather than structural strength. Further details are given in Defence Estates Design & Maintenance Guide 33[52])

Example 7.10 RIGID OVERSLAB OF AN EXISTING RIGID PAVEMENT

Guide Reference

1. REQUIREMENT: An existing hardstanding is to be uprated to take dual wheel gear short/medium range transport aircraft, of which McDonnell-Douglas DC9-51 will be the most severe loading case.

2. AIRCRAFT DATA:

Appendix B

a) ACN

Aircraft Type	RIGID PAVEMENT SUBGRADES			
	High	Medium	Low	Ultra Low
DC 9-51	35	37	39	40

Appendix B

b) Main Wheel Gear: Dual

Para 4.9.2, Table 6

c) Pass-to-Coverage Ratio: 3.2.

3. AIRCRAFT USE: The expected use is less than 10,000 Coverages by DC9-51 in 30 years, i.e. Low Frequency Trafficking.

Para 4.7.3

4. DESIGN LIFE: 30 years.

5. EXISTING CONSTRUCTION:
175mm Pavement Quality Concrete
300mm Granular Sub-base.

6. SUBGRADE: k = 40 $MN/m^{2/}/m$.

7. MATERIAL QUALITY: The concrete is of good quality and from the available information it is thought appropriate to use the 5.3 N/mm^2 line on Chart 6. A few of the existing bays have one or more corner cracks and about 30% of the bays have halved. There are no signs of differential settlement or mud-pumping.

8. EVALUATION: Only an estimate of the effect of the granular sub-base is needed.

Para 3.8.4. Figure 10

300mm of granular sub-base on k = 40 $MN/m^2/m$ gives an effective K of 60$MN/m^2/m$.

9. DESIGN REQUIREMENT FOR NEW LOADING:

The design requirement is for Low Frequency Trafficking by DC9-51. Design ACN = 38 (on k = 60$MN/m^2/m$).

Chart 6

325mm Pavement Quality Concrete

10. OVERLAY DESIGN

Para 7.10.4

$$\mathbf{h}_0 = \sqrt[n]{\mathbf{h}_c{}^n - \mathbf{C}_2\mathbf{h}_e{}^n}$$

The overslab will be laid on a polythene separating membrane, directly on the underlying slab; therefore n = 1.6.

Table 18

From the existing degree of cracking $\mathbf{C}_2$ = 0.75.

$$\mathbf{h}_0 = \sqrt[1.6]{325^{1.6} - 0.75x175^{1.6}}$$

= 265mm PQC

Overlay with 275mm PQC.

Para 7.10.4

NB.Chart 6 shows that if the existing construction is considered as a sub-base giving an effective k of 150 $MN/m^2/m$, the overlay requirement is also 275mm. If the existing slab was thinner, the method used above to calculate the overslab thickness becomes pessimistic and it is more economic to consider the existing construction as a good sub-base.

8 Overload and High Tyre Pressure Operations

8.1 OVERLOAD OPERATIONS

8.1.1. Individual aerodrome authorities are generally free to decide on their own criteria for permitting overload operations as long as pavements remain safe for use by aircraft. Unless severely overloaded, (e.g. an aircraft with an ACN four times greater than the PCN) it is most unlikely that a pavement will suddenly or catastrophically fail. Nevertheless regular overload can substantially reduce the design life of a pavement, resulting in high rehabilitation costs and the inconvenience of a main runway or taxiway out of action. The limiting criteria for overload must be somewhat arbitrary, representing a reasonable balance between operational flexibility and the need to avoid undue damage to pavements. On that basis the following guidance has been developed:

(i) A 10% difference in ACN over PCN involves an increase in pavement working stresses which is generally considered acceptable provided the following conditions are satisfied.

a. The pavement is more than 12 months old.
b. The pavement is not already showing signs of structural distress.
c. Overload operations do not exceed 5% of the annual departures and are spread throughout the year.
The 5% must be calculated from the number of departures of aircraft with ACNs at or near the PCN of the pavement (i.e. 5% of the 'design traffic'). Otherwise if there is a high frequency of use by light aircraft which are well below the PCN, 5% of the *total* movements could represent a substantial proportion of the actual coverage level for the pavement (see para 4.6.2) and lead to an unacceptable rate of deterioration.

The effect of maintaining overload operations at this level and frequency cannot be accurately predicted owing to the number of variables; for example, the type of construction (i.e. rigid and flexible), its condition, the type of aircraft (e.g. pass-to-coverage ratios) and the environmental factors at the time. As an approximate guide the standard rigid and flexible pavement design models were used to establish average results; these gave a 5-15% reduction in the remaining design life.

(ii) Overload operations representing a difference in ACN over PCN from 10% to 25% justify regular inspections of the pavements by a competent person in addition to satisfying the criteria for 10% overload. Overload operations should stop as soon as distress becomes evident; the higher loading should not be reimposed until appropriate pavement strengthening work has been completed.

As for the 10% overload case the standard rigid and flexible design models were used to assess the implications of maintaining 25% overload operations at a frequency of 5% of the 'design traffic' The results varied from 25-75% reduction in design life depending on the pavement type and aircraft type. Therefore overload operations at this level and frequency should only be short-term.

(iii) Overload operations representing a difference in ACN over PCN from 25% to 50% should only be permitted very occasionally. They call for scrutiny of available pavement construction records and test data and a thorough pavement inspection by a pavement engineer before and on completion of the movement to assess any signs of pavement distress.

(iv) Overload operations representing a difference in ACN over PCN of more than 50% should only be undertaken in an emergency.

8.2 HIGH TYRE PRESSURE OPERATIONS

8.2.1. For practical design purposes the tyre pressure produces the intensity of the load on the pavement. The primary consideration for excess tyre pressure operations is the risk of undue damage to the surfacing. The consequences of pavement damage as a result of overstressing of the surfacing layers are likely to be less serious than a deep seated structural failure. Nevertheless an engineer must carefully weight the problems of carrying out maintenance work in the event of damage before allowing occasional excess tyre pressure operations for the sake of maintaining operational flexibility. The following notes are for guidance:

(i) Occasional movements by aircraft with tyre pressures over the maximum authorised for unrestricted use (see para **Error! Reference source not found.**) of the pavements re unlikely to have significant effect on the performance of the pavement except in circumstances described in (iii). The factors which affect surface stability make it inappropriate to lay down rules.

(ii) Concrete pavements are not subject to surface indentation by high tyre pressure aircraft.

(iii) Bituminous surfacing of other than high stablility Marshall asphalt or with less than 100mm of Marshall asphalt are liable to indentation by high pressure tyres. The amount of indentation depends on the following factors:

  a. The stability of blacktop surfacings is temperature dependant and therefore they are more liable to identation by high pressure tyres on hot days. This is particularly the case for tar-bound surfacings (e.g. dense tar or tar macadam).
  b. Although ready for use within hours of laying, bituminous surfacings continue to harden for some months. This depends on the type of mix and the climatic conditions. The full stability of surfacing is not realised for several months after laying.
  c. Due to creep, indentation is more likely to occur on a bituminous surface when aircraft are parked on it. Metal plates can be used to spread the load beneath the tyres of parked aircraft, they will protect a low stability black-top surfacing.
  d. Shallow pavements comprising less than 100mm of bituminous surfacing on low-grade granular bases (i.e. CBR <80%) are liable to structural damage by high tyre pressure aircraft, particularly where the aircraft are parked. In practice this situation will effectively represent a combination of overload and excess tyre pressure and will therefore need to be carefully considered.

(iv) Use of pavements by aircraft with tyre pressures three categories above the designated PCN should only be considered in an emergency.

9 Stopways, Shoulders and Blast Pads

9.1 GENERAL

9.1.1. Whether stopways and shoulders should be provided is explained in the ICAO publications Annex 14[13] and the Aerodrome Design Manual Parts 1 and 2[11]. Stopways and shoulders should be strong enough to support any aircraft which the runway is designed for, without introducing structural damage to the aircraft. They should also be able to support rescue and fire fighting vehicles. The definitions of strength and serviceability are open to some interpretation; the following sets out the design concept and method Defence Estates uses for establishing Stopways and shoulder construction

9.2 STOPWAYS

9.2.1. A stopway provides a safe 'run out' for an aircraft if take off is aborted. It can be included as part of the Accelerate Stop Distance Available (ASDA) which is one of the four declared runway distances in Annex 14. Note: this distance is referred to as the Emergency Distance Available in the AIP).[14]

9.2.2. A stopway surface can be unpaved or paved. A low-cost unpaved stopway could be designed in accordance with the procedure set out in Section 9.3 for shoulder construction. However, such a stopway would probably require some regrading and reconstruction after each pass of an aircraft. It would also result in a surface with a variable ridability and braking characteristics. The degree of variability will depend on the prevailing moisture contents of the pavements and the subgrade.

9.2.3. The paved stopway designs are intended to provide support to the Design Aircraft for 0.1% of the design frequency of trafficking for the runway, before major maintenance is required. This is achieved by designing for a reduced ACN at the design level of trafficking.

9.2.4. Using Charts 1-6 a paved stopway design can be established by the following procedures:

(i) The design ACN is the ACN of the Design Aircraft divided by 3 for flexible pavements or 2 for rigid pavements. To allow for use by emergency vehicles the design ACN should not be less than 5 with a minimum concrete thickness of 150mm.

(ii) The frequency of trafficking used for the runway design should be selected for the Charts.

(iii) The pavement thickness is obtained form the relevant chart. The actual make-up of the construction should be in accordance with Table 19.

9.2.5. With the exception of blast pads a flexible pavement is preferable to a rigid one to provide easy future rehabilitation. A failed flexible pavement with a granular base and sub-base can be recompacted, regraded and surfaced. A flexible pavement with a cement-bound base can be provided with a thin bituminous overlay, or the existing surfacing can be planed off and replaced. However, a failed rigid pavement requires a thick bituminous overlay or complete replacement.

9.2.6. As the use of a stopway is unpredictable, and it is designed and constructed to a lower standard than the movement areas, some maintenance work should be expected if the stopway is to have the same life as the runway.

9.3 SHOULDERS

9.3.1. The shoulders should be able to support an aircraft running off a runway or taxiway. The surface of the shoulders should not be susceptible to erosion and the blowing up of debris. On a runway, grassed surface shoulders will generally suffice provided the climate and topsoil are capable of sustaining them. However, taxiways used by large jets with outboard engines extending beyond the edge of the pavement may need shoulders with a paved surface to prevent erosion and foreign object damage to the aircraft.

9.3.2. Using Chart 6 or Figure 7 and Figure 8 a shoulder construction can be established in accordance with the following procedure:

(i) The ACN design parameter is the ACN of the Design Aircraft divided by 3. To allow for use by rescue vehicles the design ACN should not be less than 5.

(ii) For paved shoulders the frequency of trafficking used for the runway design should be selected. For grassed shoulders the Low frequency of trafficking should be used.

(iii) The pavement construction should accord with Table 20.

9.3.3. On grassed shoulders regrading and some reconstruction would most likely be required after each pass of an aircraft. Wheel penetration is unlikely to exceed 150mm on prepared grassed shoulders and it would be substantially less on paved shoulders. The design concept is based on References 28 and 29. As failure criteria and design methods are not precise, the designs cannot be expected to be accurate and may therefore be a little conservative.

9.3.4. Paved surfaces can give rise to a lack of visual contrast between the runway and the shoulders. This can be overcome either by providing a good visual contrast between the surfacings of the runway and shoulders or by applying a distinctive marking at the edge of the runway.

9.4 BLAST PROTECTION

9.4.1. Areas adjacent to movement areas, especially those immediately off the end of runways may be subject to blast from jet engines. In these situations paved shoulders and blast pads should be provided. They should be large enough to prevent surface erosion and migration of foreign materials onto the movement areas. The width of the paved shoulders will depend on the taxiway width and the position of the outboard jet engines of the user aircraft.

9.4.2. Shoulders and blast pads forming part of a stopway should be designed to the recommendations given in Sections 9.2 and 9.3. For aircraft with high velocity turbojet engines (e.g. fighters) a concrete surface is preferable for the blast pad, otherwise the minimum thickness of asphalt surfacing should be 75mm.

Table 19 Stopway Constructions

PAVEMENT TYPE	SURFACING	BASE/SUB-BASE		DESIGN CHART
Rigid	Pavement Quality Concrete (PQC) (Section 5.2)	DLC Cement-stabilised base Granular base.	- Section **1.1** - Section **1.1** - Section 3.8	Chart 1, 2, 3 or 5 as appropriate.
Flexible	Marshall asphalt or Hot rolled asphalt or Dense bituminous macadam The thickness of surfacing can be reduced to 50mm with these materials but the total thickness of construction (including surfacing + base/sub-base) should be kept the same as that required by the Chart i.e. increase the base course thickness by 50mm.	DLC Cement-stabilised base Cement–stabilised base Granular base and sub-base If the base is either wholly or partly cement-bound the minimum thickness of surfacing should be not less than 1/5th of the total thickness of bound pavement construction.	- Section 6.3 - Section 6.3 - Section 6.3 - Section 7.7	Chart 4 or 6 as appropriate.
	Proprietary surfacing This should be of proven durability. If laid over a cement-bound base it should not be subject to premature reflective cracking. Possibilities are proprietary blacktop materials, concrete blocks and grouted macadam.	DLC Cement-stabilised base Granular base and sub-base.	- Section 6.3 - Section 6.3 - Section 7.7	Chart 4 or 6 as appropriate. The total thickness of construction (including surfacing + base/sub-base) should not be less than that required by the Charts.

Table 20 Shoulder Construction

DESIGN AIRCRAFT ACN AND TYRE PRESSURE	SURFACING	BASE/SUB-BASE	CONSTRUCTION THICKNESS CALCULATION
ACN … 30 Tyre pressure … 1.5 MPa	Either topsoiled and grassed with a maximum topsoil depth of 100mm or as for Table 19(Flexible).	If the subgrade is equal to or better than CBR 15% no base/sub-base is required. The CBR 15% must still be valid in wet weather. If the subgrade is less than the design requirement this can be improved with granular fill (Section 3.8).	Use Figure 7 and Figure 8 to calculate the thickness of granular fill required to provide a CBR 15% support level or use Chart 6 if the depth of granular subbase required to give CBR 30% proves more economical.
ACN > 30 and all the aircraft with tyre pressures 1.5 MPa NB. The ACN is the design Aircraft CAN before dividing by 3.		If the subgrade is equal to or better than CBR 30% no base/sub-base is required. The CBR 30% must still be valid in wet weather. If the subgrade is less than the design requirement this can be improved with granular sub-base or, for paved shoulders only, the equivalent thickness of cement-bound base (Section 6.3).	Use Chart 6 to calculate the thickness of granular sub-base required; the thickness is the X ordinate minus the Y ordinate. This will provide CBR 30%. The equivalent thickness of cement-bound base can be calculated. (Section **1.1** and 6.3).

References

1. Air Ministry Works Department. Design and Construction of Concrete Pavements. Air Publication No. AP3129A. 1945.
2. Air Ministry Works Department. Load Classification of Runways an Aircraft Technical Publication 102. 1948.
3. Air Ministry Works Department. Airfield Evaluation. Technical Publication 104. 1952
4. Air Ministry Works Department. The Fundamentals of Airfield Pavement Design. Technical Publication 107 1953.
5. Air Ministry Works Department. Airfield Design and Evaluation. Technical Publication 109. 1959.
6. J L Dawson and R L Mills. Undercarriage Effects on (a) Rigid Pavements (b) Flexible Pavements. ICE Proceedings of Symposium on Aircraft Pavement Design 1970.
7. F R Martin, A R Macrae. Current British Pavement Design ICE Proceedings of Symposium on Aircraft Pavement Design 1970.
8. H Jennings, F L H Straw. Strengthening of Pavements. ICE Proceedings of Symposium on Aircraft Pavement Design 1970
9. Department of the Environment. Design and Evaluation of Aircraft Pavements 1971. 1971
10. International Civil Aviation Organisation, Aerodrome Design Manual Parts 1-3, First Edition 1977, Second Edition 1983.
11. Property Services Agency. A Guide to Airfield Pavement Design and Evaluation. HMSO. 1989.
12. US Army Engineer Waterways Experiment Station. Procedures for Development of CBR Design Curves – Instruction Report S-77-1. 1977.
13. International Civil Aviation Organisation. Annex 14. Aerodromes – Internationa Standards and Recommended Practices. Eight Edition 1983
14. Civil Aviation Authority. United Kingdom Aeronautical Information Publication. London 1982.
15. BSI. Methods of test for soils for civil engineering purposes - Part 2: Classification tests. BS 1377-2: 1990 incorporating Amendment No. 1 May 1996.
16. Road Research Laboratory. Soil Mechanics for Road Engineers. HMSO 1952.
17. D Croney. The Design and Performance of Road Pavements. HMSO London 1977.
18. US Army Engineer Waterways Experiment Station. Field Moisture Content Investigation, October 1945 – November 1952 Phase. Report No. 2 1955.
19. US Army Engineer Waterways Experiment Station. Field Moisture Content Investigation, November 1952 – May 1956 Phase. Report No. 3. 1961.
20. W A Lewis. Full Scale Compaction Studies at the British Road Research Laboratory. Highways Research Board Bulletin 254. Washington 1960.
21. US Army Engineer Waterways Experiment Station. Compaction Requirements for Soil Components of Flexible Airfield Pavements. Technical Report No 3-529. 1959
22. W J Turnbull and Charles R Foster. Proof Rolling of Subgrades. Highway Research Board Bulletin 254. Washington. 1960.
23. W D Powell, J F Potter, H C Mayhew and M E Nunn. The Structural Design of Bituminous Roads. Report LR 1132. Transport and Road Research Laboratory, Crowthorne, Berks. 1984.
24. D Croney and J C Jacobs. The Frost Susceptibility of Soils and Road Materials. RRL Report LR 90. Transport and Road Research Laboratory, Crowthorne, Berks. 1967.

25. BSI. Testing aggregates - Method for determination of frost-heave. BS 812-124:1989. 1989.
26. P G Roe and D C Webster. Specification for the TRRL Frost Heave Test, Supplementary Report 829. Transport and Road Research Laboratory, Crowthorne, Berks. 1984.
27. R G Packard. Fatigue Concepts for Concrete Aircraft Pavement Design. Transportation Engineering Journal. 1974.
28. R L Hutchinson. Basis for Rigid Pavement Designs for Military Airfields. Miscellaneous Paper No 5-7. US Army Corps of Engineers, Ohio River Institute. Washington DC 1958.
29. US Army Corps of Engineers Waterways Experiment Station. Validation of Soil Strength Criteria for Aircraft Operations on Unprepared Landing Strips. Technical Report No.3-554. July 1960.
30. Military Engineering Experimental Establishment Sinkage of a Dual Aircraft Wheel Assembly. Report No 925. Christchurch. October 1965.
31. D N Brown and O O Thompson. Lateral Distribution of Aircraft Traffic. Miscellaneous Paper S-73-56 July 1973. US Army Engineer Waterways Experimental Station, Vicksburg. 1973.
32. H M Westergaard. Stresses in Concrete Pavements Computed by Theoretical Analysis Public Roads. Vol 7 No 2. 1926.
33. G Pickett, M E Raville, W C Jones, F J McCormick. Deflections, Movements and Reactive Pressures for Concrete pavements. Kansas State College Bulletin 65. October 1951.
34. R G Packard. Computer Programme for Airport Pavement Design, Portland Cement Association. Chicago, Illinois. 1967
35. G Pickett. Concrete Pavement Design, Appendix III: A Study of Stresses in the Corner Region of Concrete Pavement Slabs under Large Corner Loads. Portland Cement Association, Skorkie.1946.
36. US Army Corps of Engineers. Final Report on the Dynamic Loading of Concrete Test Slabs – Wright Field Slab Tests. Ohio River Division Laboratories, Mariemont, Ohio August 1943.
37. R H Ledbetter. Pavement response to Aircraft Dynamic Loads. FAA Report No. FAA-RD-74-39-III. June 1973.
38. Highway Research Board. Joint Spacing in Concrete Pavements: 10 year Reports on Six Experimental Projects. Research Report 17B. 1956.
39. B E Colley and H A Humphrey. Aggregate Interlock in Joints in Concrete Pavements. Highway Research Record Number 189. 1967.
40. L D Childs. Effect of Granular and Soil-Cement Sub-bases on Load Capacity of Concrete Slabs. Journal of the PCA Research and Development Laboratories, Vol 2, No. 2. 1960.
41. L D Childs and J W Kaperick. Tests of Concrete Pavement Slabs on Gravel Sub-bases. Proceedings ASCE, Vol 84 (HW3). October 1958.
42. L D Childs. Tests of Concrete Pavement Slabs on Cement Treated Sub-bases. Highway Research Record 60, Highway Research Board. 1964.
43. L D Childs. Cement Treated Sub-bases for Concrete Pavements. Highway Research Record 189, Highway Research Board 1967.
44. Air Ministry Works Department. Investigation into the Value of Lean Concrete as a Base in Rigid Pavements. Air Ministry Tech Memo No.4. 1955.
45. R D Bradbury. Reingforced Concrete Pavements. Wire Reinforcement Institute. Washington DC. 1938.
46. L W Teller and E C Sutherland. The Structural Design of Concrete Pavements. Public Roads Vol 16, No 8, 9 and 10, 1935; Vol 17 No 7 an 8, 1936; and Vol 23, No. 8 1943.
47. J Thomlinson. Temperature Variations and Consequent Stresses Produced by Daily and Seasonal Temperature Cycles in Concrete Slabs. Road Research Laboratory. June 1940.
48. US Army Corps of Engineers. Lockbourne No 1 Test Track. Final Report. Ohio River Division Laboratories, Mariemont, Ohio. March 1946.
49. Charles R Foster and R G Ahlvin. Notes on the Corps of Engineers CBR Design Procedures. Highways Research Board Bulletin 210, Washington, 1959.
50. F R Martin. A Heavy-Duty Airfield pavement Embodying Soil Stabilisation. ICE. Airport Paper 34.

51. M A Napier. A Report on the Testing of Stabilised Soil Airfield Pavements 1955-60. MEXE Report No 592.
52. Defence Estates. Reflection cracking on airfield pavements – a design guide for assessment, treatment selection and future minimisation. Design & Maintenance Guide 33. Defence Estates. Sutton Coldfield, UK, 2005.
53. Defence Estates. The Use of the Crack and Seat Treatment. Design & Maintenance Guide 21. Defence Estates. Sutton Coldfield, UK, 2003.
54. Defence Estates. Pavement Quality Concrete for Airfields. Specification 33. Ministry of Defence. 2005.
55. Defence Estates. Marshall Asphalt for airfield pavement works. Defence Works Functional Standard 13. Ministry of Defence. 1996.
56. Defence Estates. Hot Rolled Asphalt and Coated Macadam for airfield works. Defence Works Functional Standards.
57. Defence Estates. Porous friction course for airfields. Specification 40. Ministry of Defence. 1996.
58. Defence Estates. Concrete block paving for airfields. Specification 35. Ministry of Defence. 1996.

Appendix A Extended Casagrande Soil Classification and CBR/k Relationship

A1. THE EXTENDED CASAGRANDE SOIL CLASSIFICATION (TABLE 21)

A1.1 The Casagrande classification system was devised in 1942. Casagrande later suggested an extension, including two additional soil sub-groups which are particularly suitable for UK soils. The use of the system is described in Section 3.2.

A2. CBR/K RELATIONSHIP (FIGURE 32)

A2.1 Figure 32 shows an empirical relationship between CBR and the Modulus of Subgrade Reaction (k) measured with a 762mm (30in.) diameter plate. This relationship may be used to assess k in circumstances where the plate test is impracticable (see para 3.2.5).

Table 21 The Extended Casagrande Soil Classification

1		2	3	4	5
MAJOR DIVISIONS		DESCRIPTION AND FIELD IDENTIFICATION	SUB-GROUPS	GROUP SYMBOL	APPLICABLE CLASSICATION TESTS (CARRIED OUT ON DISTURBED SAMPLES)
COARSE-GRAINED SOILS	Boulders and Cobbles	Soils consisting chiefly of boulders larger than 200mm or cobbles between 200mm and 75mm identifiable by visual inspection	Boulder gravels	-	Particle-size analysis
	Gravel and gravelly soils	Soils with an appreciable fraction between the 75mm and 2.36mm. Generally easily identifiable by visual inspection. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates the absence of clay.	Well graded gravel sand mixtures, little or no fines	GW	Particle-size analysis
			Well-graded gravel-sands with small clay content	GC	Particle-size analysis, liquid and plastic limits on binder
			Uniform gravel with little or no fines	GU	Particle-size analysis
			Poorly graded gravel-sand mixtures, little or no fines	GP	Particle-size analysis
			Gravel-sand mixtures with excess of fines	GF	Particle-size analysis, liquid and plastic limits on binder if applicable
	Sands and sandy soils	Soils with an appreciable fraction between the 2.36mm and the 75 micron sieve. Majority of particles can be distinguished by eye. Feel gritty when rubbed between the fingers. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates absence of clay	Well graded sands and gravelly sands little or no fines	SW	Particle-size analysis
			Well graded sands with small clay content	SC	Particle-size analysis, liquid and plastic limits on binder
			Uniform sands with little or no fines	SU	Particle-size analysis
			Poorly graded sands, with little or no fines	SP	Particle-size analysis
			Sands with excess of fines	SF	Particle-size analysis, liquid and plastic limits on binder if applicable
FINE-GRAINED SOILS Containing little or no coarse-grained material	Fine-grained soils having low plasticity (silts)	Soils with an appreciable fraction passing the 75 micron sieve and with liquid limits less than 35. Not gritty between the fingers. Cannot be readily rolled into threads when moist. Exhibit dilatancy	Silts (inorganic) rock flour, silty fine sands with slight plasticity.	ML	Particle-size analysis, liquid and plastic limits if applicable
			Clayey silts (inorganic)	CL	Liquid and plastic limits
			Organic silts of low plasticity	OL	Liquid and plastic limits from natural conditions and after oven-drying
	Fine-grained soils having medium plasticity	Soils with liquid limits between 35 and 50. can be readily rolled into threads when moist. Do not exhibit dilatancy. Show some shrinkage on drying.	Silty clays (inorganic) and sandy clays	MI	Particle size analysis, liquid and plastic limits if applicable
			Clays (inorganic) of medium plasticity	CI	Liquid and plastic limits
			Organic clays of medium plasticity	OI	Liquid and plastic limits from natural conditions and after oven-drying
	Fine grained soils having high plasticity	Soils with liquid limits greater than 50. Can be readily rolled into threads when moist. Greasy to the touch. Show considerable shrinkage on drying. All highly compressible soils	Highly compressible micaceous or diatomaceous soils	MH	Particle size analysis, liquid and plastic limits if applicable
			Clays (inorganic) of high plasticity	CH	Liquid and plastic limits
			Organic clays of high plasticity	OH	Liquid and plastic limits from natural conditions and after oven-drying
Fibrous organic soils with very high compressibility		Usually brown or black in colour. Very compressible Easily identifiable visually.	Peat and other highly organic swamp soils.	Pt	Moisture content and

*These unit weights apply only to soils with specific gravities ranging between 2.65 and 2.75

6	7	8	9	10	11
APPLICABLE OBSERVATIONS AND TESTS RELATING TO THE MATERIAL IN PLACE (OR CARRIED OUT ON UNDISTURBED SAMPLES)	VALUE AS A ROUND FOUNDATION WHEN NOT SUBJECT TO FROST ACTION	POTENTIAL FROST ACTION	SHRINKAGE OR SWELLING PROPERITES	DRAINAGE CHARAC-TERISTICS	MAX. DRY DENSITY AT OPTIMUM COMPACTION (kg/m³) & VOIDS RATIO, e*
Dry density and relative compaction	Good to excellent	None to very slight	Almost none	Good	-
Moisture content and voids ratio	Excellent	None to very slight	Almost none	Excellent	>2000 e <0.35
Cementation Durability of grains	Excellent	Medium	Very slight	Practically impervious	>2080 e <0.50
Stratification and drainage characteristics	Good	None	Almost none	Excellent	>1760 e <0.50
	Good to excellent.	None to very slight	Almost none	Excellent	>1840 e <0.40
Ground water conditions	Good to excellent	Slight to medium	Almost none to slight	Fair to practically impervious	>1920 e <0.40
Large-scale loading tests	Excellent to good	None to very slight	Almost none	Excellent	>1920 e <0.40
California bearing ratio tests, sheer tests and other strength tests	Excellent to good	Medium	Very slight	Practically impervious	>2000 e <0.35
	Fair	None to very slight	Almost none	Excellent	>1600 e <0.70
	Fair to good	None to very slight	Almost none	Excellent	>1600 e <0.70
	Fair to good	Slight to high	Almost none to medium	Fair to practically impervious	>1680 e <0.60
	Fair to poor	Medium to very high	Sight to medium	Fair to poor	>1600 e <0.70
Dry density and relative compaction	Fair to poor	Medium to high	Medium	Practically impervious	>1600 e <0.70
Moisture content and voids ratio	Poor	Medium to high	Medium to high	Poor	>1440 e <0.90
Stratification, fissures, etc	Fair to poor	Medium	Medium to high	Fair to poor	>1600 e <0.70
Drainage and ground water conditions	Fair to poor	Slight	High	Fair to practically impervious	>1520 e <0.80
Consolidation tests	Poor	Slight	High	Fair to practically impervious	>1520 e <0.80
Large-scale loading tests	Poor	Medium to high	High	Poor	>1600 e <0.70
California bearing ratio tests, sheer tests and other strength tests	Poor to very poor	Very slight	High	Practically impervious	>1400 e <0.90
	Very poor	Very slight	High	Practically impervious	>1600 e <0.70
consolidation tests	Extremely poor	Slight	Very high	Fair to poor	-

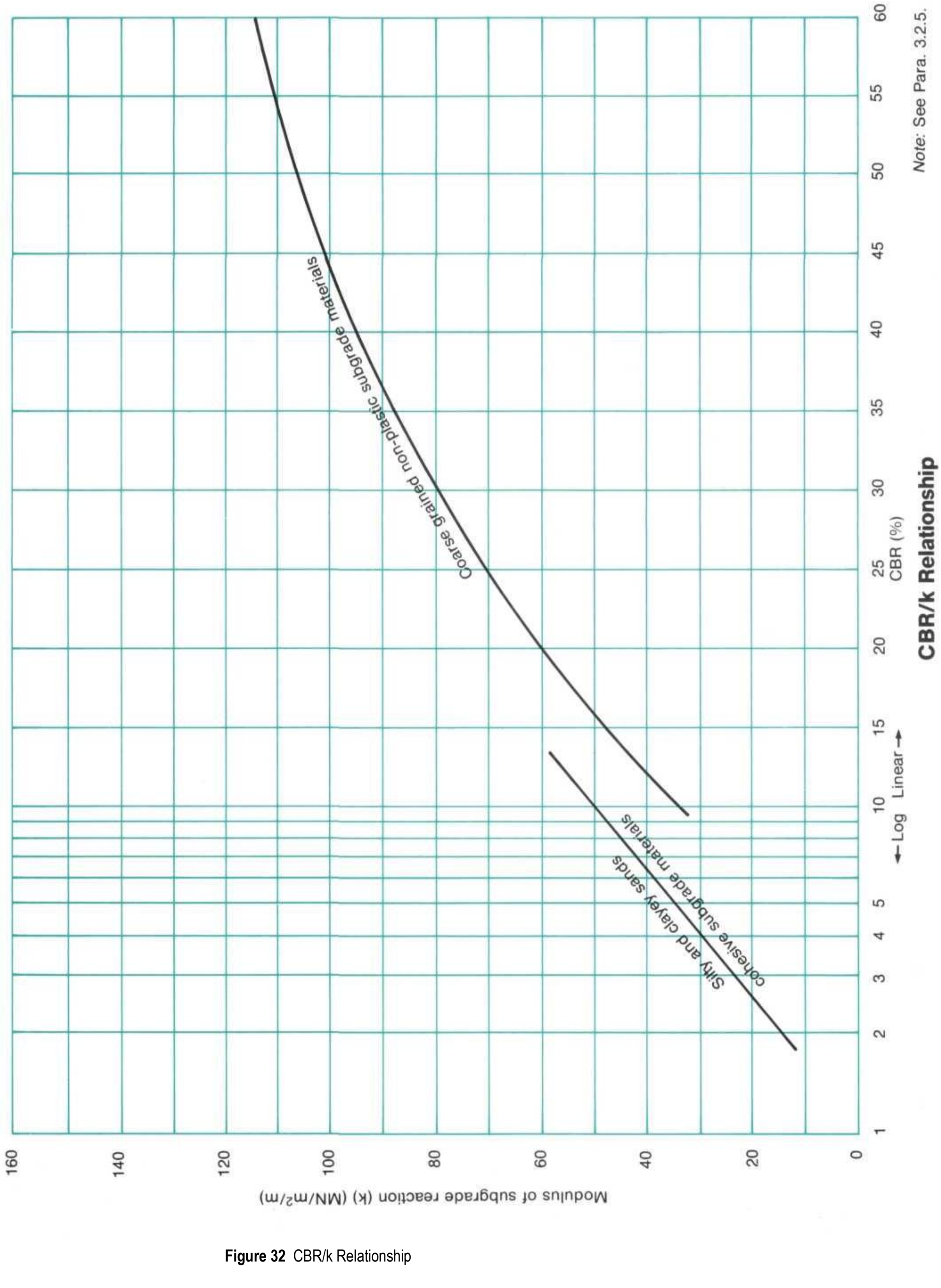


Figure 32 CBR/k Relationship

Appendix B ACNs for Several Aircraft Types

The two all-up masses shown in Column 2 for each aircraft type are respectively the Maximum Ramp Weight and a representative operating empty weight. To compute the ACN for any intermediate value, assume that the ACN varies linearly between the operating empty weight and the Maximum Ramp Weight.

The information provided is believed to be correct at the dates given. As the aircraft are often modified during service, it would be prudent to confirm that the information given is correct at the time of use.

Where there are a number of weight options for a particular aircraft model, the heaviest weight has been used for the ACN calculation.

The Flexible Pavement ACNs are based on the revision to the ACN-PCN method promulgated by ICAO on 17 October 2007. Considerebale care must be taken to ensure that Charts 5, 6 and 8 are used with updated ACNs.

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
A10A	22,680	45	1.28	20.6	20.6	20.5	20.5	20.7	20.2	20.3	20.3	Single
	9,072			8.2	8.2	8.2	8.1	8.3	8.1	8.1	8.1	
A300-600	165,000	46.7	1.81	51	59.2	68.2	76.4	46.8	51	61.8	78.6	Dual Tandem
	83,347			21	23.5	27	30.8	19.6	20.7	23	30.2	
A300B2	142,900	46.9	1.24	37.4	44.9	53.2	60.9	37.8	42	51	65.8	Dual Tandem
	86,275			19.3	22.4	26.5	30.8	19.9	21.3	24.5	32.4	
A300B4-100	158,400	46.8	1.42	44.4	52.7	61.8	70	43.1	47.9	58.2	74.7	Dual Tandem
	88,330			20.7	23.6	27.7	32	20.7	22	24.9	32.9	
A300B4-200	165,900	46.8	1.24	44.9	54.1	64.1	73	45.1	50.8	62.1	79.1	Dual Tandem
	88,500			19.6	22.6	26.8	31.2	20.3	21.7	24.8	33	
A300C4	165,900	46.8	1.24	44.9	54.1	64.1	73	45.1	50.8	62.1	79.1	Dual Tandem
	84,000			18.3	21.1	25	29.1	19.2	20.2	23.1	30.6	
A300F4-600	171,400	47.3	1.34	49.2	58.9	69.2	78.3	48.1	54.3	66.3	83.7	Dual Tandem
	79,660			18	20.5	24.1	27.9	18.3	19.4	21.9	28.7	
A310-200	132,900	46.8	1.23	32.9	39.3	46.9	54	33.8	37.1	44.7	59	Dual Tandem
	73,783			15.9	17.8	20.9	24.4	16.3	17.2	19.4	25.3	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
A310-300	164,900	47.3	1.29	45.9	55.1	65	73.9	45.5	51.2	62.6	79.7	Dual Tandem
	77,037			17	19.4	22.8	26.5	17.5	18.5	20.9	27.4	
A318	68,400	44.5	1.24	36	38.4	40.6	42.5	32.7	33.8	37.4	43.1	Dual
	38,818			18.6	19.8	21.1	22.2	17.4	17.5	18.8	21.5	
A319	75,900	45.8	1.45	44.3	46.8	49.1	51	38.8	40.6	44.6	50.5	Dual
	39,225			20.5	21.6	22.8	23.9	18.4	18.6	19.9	22.7	
A320-100	68,400	47.1	1.24	38.6	41.1	43.4	45.4	34.9	36.1	40.1	46.1	Dual
	39,670			20.3	21.7	23.1	24.3	19	19.1	20.6	23.8	
A320-200	77,400	46.6	1.44	46.2	48.8	51.2	53.2	40.5	42.4	46.8	52.6	Dual
	40,526			21.6	22.9	24.2	25.3	19.5	19.7	21.1	24.2	
A321-100	83,400	48	1.36	51.5	54.4	57.1	59.3	45.3	47.7	53.3	59.1	Dual
	47,486			26.4	28	29.6	31	23.7	24.1	26.3	30.6	
A321-200	89,400	47.5	1.46	56.5	59.4	62.1	64.3	49.4	52	57.6	63.2	Dual
	47,000			26.4	27.9	29.4	30.7	23.5	24	25.8	29.9	
A330-200	233,900	47.8	1.39	53.7	62.4	74.3	86.9	58.5	63.5	73.8	99.8	Dual Tandem
	120,300			27.8	27.2	30.9	35.5	26.4	27.5	30.1	37.3	
A330-300	233,900	47.6	1.42	54	62.6	74.3	86.7	58.2	63.3	73.4	99.3	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	129,646			29.7	29.9	33.9	39.2	28.8	30.1	33	41.3	Tandem
A340-200	275,900	37	1.42	72	76.7	80.9	84.4	67	70.9	79.7	87.5	Dual Tandem
	131,581			29.5	31.7	33.9	35.8	27.6	28.4	31.5	36.9	
A340-300	275,900	37.5	1.42	66.9	71.4	75.4	78.7	62.3	65.6	74.2	89.2	Dual Tandem
	140,000			29.5	31.8	34	35.8	27.7	28.5	31.6	37	
A340-500	369,200	36.5	1.61	72.8	84.7	100	115.3	75.3	82.2	97.8	129.8	Dual Tandem
	168,468			30.7	31.2	35.1	40.2	29.1	30.3	33	41.1	
A340-600	369,200	34.4	1.61	67.5	78.1	92.1	106.4	69.9	75.9	89.3	119.6	Dual Tandem
	174,867			30.1	30.3	34.1	39.1	28.3	29.5	32.1	39.8	
A380-800	562,000	19	1.5	56.3	65.9	78.2	94.6	58.4	63.7	75.3	105.5	Dual Tandem
	270,281			24.4	25.8	29.3	33.8	24.1	25.1	27.5	34.3	
A380-800F	592,000	19	1.5	59	68.9	82.6	102.3	61.7	67.2	81.3	113.8	Dual Tandem
	250,826			22.9	23.3	26.1	29.9	21.9	22.8	24.7	30.4	
An124	405,000	48.2	1.03	36.5	50	75.6	103.8	53.6	63.2	81.6	112.5	Not covered
	251,744			20.1	23.5	31.3	45.1	27.2	30.9	38.1	55.1	
Andover C Mk 1	22,680	45.6	0.55	10.8	11.8	12.7	13.5	8.6	10.7	12.7	14.7	Dual
	13,472			5.7	6.3	6.9	7.3	4.8	5.5	6.5	8	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
Andover CC Mk 2	20,183	46.2	0.53	9.4	10.3	11.2	11.9	7.5	9.3	11	13.1	Dual
	11,884			4.9	5.5	6	6.4	4.3	4.8	5.6	6.9	
Andover CC Mk 2 (TQF)	21,090	46.2	0.59	10.3	11.2	12.1	12.7	8.2	10.2	11.8	13.8	Dual
	11,884			5.1	5.7	6.2	6.5	4.2	5	5.7	7	
Andover E Mk 3	22,680	45.6	0.55	10.8	11.8	12.7	13.5	8.6	10.7	12.7	14.7	Dual
	14,747			6.4	7.1	7.7	8.2	5.2	6.2	7.2	8.9	
ASTOR Sentinel R1	43,658	47.5	1.25	29.5	30.8	32	32.9	25.4	27.4	30	31.8	Single
	25,401			15.5	16.3	17	17.7	13.2	13.7	15.3	17.4	
B-1B	216,364	46.5	1.52	67.4	79.8	92.5	103.5	63.4	71.9	89.3	108.7	Dual Tandem
	85,729			19.8	22.3	25.9	29.9	19.5	20.7	23.1	30.3	
B2	152,634	45.9	1.48	43.8	51.4	59.9	67.5	41.3	46	56.7	72.3	Single
	49,895			11.9	12	13.4	15.2	10.5	10.9	11.8	14.5	
B52	217,725	52	1.83	100.3	110.6	121.3	130.9	71.2	76.2	85.6	101.9	Dual
	104,326			37.6	41.6	46.2	50.8	27.8	29.4	32.4	39.1	
B52 2 wheels	217,725	26	1.83	84.5	87.5	90.3	92.6	71	75.3	81.8	86.8	Dual
	104,326			35.2	36.8	38.4	39.8	29.9	30.8	32.8	37.7	
B52 2 wheels (1990	221,353	27.2	2.1	94.4	97.1	99.5	101.6	78.2	82	88.4	93	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m³				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
data)	81,329			28.9	30.1	31.3	32.3	24.2	24.6	25.8	29.5	
B52 8 wheels	217,725	100	1.83	95.1	104.9	115.2	124.5	49.7	52.7	59.8	72.7	Dual
	104,326			35.8	39.6	43.9	48.3	19.6	20.6	22.4	27.2	
B52 8 Wheels (1990 data)	221,353	100	2.1	87.8	94.3	102.1	110.3	47.4	49.6	55.5	65.8	Dual
	81,329			26.2	27.3	29.1	31.4	14.2	14.4	15.3	17.7	
B52G (1990 data)	221,353	54.4	2.1	97.6	105.1	113.8	122.8	71.2	75.2	83.5	95.3	Dual
	81,329			28.9	30.1	32.2	34.8	21.1	21.5	23	27	
B52H (1990 data)	221,353	54.4	2.1	97.6	105.1	113.8	122.8	71.2	75.2	83.5	95.3	Dual
	83,189			29.7	30.9	33.1	35.8	21.6	22.1	23.6	27.9	
B707-120B	117,027	46.7	1.17	27.9	33.2	39.7	45.9	28.8	31.7	37.8	49.5	Dual Tandem
	57,883			12.7	13.2	15.3	17.8	12.4	12.8	14.3	18	
B707-320 & 420	143,500	46	1.24	36.5	43.7	51.9	59.4	37	41	49.7	64.1	Dual Tandem
	64,600			14	15	17.4	20.3	13.8	14.4	16.2	20.6	
B707-320B	148,778	46	1.24	38.4	46	54.6	62.4	38.8	43.2	52.4	67.2	Dual Tandem
	64,764			14	15.1	17.5	20.3	13.8	14.5	16.2	20.6	
B707-320C	152,407	46.7	1.24	40.5	48.7	57.6	65.7	40.8	45.7	55.4	70.8	Dual Tandem
	61,463			13.5	14.4	16.6	19.3	13.2	13.8	15.4	19.5	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
B717-200	53,977	47.9	1.13	34.2	36.1	37.8	39.2	30.4	32.1	36.1	39	Dual
	30,617			17.3	18.4	19.5	20.4	15.6	15.9	17.7	20.6	
B717-200 High Gross Weight	55,338	47.2	1.13	34.7	36.5	38.2	39.6	30.8	32.5	36.5	39.5	Dual
	31,071			17.3	18.4	19.5	20.4	15.6	15.9	17.7	20.6	
B720	104,400	47.4	1	25.1	30.7	37	42.8	26.5	28.9	35.8	47.2	Dual Tandem
	50,300			10.2	11.4	13.5	15.9	10.8	11.3	12.8	16.4	
B720B	106,700	46.3	1	25.1	30.6	37	42.7	26.5	28.8	35.7	47.1	Dual Tandem
	52,200			10.3	11.6	13.7	16.2	11	11.4	13	16.8	
B727-100	77,200	47.5	1.09	44.9	48	50.7	53	41.3	43.2	49.2	54.3	Dual
	39,800			20.4	21.9	23.4	24.6	19	19.5	21.4	25	
B727-200	78,500	48.5	1.15	47.8	50.9	53.6	55.8	43.5	45.6	51.5	56.6	Dual
	44,330			24.1	25.8	27.5	28.9	22.5	22.8	25.1	29.3	
B727-200 Advanced	95,300	46.5	1.19	58.2	61.5	64.5	67	52.2	55.4	62.2	67.1	Dual
	45,720			24.1	25.7	27.3	28.7	22.2	22.6	24.8	29	
B727-200 Advanced Basic	86,700	47.6	1.06	51.4	54.9	57.9	60.4	47.5	50	56.8	61.8	Dual
	44,390			23	24.7	26.4	27.9	21.4	22.1	24.4	28.6	
B737 BBJ	77,791	45.9	1.41	47.1	49.6	51.8	53.7	41	43.2	48.1	53	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	41,887			22.6	23.9	25.2	26.4	20.1	20.5	22.1	25.7	
B737 BBJ2	79,245	46.8	1.41	49.3	51.8	54.2	56.1	42.9	45.4	50.4	55.3	Dual
	43,875			24.4	25.8	27.2	28.5	21.7	22.3	24	27.8	
B737-100	50,340	46	1.08	27.1	29	30.7	32.2	24.6	25.7	28.8	33.1	Dual
	28,120			13.7	14.6	15.6	16.5	12.7	13	14.1	16.3	
B737-200	52,610	46.4	1.13	29.3	31.2	33	34.5	26.4	27.5	30.9	35.2	Dual
	27,120			13.4	14.4	15.3	16.1	12.4	12.6	13.7	15.7	
B737-200 Advanced	58,330	45.9	1.26	33.8	35.8	37.6	39.1	29.7	31	35	39.2	Dual
	29,620			15.2	16.2	17.1	18	13.9	14	15.1	17.4	
B737-200 Advanced Low Tyre Pressure	53,290	46.4	0.66	24.7	27.3	29.7	31.6	22.1	26.3	29.8	35.1	Dual
	29,250			12	13.3	14.6	15.7	10.8	12.7	14.1	17.1	
B737-200 Basic	45,720	46.5	0.97	23.6	25.5	27.2	28.6	21.7	22.9	25.7	29.9	Dual
	27,170			12.8	13.8	14.8	15.7	12	12.4	13.6	15.8	
B737-200C & QC	58,330	45.9	1.26	33.8	35.8	37.6	39.1	29.7	31	35	39.2	Dual
	29,620			15.2	16.2	17.1	18	13.9	14	15.1	17.4	
B737-300	63,500	45.8	1.39	38.5	40.4	42.3	43.8	33.2	35	39	43.1	Dual
	32,900			17.7	18.7	19.7	20.6	15.6	15.8	17.1	19.9	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
B737-400	68,250	46.9	1.28	42.1	44.4	46.5	48.2	37	39.1	43.9	47.8	Dual
	33,650			18.2	19.3	20.4	21.4	16.4	16.6	18.1	21	
B737-500	60,800	46.1	1.34	36.4	38.4	40.2	41.7	31.7	33.2	37.2	41.3	Dual
	28,050			14.6	15.5	16.4	17.2	13.2	13.3	14.3	16.4	
B737-600	65,771	45.8	1.41	38.4	40.5	42.5	44.2	33.4	35.1	38.7	43.8	Dual
	36,378			19.2	20.3	21.4	22.4	17.1	17.3	18.6	21.5	
B737-700	70,307	45.8	1.41	41.6	43.9	46	47.7	36.2	38	42.2	47.2	Dual
	37,648			19.9	21.1	22.3	23.3	17.8	18	19.4	22.4	
B737-800	79,243	46.8	1.41	49.3	51.8	54.2	56.1	42.9	45.4	50.4	55.3	Dual
	41,413			22.8	24.2	25.5	26.6	20.3	20.7	22.4	25.9	
B737-900	79,243	46.8	1.41	49.3	51.8	54.2	56.1	42.9	45.4	50.4	55.3	Dual
	42,901			23.8	25.2	26.5	27.7	21.1	21.7	23.3	27.1	
B747-100 & 100B	341,500	23.1	1.32	41.4	49.3	58.9	68	42.9	47.3	56.5	76	Dual Tandem
	171,840			18	19.9	23.1	26.9	18.5	19.4	21.6	27.6	
B747-100SF	334,700	23.1	1.32	40.4	48	57.3	66.1	41.8	46	54.7	74	Dual Tandem
	173,010			18.1	20.1	23.3	27.1	18.7	19.6	21.8	27.9	
B747-100SR	273,500	24.1	1.11	30.3	36.2	43.8	51.5	33.5	36.4	42.9	59.1	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	162,890			16.5	18.5	21.6	25.4	18.1	18.7	21.1	27	Tandem
B747-200	379,100	22.7	1.3	46.3	55.5	66.3	76.2	47.8	53.2	64.6	85.4	Dual Tandem
	170,600			17.4	19.2	22.3	25.9	18	18.8	20.9	26.5	
B747-200C	379,100	23.2	1.39	49	58.5	69.4	79.4	49.7	54.9	66.9	87.9	Dual Tandem
	178,640			19.2	21.4	24.8	28.8	19.7	20.7	22.9	29.4	
B747-200F	379,100	23.2	1.39	49	58.5	69.4	79.4	49.7	54.9	66.9	87.9	Dual Tandem
	156,610			16.7	18.2	20.9	24.2	16.8	17.6	19.3	24.2	
B747-300	341,500	23.1	1.37	42.1	49.9	59.4	68.4	42.9	47.3	56.5	76.1	Dual Tandem
	173,030			18.4	20.4	23.5	27.3	18.8	19.7	21.9	27.9	
B747-300SR	273,500	24.1	1.11	30.3	36.2	43.8	51.5	33.5	36.4	42.9	59.1	Dual Tandem
	164,640			16.7	18.7	21.9	25.8	18.3	19	21.4	27.5	
B747-400	397,800	23.3	1.37	52.4	62.7	74.4	85.1	53	59	72.5	94.1	Dual Tandem
	178,756			19.2	21.4	24.9	28.9	19.7	20.8	23.1	29.7	
B747-400D	278,279	23.7	1.03	29.3	35.2	43	50.8	33.5	35.7	42.8	59.2	Dual Tandem
	181,723			17.7	20.1	23.9	28.3	20.1	20.8	23.8	31.1	
B747-400ER	414,134	23.4	1.59	59.2	69.8	81.7	92.5	56.7	63.4	77.8	99.8	Dual Tandem
	184,567			21.1	23.6	27.2	31.3	21	22	24.3	31.4	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
B747SP	318,800	21.9	1.4	36.2	42.5	50.4	58.2	36.9	40.2	47	64.3	Dual Tandem
	147,970			14.8	16	18.1	20.8	14.8	15.3	16.7	20.5	
B757-200	116,100	45.6	1.26	30.7	36.8	43.4	49.3	29.7	33	40.5	53	Dual Tandem
	59,350			12.8	14.7	17.2	20	13	13.5	15.4	20.2	
B757-200PF	116,100	45.6	1.26	30.7	36.8	43.4	49.3	29.7	33	40.5	53	Dual Tandem
	51,700			10.7	12.3	14.3	16.7	11	11.5	12.8	16.3	
B757-300	122,930	46.3	1.34	34.9	41.5	48.6	54.8	32.8	36.7	45.2	58.2	Dual Tandem
	64,580			14.8	17	20	23.2	14.6	15.6	17.6	23.5	
B767-200	143,789	46.2	1.31	33.2	39.1	46.6	54.1	34.6	37.6	43.9	60.6	Dual Tandem
	80,127			16.8	18.4	21.2	24.6	17.2	17.8	19.8	25	
B767-200ER	179,623	45.5	1.31	43.6	52.2	62.3	71.7	45	49.8	60.1	80.5	Dual Tandem
	82,377			17	18.6	21.5	25	17.4	18.1	20.2	25.5	
B767-300	159,665	46.1	1.34	38.3	45.4	54.1	62.5	39.5	43.3	51.1	69.9	Dual Tandem
	86,069			18.2	20.1	23.3	27.1	18.6	19.5	21.7	27.8	
B767-300 Freighter	187,334	46.2	1.38	48.2	57.5	68.3	78.2	48.8	54	65.9	86.8	Dual Tandem
	86,183			18.5	20.4	23.6	27.4	18.8	19.7	21.8	27.9	
B767-300ER	187,334	46.2	1.38	48.2	57.5	68.3	78.2	48.8	54	65.9	86.8	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	90,011			19.4	21.6	25	29.1	19.8	20.8	23.1	29.8	Tandem
B767-400ER	204,630	47	1.48	58	69	81.1	92	56.8	63.2	78.9	100.3	Dual Tandem
	103,150			23.9	27	31.6	36.6	24	25.4	28.6	38.5	
B777-200	243,500	47.7	1.28	38	47.3	62.1	77.7	39.3	43.7	52.6	74.6	Tridem
	133,350			20.5	20.6	24.7	31.1	18.2	19.5	22.1	29.1	
B777-200 High Gross Weight	287,800	46.9	1.48	49.6	63	81.5	99.7	49.2	54.2	66.6	92.9	Tridem
	135,600			21.6	21.6	25.8	32.1	18.5	19.7	22.2	29.2	
B777-200LR	348,721	45.8	1.5	63.5	82.3	105.8	127.6	61.7	69.1	86.4	116.9	Tridem
	145,149			22.8	22.8	27.4	34.2	19.5	20.8	23.5	31	
B777-300	300,280	47.4	1.48	53.6	68.6	88.6	107.9	52.5	58.6	72.3	100.1	Tridem
	157,800			26.1	26.6	32.7	40.9	22.6	24.3	27.8	37.6	
B777-300ER	352,441	46.2	1.52	65.8	85.3	109.3	131.5	63.6	71.1	89.1	120.1	Tridem
	167,829			27.4	28	34.5	43.2	23.6	25.4	29.1	39.5	
BAC 1-11 Series 400	39,690	47.5	0.93	24.8	26.2	27.5	28.5	22.1	23.9	26.7	28.6	Dual
	22,498			12.5	13.4	14.1	14.8	10.8	11.6	13.2	15.2	
BAC 1-11 Series 475	44,679	47.5	0.57	22.6	24.8	26.8	28.3	19.1	24.1	27.5	31.1	Dual
	23,451			10.2	11.4	12.4	13.3	8.6	10.3	12.1	14.8	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
BAC 1-11 Series 500	47,400	47.5	1.08	29.5	31.2	32.7	33.9	26.3	27.7	31.3	33.9	Dual
	24,757			13.6	14.5	15.3	16	12	12.4	13.8	16.2	
BAe 146 (TQF)	37,535	46.5	0.93	19.3	20.8	22.2	23.4	17.5	18.6	21	24.4	Dual
	21,183			9.9	10.7	11.4	12.1	9.1	9.6	10.4	12.1	
BAe 146 Series 100	37,308	46	0.8	18	19.6	21.1	22.3	16.3	18	20.2	23.9	Dual
	23,000			10.2	11.1	12	12.8	9.4	10.2	11.1	13.2	
BAe 146 Series 100 Low Tyre Pressure	37,308	46	0.52	15.2	17.2	19	20.5	13.9	16.1	19.3	23.4	Dual
	23,000			8.5	9.6	10.7	11.6	7.2	9	10.5	12.9	
BAe 146 Series 200	40,600	47.1	0.88	21.2	22.9	24.4	25.7	19.1	20.6	23.5	27.1	Dual
	23,000			10.8	11.7	12.6	13.4	9.9	10.6	11.6	13.6	
BAe 146 Series 200 Low Tyre Pressure	40,600	47.1	0.61	18.6	20.6	22.5	24	16	19.6	22.5	26.8	Dual
	23,000			9.4	10.5	11.5	12.4	8.3	9.8	11	13.4	
Buccaneer S Mk 2A	26,935	46.1	1.79	27.8	27.5	27.2	27	26.4	25.7	25.3	24.9	Single
	14,014			14.4	14.2	14.1	14	13.7	13.4	13.2	13	
Buccaneer S Mk 2B	26,935	46.7	1.79	28.2	27.9	27.6	27.3	26.8	26	25.6	25.3	Single
	14,286			14.9	14.7	14.6	14.4	14.2	13.8	13.6	13.4	
C130B	61,235	47.5	0.68	24.8	26.9	29.3	31.5	21.9	25.4	27.4	31.6	Not

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	32,795			12.7	13.6	14.5	15.5	11.1	12.8	13.7	15.3	covered
C130D	56,336	47.5	0.63	21.9	23.8	26	28.1	19.4	22.4	24.8	28.6	Not covered
	31,252			11.7	12.5	13.4	14.4	10.2	11.7	12.9	14.4	
C130E	79,379	47.5	0.72	34	36.9	40.1	43.1	30.4	34.5	37.2	43.6	Not covered
	32,659			13	13.8	14.7	15.7	11.4	13	13.8	15.3	
C130H	79,379	47.8	0.8	35.8	38.6	41.6	44.5	32.5	35.6	37.8	44.2	Not covered
	34,156			14.3	15.1	16	17	12.9	14.1	14.8	16.3	
C141B	146,556	47.2	1.31	45.1	53.7	62.5	70.1	42.3	48.2	59.2	73.3	Dual Tandem
	68,039			16	18.4	21.7	25	15.9	16.9	19.3	25.6	
C17A	263,084	47.5	0.95	52.4	46.9	54.5	67.7	41.7	46.3	55.2	72.4	Tridem
	201,395			39	37	37.9	45.4	29.3	31.6	37.7	49.4	
C5A	348,813	23.8	0.73	25.7	30	39	49.4	25.8	28.2	34.3	45.9	Tridem
	288,485			20.8	22.9	29.2	37.1	19.9	21.9	26.1	34.6	
C5B	381,018	23.8	0.77	29.4	34.9	45.4	57	29.5	32.1	38.9	52.5	Tridem
	169,644			12.5	12.5	13.9	17.1	10.2	11	12.7	16.1	
Caravelle Series 10	52,000	46.1	1.17	15.6	18	20.7	23.1	14.2	15.9	18.2	21.5	Dual Tandem
	29,034			7.3	8.2	9.4	10.7	6.7	7.1	8.3	10.2	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
Caravelle Series 12	55,960	46	1.08	17.4	20.2	23	25.6	16	17.9	20.1	23.7	Dual Tandem
	31,800			8.2	9.2	10.7	12.1	7.5	8.2	9.6	11.5	
Chinook HC Mk 1	22,700	30.8	0.6	10.7	11	11.3	11.6	9.6	11.4	12.6	13.2	Dual
	10,411			5	5.1	5.3	5.3	4.4	5.2	5.8	6	
CL44	95,708	48	1.26	31.6	36.7	42.1	46.7	29.8	33.5	37.8	45.1	Dual Tandem
	40,370			10.4	11.3	13	14.9	9.6	10.3	11.8	14.7	
Convair 880M	87,770	46.6	1.03	25.8	30.8	35.9	40.3	24.4	28.1	33	40.6	Dual Tandem
	40,195			9	10.4	12.2	14.1	9	9.5	11	14.2	
Convair 990	115,666	48.5	1.28	40.4	47.3	54.1	59.9	36.7	42	48.8	59.4	Dual Tandem
	54,685			14.3	16.5	19.3	22.1	13.6	14.7	17.1	21.8	
Dash 7	19,867	46.8	0.74	11.2	12	12.6	13.2	9.5	10.7	12.3	13.7	Dual
	11,793			6	6.5	6.9	7.2	5	5.7	6.2	7.5	
DC10-10	196,406	46.7	1.34	44.7	52.2	62.4	72.8	48.1	52.1	60.7	84	Dual Tandem
	108,940			23.1	24.7	28.3	32.8	23.6	24.6	27.3	34.3	
DC10-10CF	200,942	46.7	1.34	46	53.8	64.4	75.1	49.4	53.7	62.8	86.7	Dual Tandem
	110,563			23.5	25.2	28.8	33.5	24	25.1	27.8	35.1	
DC10-30	253,105	37.5	1.22	48.7	52.8	63.8	74.9	49.5	54	63.6	88.1	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	120,742			20.2	21.8	23.7	27.5	20.4	20.9	23.3	28.8	Tandem
DC10-30CF	253,105	37.5	1.22	48.7	52.8	63.8	74.9	49.5	54	63.6	88.1	Dual Tandem
	121,904			20.4	22	23.9	27.9	20.6	21.2	23.6	29.2	
DC10-40	253,105	37.5	1.22	48.7	52.8	63.8	74.9	49.5	54	63.6	88.1	Dual Tandem
	122,567			20.6	22.2	24.1	28.1	20.8	21.3	23.8	29.5	
DC10-40CF	253,105	37.5	1.22	48.7	52.8	63.8	74.9	49.5	54	63.6	88.1	Dual Tandem
	123,728			20.8	22.4	24.4	28.4	21	21.5	24	29.9	
DC3	11,430	46.8	0.31	6	6.5	6.9	7.2	3.9	5.5	7.4	9.2	Single
	7,767			4.2	4.5	4.8	4.9	2.6	3.7	5.1	6.3	
DC4	33,113	46.8	0.53	13.2	14.9	16.6	17.9	12.3	13.9	16.5	20.4	Dual
	22,075			8.2	9.2	10.3	11.2	7	8.7	10.1	12.2	
DC8-43	144,245	46.6	1.22	40.8	48.8	57.3	64.8	40.1	45.5	54.2	67.7	Dual Tandem
	61,920			14.2	15.3	17.9	20.8	13.7	14.5	16.4	21.2	
DC8-55	148,781	47.3	1.28	44.4	52.8	61.7	69.4	42.9	48.8	57.9	71.9	Dual Tandem
	62,717			14.6	16.1	18.8	21.8	14.2	15.1	17.1	22.1	
DC8-55F	148,781	47.4	1.28	44.5	53	61.8	69.6	43	49	58	72.1	Dual Tandem
	59,526			14.1	15.1	17.6	20.4	13.4	14.2	16	20.6	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
DC8-61	148,781	48.1	1.3	45.7	54.3	63.3	71.1	43.8	50.1	59.2	73.5	Dual Tandem
	68,993			16.5	18.8	22	25.4	16.4	17.5	19.9	25.8	
DC8-61F	150,142	48	1.31	46.3	55	64	71.9	44.3	50.6	59.8	74.1	Dual Tandem
	66,002			15.7	17.7	20.7	23.9	15.5	16.5	18.7	24.2	
DC8-62	160,121	46.7	1.32	47.1	56.1	65.5	73.8	45.4	51.6	61.8	77	Dual Tandem
	64,980			14.9	16.3	19	22	14.5	15.3	17.2	22.2	
DC8-62F	160,121	47.5	1.32	48.3	57.5	67.1	75.5	46.4	52.9	63.3	78.7	Dual Tandem
	62,851			14.7	16	18.6	21.5	14.2	15	16.8	21.7	
DC8-63	162,389	48.1	1.35	50.6	60.1	69.9	78.5	48.3	55	65.8	81.5	Dual Tandem
	72,004			17.2	19.5	22.8	26.3	17.1	18.2	20.6	26.8	
DC8-63F	162,389	48.1	1.35	120.1	122.8	125.6	128.3	68.2	74.4	81	88.1	Dual Tandem
	64,107			41.1	42.1	43.3	44.6	16.4	18.1	23.7	29.2	
DC8-71	148,781	48.1	1.3	45.7	54.3	63.3	71.1	43.8	50.1	59.2	73.5	Dual Tandem
	74,254			18	20.7	24.3	28	17.9	19.3	22	28.5	
DC8-71F	150,142	48	1.31	46.3	55	64	71.9	44.3	50.6	59.8	74.1	Dual Tandem
	66,002			15.7	17.7	20.7	23.9	15.5	16.5	18.7	24.2	
DC8-72	153,317	46.7	1.32	44.3	52.7	61.7	69.6	42.9	48.4	58.1	72.9	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	69,492			15.8	17.8	20.8	24	15.8	16.7	18.8	24.4	Tandem
DC8-72F	153,317	47.5	1.32	45.4	54	63.1	71.2	43.8	49.6	59.5	74.5	Dual Tandem
	63,595			14.9	16.2	18.9	21.8	14.4	15.2	17.1	22.1	
DC8-73	162,389	48.1	1.35	120.1	122.8	125.6	128.3	68.2	74.4	81	88.1	Dual Tandem
	75,388			49.4	50.6	52.1	53.6	20.1	24	29.9	35.7	
DC8-73F	162,389	48.1	1.35	120.1	122.8	125.6	128.3	68.2	74.4	81	88.1	Dual Tandem
	67,677			43.7	44.8	46.1	47.4	17.5	19.9	25.6	31.2	
DC9-15	41,504	46.4	0.9	23.4	25	26.4	27.6	20.9	22.4	25.8	28.5	Dual
	22,300			11.1	12	12.8	13.5	9.9	10.6	11.7	13.9	
DC9-15F	41,504	46.4	0.9	23.4	25	26.4	27.6	20.9	22.4	25.8	28.5	Dual
	24,131			12.2	13.1	14	14.8	10.8	11.6	12.9	15.3	
DC9-21	41,813	47.4	0.99	24.3	25.9	27.3	28.5	21.6	22.9	26.3	29.2	Dual
	23,879			12.4	13.3	14.2	14.9	11.2	11.7	12.9	15.2	
DC9-32	49,442	46.2	1.07	29.5	31.2	32.7	34	26.3	27.7	31.4	34.2	Dual
	25,789			13.5	14.5	15.3	16.1	12.1	12.5	13.8	16.2	
DC9-41	52,163	46.8	1.1	31.7	33.5	35.1	36.5	28.3	29.7	33.7	36.6	Dual
	27,821			14.9	15.9	16.9	17.7	13.4	13.8	15.3	17.8	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
DC9-51	55,338	47	1.19	35	36.8	38.4	39.8	30.6	32.4	36.5	39.3	Dual
	29,336			16.4	17.4	18.3	19.2	14.5	14.9	16.4	19.1	
Dominie T Mk 1	9,662	44.5	0.7	4.5	4.9	5.2	5.4	3.7	4.2	4.7	5.7	Dual
	5,171			2.2	2.3	2.5	2.6	1.8	2.1	2.2	2.6	
E3	147,418	47.8	1.34	41.2	49.1	57.8	65.7	40.6	45.3	54.7	69.9	Dual Tandem
	76,022			17.5	19.9	23.3	26.9	17.7	18.8	21.1	27.6	
EMB120	12,070	47.5	1	10.7	10.8	10.8	10.8	10.7	11	11	11.2	Single
	0			0	0	0	0	0	0	0	0	
Embraer 170	35,990	47.5	1	31.6	31.9	32.1	32.3	31.8	32.9	32.8	33.5	Single
	20,940			18.4	18.6	18.7	18.8	18.5	19.1	19.1	19.5	
Embraer 175	37,500	47.5	1	32.9	33.2	33.5	33.7	33.1	34.2	34.1	34.9	Single
	21,810			19.2	19.4	19.5	19.6	19.3	19.9	19.9	20.3	
Embraer 190	47,790	47.5	1	41.8	42.2	42.6	42.9	42.2	43.6	43.5	44.5	Single
	28,080			24.7	24.9	25.1	25.2	24.8	25.6	25.6	26.1	
Embraer 195	50,790	47.5	1	44.4	44.8	45.2	45.6	44.9	46.4	46.2	47.3	Single
	28,970			25.4	25.7	25.9	26	25.6	26.4	26.4	27	
ERJ135	20,100	47.5	1	17.7	17.9	18	18.1	17.8	18.3	18.3	18.7	Single

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	11,501			10.2	10.2	10.3	10.3	10.2	10.5	10.5	10.7	
ERJ140	21,200	47.5	1	18.7	18.8	19	19.1	18.7	19.4	19.3	19.7	Single
	11,808			10.5	10.5	10.6	10.6	10.4	10.8	10.7	11	
ERJ145	22,100	47.5	1	19.5	19.6	19.8	19.9	19.5	20.2	20.1	20.6	Single
	12,114			10.7	10.8	10.9	10.9	10.7	11.1	11	11.3	
F100	44,680	47.8	0.98	27.7	29.3	30.8	31.9	24.7	26.4	29.8	32.2	Dual
	24,375			13.4	14.3	15.1	15.9	11.8	12.4	13.9	16.3	
F111	45,359	45	1.24	40.7	40.7	40.8	40.8	41	40.1	40.4	40.4	Dual
	22,135			19.9	19.9	19.9	19.9	20	19.6	19.7	19.7	
F15A/B	25,401	43.5	1.79	24.7	24.5	24.2	24	23.5	22.8	22.5	22.2	Single
	12,474			12.1	11.9	11.8	11.7	11.5	11.2	11.1	10.9	
F15C/D	30,844	43.5	2.34	32.5	31.9	31.4	30.9	29.7	28.7	27.7	27.1	Single
	12,791			13.3	13.1	12.9	12.7	12.3	11.9	11.5	11.3	
F15E	36,741	43.5	2.1	37.7	37	36.5	36	34.8	33.8	32.9	32.3	Single
	14,379			14.6	14.3	14.2	14	13.6	13.2	12.9	12.6	
F16A	16,057	41.7	1.9	15.2	15	14.8	14.7	14.4	13.9	13.7	13.5	Single
	7,348			6.9	6.8	6.8	6.6	6.6	6.4	6.3	6.2	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
F16B	16,057	41.7	1.9	15.2	15	14.8	14.7	14.4	13.9	13.7	13.5	Single
	7,666			7.2	7.1	7	6.9	6.9	6.7	6.5	6.4	
F16C	17,101	41.7	1.9	16.2	16	15.8	15.6	15.4	14.9	14.6	14.3	Single
	7,620			7.2	7.1	7	6.8	6.8	6.6	6.5	6.4	
F16D	17,101	41.7	1.97	16.3	16.1	15.9	15.7	15.4	15	14.6	14.4	Single
	7,893			7.5	7.4	7.3	7.1	7.1	6.9	6.7	6.6	
F27 Mk 500	19,777	47.5	0.54	9.9	10.9	11.7	12.3	7.9	9.8	11.6	13.4	Dual
	11,879			5.3	5.9	6.4	6.8	4.4	5.1	6	7.4	
F28 Mk 1000HTP	29,484	46.3	0.69	15	16.3	17.4	18.4	12.9	15	17.2	19.7	Dual
	16,550			7.5	8.2	8.9	9.4	6.4	7.4	8.2	10	
F28 Mk 1000LTP	29,484	46.3	0.59	14.3	15.7	16.9	17.9	11.7	14.6	16.9	19.6	Dual
	16,550			7.1	7.8	8.5	9.1	5.9	7.1	8.1	9.9	
F4	26,308	42.7	1.83	25.3	25	24.7	24.5	24	23.3	22.9	22.6	Single
	14,424			13.8	13.6	13.5	13.4	13.2	12.8	12.6	12.4	
F50	20,820	47.8	0.59	10	11	11.9	12.6	8.1	10	11.5	13.9	Dual
	12,520			5.4	6	6.6	7	4.5	5.3	6.1	7.5	
F5F	11,431	43	2.19	11.5	11.4	11.2	11	10.8	10.4	10.1	9.9	Single

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	4,763			4.8	4.7	4.6	4.4	4.5	4.3	4.2	4.1	
FB111A	51,846	45	1.48	49.5	49.2	48.9	48.7	48.8	47	46.9	46.5	Dual
	22,498			21.4	21.3	21.2	21.1	21.2	20.4	20.4	20.2	
Harrier GR Mk 3	11,475	42.7	0.83	8.6	8.7	8.8	8.8	8.3	9	9.2	9.5	Single
	5,700			4.3	4.3	4.4	4.3	4.1	4.5	4.6	4.7	
Harrier GR Mk 5	13,495	51.8	0.86	12.3	12.5	12.6	12.7	12.1	13	13.2	13.6	Single
	7,196			6.6	6.7	6.8	6.7	6.4	6.9	7	7.2	
Harrier T Mk 4	11,885	42.4	0.83	8.8	8.9	9	9.1	8.6	9.3	9.4	9.8	Single
	5,950			4.4	4.5	4.6	4.4	4.3	4.7	4.7	4.9	
Harrier T Mk 4A	11,885	42.4	0.83	8.8	8.9	9	9.1	8.6	9.3	9.4	9.8	Single
	6,240			4.7	4.7	4.8	4.7	4.5	4.9	4.9	5.1	
Harrier T Mk 4N	11,885	42.4	0.83	8.8	8.9	9	9.1	8.6	9.3	9.4	9.8	Single
	6,170			4.6	4.7	4.7	4.6	4.4	4.8	4.9	5.1	
Hawk T Mk 1	5,700	47	1.05	5.1	5.1	5.1	5	5	5.2	5.1	5.3	Single
	3,510			3.1	3.2	3.2	3	3.1	3.2	3.2	3.2	
Hercules C Mk 1	70,760	48	0.76	30.9	33.4	36.1	38.7	27.9	31	33.2	38.5	Not covered
	34,632			14.3	15.1	16.1	17.2	12.8	14.2	15	16.6	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
Hercules C Mk 3	73,028	48.4	0.76	32.3	35	37.8	40.5	29.2	32.5	34.7	40.4	Not covered
	36,623			15.3	16.2	17.3	18.4	13.7	15.3	16	17.9	
HS125-400	10,600	45.5	0.77	5.9	6.3	6.6	6.8	4.8	5.4	6.2	7	Dual
	5,683			2.8	3	3.2	3.2	2.3	2.6	2.8	3.4	
HS125-600	11,340	45.5	0.83	6.5	6.9	7.2	7.5	5.4	5.9	6.8	7.6	Dual
	5,683			2.9	3.1	3.3	3.3	2.4	2.6	2.8	3.4	
HS748	21,092	43.6	0.59	9.6	10.5	11.3	11.9	7.6	9.5	10.9	13	Dual
	12,183			4.9	5.4	5.9	6.3	4.1	4.8	5.5	6.7	
IL62	162,600	47	1.08	41.6	50.1	59.7	68.6	44	49.9	59.1	73.1	Dual Tandem
	66,400			15.9	15.2	17.5	20.4	14.4	15	17	21.6	
IL76T	171,000	23.5	0.64	22.9	24.1	23.4	24.2	7.3	8.3	9.9	12.9	Dual Tandem
	83,800			10.2	10.8	11.3	11.4	2.9	3.4	3.7	4.7	
IL86	209,500	31.2	0.88	25.6	30.6	37.9	45.7	31.3	33.3	39.7	55.5	Dual Tandem
	111,000			13.2	14.1	16.2	19.2	14.4	15.3	17.1	21.1	
Jaguar GR Mk 1	15,700	45	0.82	8.5	9	9.5	9.9	7	7.8	9	10.2	Dual
	7,424			3.5	3.8	4	4.1	3	3.2	3.5	4.2	
Jaguar T Mk 2	15,700	45	0.82	8.5	9	9.5	9.9	7	7.8	9	10.2	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	7,424			3.5	3.8	4	4.1	3	3.2	3.5	4.2	
Jetstream 31	6,650	45.6	0.23	2.9	3.3	3.5	3.7	1.6	2.3	3.5	4.9	Single
	4,015			1.9	2	2.2	2.2	1	1.4	2.1	2.9	
Jetstream T Mk 1	5,700	43.8	0.38	3.2	3.4	3.6	3.6	2.3	3	3.9	4.4	Single
	4,184			2.4	2.5	2.6	2.6	1.7	2.2	2.9	3.2	
Jetstream T Mk 2	6,000	44.3	0.38	3.4	3.6	3.8	3.8	2.4	3.2	4.2	4.7	Single
	4,473			2.6	2.7	2.8	2.8	1.8	2.4	3.1	3.5	
Jetstream T Mk 3	6,950	45.5	0.54	4.7	4.9	5	5	4	4.8	5.6	5.9	Single
	4,241			2.9	3	3.1	3	2.4	2.9	3.4	3.6	
KC10A	267,620	37.5	1.31	49.6	58.6	70.3	82	53.6	58.6	69.3	95.2	Dual Tandem
	108,862			18.2	19	21.3	24.5	18.2	18.9	20.6	25	
KC135A	136,804	46.8	1.07	31.2	37.6	45.4	52.9	33.7	37.1	44.9	58.6	Dual Tandem
	47,310			10.7	10.1	11	12.7	9.6	9.7	10.7	12.9	
KC135E	136,804	46.8	1.07	31.2	37.6	45.4	52.9	33.7	37.1	44.9	58.6	Dual Tandem
	50,893			11.6	10.9	12	13.9	10.5	10.6	11.8	14.3	
KC135R	136,804	46.8	1.07	31.2	37.6	45.4	52.9	33.7	37.1	44.9	58.6	Dual Tandem
	50,893			11.6	10.9	12	13.9	10.5	10.6	11.8	14.3	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
L1011-1	195,952	47.4	1.33	44.3	51.7	61.7	72.2	48.2	52	60.5	82.7	Dual Tandem
	108,862			23.6	24.7	28.2	32.7	23.7	24.7	27.4	34.2	
L1011-100 & 200	212,281	46.8	1.21	46.1	54.7	66	77.6	51.7	56.5	66.6	91	Dual Tandem
	110,986			23.1	24	27.6	32.2	23.7	24.4	27.5	34.5	
L1011-500	225,889	46.2	1.27	50.3	59.5	71.7	83.9	55.4	60.6	71.8	97.5	Dual Tandem
	108,924			22.8	23.5	26.9	31.3	23	23.9	26.5	32.9	
MD10-10F	200,942	46.8	1.07	41.3	49.6	60.7	72	48.3	52.1	62.5	86.9	Dual Tandem
	97,967			18.9	20	23	27	20.6	21.1	23.5	29.3	
MD10-30F	264,444	38	1.21	51.3	57	68.9	80.8	53.1	58.3	69.3	95.3	Dual Tandem
	107,275			17.6	19	20.7	23.9	18.1	18.6	20.4	24.9	
MD11	274,500	39.5	1.42	56.7	67	79.9	92.5	59.5	65.4	78.1	105.8	Dual Tandem
	128,808			23.6	25.3	28.8	33.3	23.8	24.9	27.3	34.3	
MD11 ER	287,122	39.5	1.42	60.2	71.3	85.1	98.3	63	69.5	83.9	112.4	Dual Tandem
	132,049			24.2	26	29.7	34.4	24.6	25.6	28.2	35.6	
MD11 Freighter	274,655	39.5	1.42	56.8	67.1	80	92.5	59.6	65.5	78.2	105.9	Dual Tandem
	112,748			20.6	21.5	24.3	27.9	20.3	21.2	23.1	28.3	
MD81	63,958	47.8	1.17	40.7	42.9	44.9	46.5	36	38	42.9	46.2	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	35,330			20	21.2	22.4	23.5	17.9	18.4	20.3	23.6	
MD82	68,266	47.5	1.24	44.4	46.6	48.6	50.2	38.9	41.5	46.2	49.3	Dual
	35,369			20.2	21.4	22.6	23.6	17.9	18.4	20.3	23.5	
MD83	73,028	47.4	1.34	49.1	51.3	53.2	54.8	42.4	46	50	53.1	Dual
	36,145			21.1	22.3	23.5	24.4	18.3	18.9	20.8	24.1	
MD87	63,957	47.4	1.17	40.2	42.4	44.4	46	35.7	37.5	42.4	45.8	Dual
	50,802			30.5	32.3	33.9	35.3	27.1	28.2	31.9	35.5	
MD87 Auxillary Tank	68,266	47.4	1.24	44.3	46.5	48.5	50.1	38.8	41.4	46.1	49.2	Dual
	50,802			31	32.7	34.3	35.6	27.1	28.2	32.1	35.6	
MD88	68,266	47.5	1.24	44.4	46.6	48.6	50.2	38.9	41.5	46.2	49.3	Dual
	35,369			20.2	21.4	22.6	23.6	17.9	18.4	20.3	23.5	
MD90-30	71,214	48.2	1.31	48.3	50.5	52.5	54.1	42	45.3	49.5	52.6	Dual
	39,994			24.1	25.5	26.8	27.9	20.9	21.7	24.2	27.6	
MD90-30ER	76,430	47	1.33	51.2	53.5	55.5	57.2	44.4	48.2	52.2	55.3	Dual
	40,396			23.8	25.1	26.4	27.5	20.5	21.5	23.7	27.1	
Nimrod MR Mk 1	80,513	47.6	1.31	29.8	34	38.2	41.8	26.7	29.3	32.3	38.2	Dual Tandem
	39,281			11.2	12.6	14.4	16.2	10.1	11	12.8	15.2	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
Nimrod MR Mk 2	83,461	47.6	1.4	31.9	36.2	40.4	44.1	28.4	30.7	33.9	40.1	Dual Tandem
	41,458			12.2	13.8	15.7	17.6	10.9	11.9	13.7	16.3	
Nimrod MRA4	106,737	47.5	1.7	46.4	52.1	57.4	61.8	38.7	42.6	48.9	56.9	Dual Tandem
	52,487			17	19.4	22.1	24.6	14.3	15.6	18.4	23	
Nimrod R Mk 1	80,513	47.6	1.4	30.3	34.4	38.5	42.1	27	29.3	32.4	38.2	Dual Tandem
	42,410			12.6	14.2	16.2	18.1	11.2	12.3	14.2	16.7	
Pembroke C (PR) Mk 1	6,124	44	0.38	3.5	3.7	3.8	3.8	2.5	3.3	4.2	4.8	Dual
	4,637			2.7	2.8	2.9	2.9	1.9	2.5	3.2	3.6	
Pembroke C Mk 1	6,124	44	0.38	3.5	3.7	3.8	3.8	2.5	3.3	4.2	4.8	Dual
	4,616			2.7	2.8	2.9	2.9	1.9	2.5	3.2	3.6	
Phantom F4J	25,579	43.8	2.07	26.2	25.8	25.4	25.1	24.4	23.7	23	22.6	Single
	14,286			14.5	14.3	14.1	13.9	13.6	13.2	12.9	12.6	
Phantom FG Mk 1	27,397	43.8	2.07	28.1	27.6	27.2	26.9	26.1	25.3	24.7	24.2	Single
	14,603			14.8	14.6	14.4	14.2	13.9	13.5	13.1	12.9	
Phantom FGR Mk 2	27,397	43.6	2.07	27.9	27.5	27.1	26.8	26	25.2	24.6	24.1	Single
	14,603			14.8	14.6	14.4	14.2	13.8	13.4	13.1	12.8	
Puma HC Mk 1	7,000	38	0.6	4.2	4.3	4.4	4.3	3.7	4.3	4.8	5	Dual

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
	3,700			2.2	2.3	2.3	2.2	1.9	2.3	2.5	2.6	
Sea Harrier FRS Mk 1	11,885	42.4	0.83	8.8	8.9	9	9.1	8.6	9.3	9.4	9.8	Single
	5,940			4.4	4.5	4.6	4.4	4.3	4.6	4.7	4.9	
Sentry AEW Mk 1	151,954	47	1.31	42	50.2	59.1	67.1	41.5	46.5	56.1	71.4	Dual Tandem
	83,915			19.3	22.1	26	30.2	19.5	20.9	23.8	31.3	
Test	610,000	19	1.5	61.3	71.8	86.4	107.1	64.1	70	85.2	118.7	Dual Tandem
	250,826			22.9	23.3	26.1	29.9	21.9	22.8	24.7	30.4	
Tornado F Mk 2	26,600	44.5	2	27.4	27	26.6	26.3	25.7	24.9	24.3	23.9	Single
	14,231			14.6	14.3	14.2	14	13.7	13.3	13	12.8	
Tornado GR Mk 1	28,584	47.3	2.17	32.1	31.5	31.1	30.7	29.6	28.7	27.9	27.3	Single
	13,747			15.3	15	14.8	14.6	14.2	13.8	13.4	13.1	
Trident 1E	61,160	46	1.03	31.8	34.2	36.5	38.4	20.8	22.6	25.1	29.6	Dual
	33,203			14	15.4	16.8	18	9.1	10.1	11.4	13.6	
Trident 2E	65,998	47	1.07	36.5	39.1	41.5	43.4	23.7	25.7	28.5	33.3	Dual
	33,980			15	16.5	17.9	19.1	9.7	10.8	12.1	14.4	
Trident 3	68,266	45.5	1.14	37	39.6	41.9	43.8	23.9	25.8	28.6	33.4	Dual
	39,060			17.6	19.1	20.7	22	11.3	12.5	14	16.5	

Aircraft type	All Up Mass (kg)	Load on one Main Wheel Gear leg (%)	Tyre Pressure (MPa)	RIGID PAVEMENT SUBGRADES - MN/m^3				FLEXIBLE PAVEMENT SUBGRADES - CBR				Main Wheel Gear Type for Pavement Design
				High 150	Medium 80	Low 40	Ultra Low 20	High 15	Medium 10	Low 6	Ultra Low 3	
				ACN				ACN				
1	2	3	4	5	6	7	8	9	10	11	12	13
Tristar K Mk 1	245,850	45.7	1.43	60.5	71.7	85.3	98.2	63.1	69.6	84	110	Dual Tandem
	109,550			24	25.1	28.6	33	23.5	24.6	27.1	34.1	
TU134A	47,600	45.6	0.83	10.3	12.6	15.2	17.6	10.6	11.6	14	18.4	Dual Tandem
	29,350			5.6	6.5	7.8	9.2	5.8	6.2	7.2	9.3	
TU154B	98,000	45.1	0.93	17.5	23.8	30.4	36.2	15.2	17.2	21.7	28.9	Tridem
	53,500			7	8.8	11.7	14.7	6.9	7.3	8.7	12	
Typhoon	23,400	47.5	2.31	26.8	26.3	25.8	25.5	24.5	23.7	23	22.5	Single
	16,600			18.9	18.6	18.3	18	17.4	16.8	16.3	15.9	
VC10 C Mk 1	147,417	47.5	0.97	34.6	42.4	51.5	59.8	38.5	42.2	52.1	66.5	Dual Tandem
	67,630			14.8	14.9	17.5	20.6	14.5	15.2	17.4	22.4	
VC10 K Mk 2	143,334	45.5	0.95	31.1	38	46.3	54.1	35.2	38.1	47	60.3	Dual Tandem
	68,000			14.2	14.1	16.5	19.5	13.8	14.5	16.5	21.2	
VC10 K Mk 3	152,860	46.6	1.02	36.2	44.2	53.3	61.8	39.3	43.7	53.5	68	Dual Tandem
	71,000			15.5	15.8	18.5	21.7	15.4	15.8	18.1	23.4	
VC10-1150	151,953	48.3	1.01	37.6	46	55.5	64.2	40.8	45.6	55.8	70.8	Dual Tandem
	71,940			16.1	16.7	19.7	23.2	16.3	16.8	19.3	25	

Appendix C Defence Estates' Specification for Airfield Pavement Works

C1. GENERAL

C1.1 Defence Estates' own Specification for airfield pavement works includes a number of materials which have been developed over a long period to give good performance in airfield pavement applications. Throughout the text these materials are referred to by Defence Estates' own terminology e.g. Pavement Quality Concrete, Drylean Concrete, Marshall Asphalt, Porous Friction Course, etc.

C1.2 It is important that pavement materials are at least comparable to Defence Estates'[54,55,56,57,58], to ensure that adequate structural capacity, durability, and tyre pressure limitations are achieved. If this is not the case the structural designs (Charts 1 to 8), recommended design lives (para 4.7.2) and allowable tyre pressures (Table 13) will no longer apply.

C1.3 The following sections give brief outlines of the necessary material qualities.

C2. PAVEMENT QUALITY CONCRETE

C2.1 Pavement Quality Concrete is a concrete containing Portland Cement (CEM1), which should have the following properties.

(i) An approved sound aggregate, free of deleterious materials.
(ii) Air entrainment when frost damage is a possibility.
(iii) A suitable mixture design to ensure that it can be laid and compacted to provide a strong and durable concrete slab with a hard wearing surface free from loose materials and sharp edges which might endanger airacraft and which also gives adequate friction and drainage characteristics.

C3. DRYLEAN CONCRETE (DLC)

C3.1 DLC is a lean concrete with a low water content. The minimum requirements for four types of DLC associated with rigid and flexible pavements in this guide are:

1) Type R DLC - used as the base in rigid pavements. The maximum aggregate to cement ratio is 15 to 1 with natural aggregates. Mean compressive strength from 7 days laboratory cubes must be at least 15 N/mm^2. The minimum in situ density should be at least 95% of a Job Standard Density set by trials.

2) Type FH DLC - used as a High Strength Bound Base Material in flexible pavements, in conjunction with Chart 5. Type FH DLC should have a maximum aggregate to cement ratio of 23:1 and the characteristic (5% defective) compressive strength from 7 day laboratory cubes should be at least 15 N/mm^2. The insitu density should be at least 95% of the Job Standard refusal density as determined from cubes in the site trials. The All-in aggregate grading should comply with Table 22.

Table 22 Drylean Concrete aggregate grading requirements.

DRYLEAN CONCRETE AGGREGATE GRADING (All-in Aggregate)	
BS EN 933-2 Sieves (mm)	**Percentage by mass passing**
63	100
31.5	70-92
16	40-74
4	22-46
0.500	6-27
0.250	2-15
0.125	0

3) Type F DLC - used as a Bound Base Material in flexible pavements, in conjunction with Chart 7. The mean compressive strength from 7 day laboratory cubes should be at least 8 N/mm^2, the maximum strength 15 N/mm^2 and the minimum 4 N/mm^2. The minimum in situ density should be at least 95% of a Job Standard Density set by trials.

4) Type W DLC - used as a working course without controls on strength and density.

C3.2 The water content should be between 5 and 7% by weight of dry materials, the final value being selected to give the maximum dry density.

C3.3 Unless used as a working course DLC should be rolled to give the maximum possible density.

C3.4 Cement is the preferred binder but other hydraulic binders may be used provided the requirements above are met.

C4. MARSHALL ASPHALT

C4.1 Marshall Asphalt is a continuously graded asphalt concrete with a bituminous binder. 60% of the material passing the 0.063 mm sieve should be added filler. They should have an approved sound and durable aggregate and are designed using the Marshall Method to give a high stability material. Test requirements for trial mixes and during laying are summarised in Table 23, with the requirements for minimum stability set out in Table 24.

C4.2 Marshall surfacing is a similar material but using a tar binder, giving it better fuel resistance but with the mechanical properties changing more with temperature. However, tar is no longer used because of its carcinogenic properties that were not previously appreciated.

C4.3 From 2008 when the harmonised European Standards for asphalt are fully implemented, Marshall Asphalt will have to comply with BS EN 13108-1.

C5. COATED MACADAM

C5.1 Coated Macadam is an asphalt concrete with a bituminous binder that has not been designed using the Marshall Method but is of a quality at least as good as a dense mixture to BS 4987: Part 1: 2005 or an equivalent specification.

C5.2 From 2008 when the harmonised European Standards for asphalt are fully implemented, BS 4987: Part 1 will be superceded by BS EN 13108-1 and Coated Macadam will have to comply with that standard.

C6. HOT ROLLED ASPHALT

C6.1 Hot Rolled Asphalt is a gap-graded asphalt mixture consisting of a binder/fine aggregate/filler mortar in which the coarse aggregate sits with a quality at least as good as BS 594: Part 1: 2005 or an equivalent specification.

C6.2 From 2008 when the harmonised European Standards for asphalt are fully implemented, BS 594: Part 1 will be superceded by BS EN 13108-4 and Hot Rolled Asphalt will have to comply with that standard.

C7. STONE MASTIC ASPHALT

C7.1 Stone Mastic Asphalt is a gap-graded asphalt mixture with coarse aggregate to coarse aggregate interlock and a bituminous binder.

C7.2 From 2008 when the harmonised European Standards for asphalt are fully implemented, Stone Mastic Asphalt will have to comply with BS EN 12697-5.

C8. POROUS FRICTION COURSE

C8.1 Porous Friction Course is a porous asphalt to the grading of Table 39 of BS 4987: Part 1: 2005 specifically developed by Defence Estates to allow free drainage of water from beneath a tyre contact area. The specification is tightly controlled to ensure adequate durability, and is summarised in Table 25.

C8.2 From 2008 when the harmonised European Standards for asphalt are fully implemented, Porous Friction Course will have to comply with BS EN 13108-7.

C9. SLURRY SEAL

C9.1 Slurry seal should be to BS 434: Part 2: 1984 or an equivalent with the following thicknesses and aggregate sizes:

(i) Standard slurry seal: Average 1.5 mm, not greater than 3 mm. Aggregate 0/1 mm.
(ii) Coarse slurry seal: Average 2.5 mm, not greater than 5 mm. Aggregate 0/4 mm.

C9.2 A tack coat should be used before laying and the material should be rolled with a rubber-tyred roller.

C9.3 When the harmonised European Standards for slurry surfacing are fully implemented, slurry seal will have to comply with BS EN 12273.

C10. GRANULAR BASE

C10.1 An approved, hard, durable angular rock of blast furnace slag, clean and free from dust and blended with sand or approved fine screenings. Gradings and test requirements are summarised in Table 26.

C11. GRANULAR SUB-BASE

C11.1 An approved durable granular material such as gravel, hard clinker, crushed rock or well burnt colliery shale. If necessary blended with sand or other approved fine screenings. Gradings and test requirements are summarised in Table 26.

C12. CONCRETE BLOCK PAVING

C12.1 Concrete Block Paving consists of modular units which are placed on a laying course material. A joint filling material is spread over the block paving and the pavement is vibrated so that the units bed into the laying course material and the joint filling material is distributed between the units. The blocks are manufactured to the requirements of BS EN 1338.

Table 23 Marshall Asphalt – Test Requirements

TRIAL MIXTURES			
REQUIREMENT	BINDER COURSE	SURFACE COURSE	REGULATING COURSE
Optimum binder content	Between 4.0 and 7.0 per cent	Between 5.0 and 7.0 per cent	Between 5.0 and 7.0 per cent
Stability	Not less than 6 kN to 10 kN †	Not less than 6 kN to 10 kN †	Not less than 6 kN to 10 kN †
Flow	Note more than 4.00mm	Not more than 4.00mm	Not more than 4.00mm
Voids total mixture	Between 3 and 5 per cent.	Between 3 and 4 per cent	Between 3 and 5 per cent.
Voids filled with binder	Between 67 and 77 per cent	Between 76 and 82 per cent	–
VARIATIONS FROM THE AGREED MIX DESIGN DURING LAYING			
REQUIREMENT	BINDER COURSE	SURFACE COURSE	REGULATING COURSE
Dry Aggregate/Filler grading Passing 6.3 mm sieve, or larger Passing 2 mm, 1 mm, 0.5 mm, 0.125 mm sieves Passing 0.063 mm sieve	 ±5 per cent ±4 per cent ±1.5 per cent	 ±4 per cent ±3 per cent ±1.5 per cent	 ±5 per cent ±4 per cent ±1.5 per cent
Binder content * Voids total mixture Voids filled with binder Stability Flow	±0.3 per cent ±1 per cent ±5 per cent Not less than 90 % of the design limit with not less than 90 % of the result less than the design limit Not more than 4.00mm		
Notes: † Limit dependent on tyre pressure and frequency according to Table 24 * For Binder Course, not less than 4% For Surface Course, not less than 5 %			

Table 24 Minimum stability requirements for Marshall Asphalt

Tyre Pressure MPa (psi)	Minimum Stability (kN)		
	Frequency of Trafficking		
	Low	Medium	High
Up to 1.4 (200)	6	8 (6)	10 (8)
More than 1.4 (200)	8	10	10

() bracketed values may be specified for cooler regions of the UK.

Table 25 Porous Friction Course

COARSE AGGREGATE	PROPERTY	CATEGORY	TEST
	Resistance to freezing and thawing	MS_{18} / MS_{30} *	BS EN 1367-1
	Shape	FI_{20}	BS EN 933-3
	Resistance to fragmentation	LA_{15}	BS EN 1097-2
	Water absorption	$WA_{24}1$	BS EN 1097-6
	Affinity between aggregate and binder	≤3 particles from 150	BS EN 12697-11 Part B
	Fines content	f_1	BS EN 933-1
	Resistance to polishing	PSV_{50} or $PSV_{declared\ 55}$	BS EN 1097-8
BINDER	160/220 pen paving grade bitumen		
FILLER	1.5% to 2.0% by mass hydrated lime (plus crushed limestone only where additional material passing the 0.063 mm sieve is required).		
BINDER CONTENT	Percentage by mass of soluble binder shall not be less than 5.2 %		
TEMPERATURES: MIXING	Aggregate	Max. 120 ºC	
	Binder	95 ºC – 135 ºC	
	Mixture at discharge	100 ºC – 120 ºC	
LAYING	Not less than 100 ºC		
ROLLING	Not less than 65 ºC		
COMPACTED THICKNESS	20mm nominal (Tolerance +6 mm and –0 mm)		
TACK COAT	Bitumen emulsion (BS 434), Anionic Class A1-40 , Cationic Class K1-40 or Cationic Class K1-70 at rate in BS 4987: Part 2.		
SURFACE ACCURACY	3 mm in 3 m (in any direction)		
NOTES.	General This mixture is for use on runways only (excluding runway ends). It allows the free penetration of surface water to the underlying layer, which must be densely graded impervious surface course of high stability. It should be of uniform compacted thickness throughout and is not suitable over deformed or poorly shaped surfaces. * Category for source / each fraction.		

Table 26 Unbound Granular Materials

BS Sieves (mm)	Granular Base Course (1)(2)(3)(5)	Granular Sub-base (1)(3)(4)(6)		
	Percentage by Mass Passing			
80	-	100	-	-
63	100	95 – 100	-	-
40	98 - 100	82 – 100	100	-
20	60 – 100	60 – 80	80 – 100	100
10	40 – 60	45 – 65	55 – 80	80 – 100
4	20 – 35	27 – 47	37 – 56	45 - 70
2	13 – 28	18 – 39	28 – 45	30 - 55
1	9 – 21	13 – 32	20 – 35	20 - 42
0.500	7 - 19	9 – 27	13 – 28	12 – 32
0.063	3 - 8	0 -9	0 - 9	0 -9

NOTES

1. Not less than 10% is to be retained between each pair of successive sieves (excepting the largest and smallest pair)
2. The percentage passing the 0.500mm sieve shall not be less than twice the percentage passing the 0.063mm sieve.
3. The material passing the 0.500mm sieve shall have a Liquid Limit ≤ 25% and a Plasticity Index ≤ 6 to BS 1377.
4. The through grading of the material is to give a smooth curve throughout the entire range of sieves.
5. Los Angeles coefficient category LA_{40}
6. Los Angeles coefficient category LA_{50}

Appendix D Aircraft Main Wheel Gear Arrangements

D1 Most aircraft have a tricycle type undercarriage arrangement comprising two main wheel gears near the centre of gravity of the aircraft and a nose wheel gear. As the load on the nose gear is usually small (5%-10% of the total load) it is rarely the critical pavement loading case.

D2 The number of wheels on the main gears varies with the aircraft type, generally increasing as the weight of the weight of the aircraft increases. The standard main wheel gear arrangements are illustrated in Figure 33. Tandem gears are not very common and for practical design purposes can be treated as duals when using the charts. Other main gears with 4 or more wheels are shown in Figure 33; for practical design purposes these can be treated as dual-tandem or tridem.

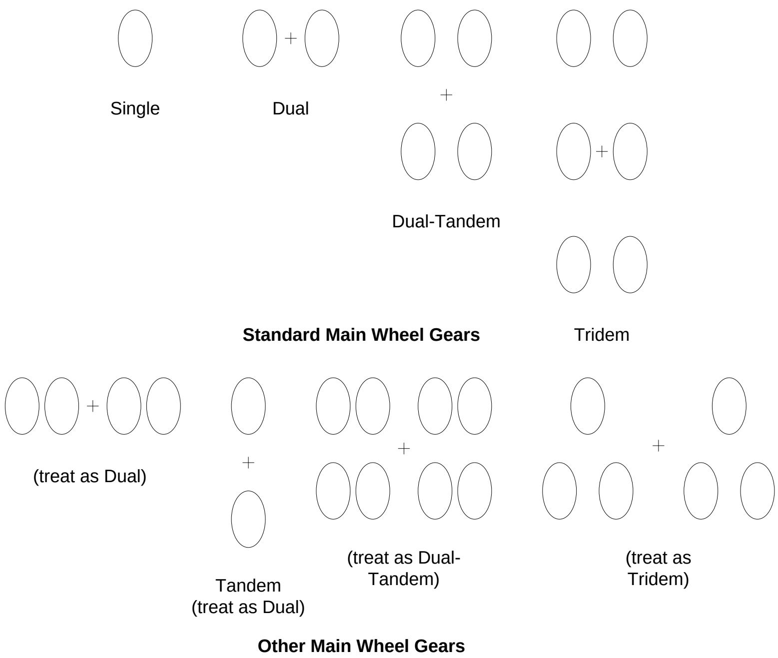


Figure 33 Main wheel gear types

Appendix E Pass-to-Coverage Ratio

E1 The method of assessing Pass-to-Coverage Ratios is that developed by the US Army Corps of Engineers[12]. Aircraft movement is usually controlled by runway and taxiway centre-lines, with the greatest concentration around the centre-line. Because of the large number of passes for which a pavement is designed, traffic can be considered to be normally distributed either side of the centre-line. Experimental research[31] shows agreement with this theory.

E2 The distribution of aircraft traffic on runways and taxiways can be represented by a General Normal Distribution (GND) Curve, plotting frequency of aircraft passes at given distances from the centre-line. Whilst the aircraft centre-line represents the position of the aircraft, the same curve represents the distribution of any of the aircraft tyres. From this curve two important parameters are defined:

a) Wander is defined as the width over which the centre-line of the aircraft (or a tyre) traffic is distributed for 75% of the time. The findings of Reference 31 demonstrated that a Wander of 1.778 m (70 in) was applicable to taxiways with a centreline. However, evidence shows that the deviation for modern aircraft movements is significantly less. This guide uses a Wander of 1.788 m for runways and 0.894 m for taxiways. For stands with a stand centreline very little wander occurs, and a Wander of 0 m is used for the Pass-to-Coverage Ratio on stands.
b) A Coverage is defined as the application of the maximum stress at a point in a pavement. For flexible pavements this is a point in the pavement surface and for rigid pavements a point on the underside of the concrete slab.

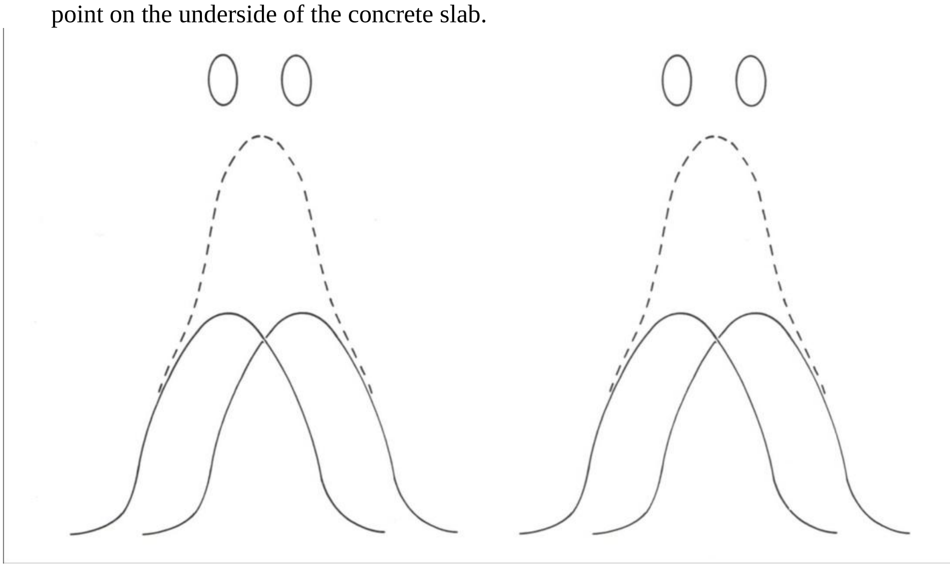


Figure 34 Distribution curves for a dual main wheel gear

E3 If the pavement is divided into strips with a width equal to that of the aircraft tyre and the GND Curve of passes against distance from centre-line position is plotted, the number of Coverages applied to each strip of pavement is obtained. The pavement has to be designed for the strip which is subject to the greatest number of Coverages. For example, if 100 passes are plotted and one strip takes 33 of them, the pavement is designed for 33 coverages, and the Pass-to-Coverage Ratio is $\frac{100}{33} = 3$

The Pass-to-Coverage Ratio (P/C) is given by the formula

$$\frac{\mathbf{P}}{\mathbf{C}} = \frac{\mathbf{1}}{\gamma \mathbf{W}_t}$$

where γ is a function of the GND Curve and $\mathbf{W}_t$ is the tyre width in metres.

For a single tyre the Pass-to-Coverage Ratio (P/C) is given by:

$$\frac{\mathbf{P}}{\mathbf{C}} = \frac{1.938}{\mathbf{W}_t} \text{ if the tyre width is in metres}$$

$$\text{or } \frac{\mathbf{P}}{\mathbf{C}} = \frac{76.285}{\mathbf{W}_t} \text{ if the tyre width is in inches.}$$

E4 If there is more than one wheel, the Pass-to-Coverage Ratio is obtained by summing the distribution curves. If the wheels are far enough apart the curves will not overlap and the Pass-to-Coverage Ratio will be the same as that for a single wheel.

E5 For dual wheels there is a considerable overlap. On flexible pavements tandem wheels which track each other give exactly twice the maximum ordinate as a single wheel. On rigid pavements the passage of two wheels tracking each other at the spacing found in dual-tandem and tandem main wheel gears causes two stress peaks, but not a complete stress reversal between the wheels. Therefore, the rear wheel should not have an effect on the Pass-to-Coverage Ratio on rigid pavements and the value for a dual-tandem or tandem on rigid pavements ought to be twice that on flexible pavements. However, to simplify matters this factor has been built into the relevant graphs so that the Pass-to-Coverage Ratios quoted in Chapter 4 apply to all pavement types.

E6 To simplify the procedure for calculating Pass-to-Coverage Ratios Figure 36 shows the relationship between γ and the distance from the point being considered to the centre-line of a wheel. For multiple wheel gear the values of γ are added and the maximum value selected for calculation of the Pass-to-Coverage Ratio. The procedure is illustrated in the following example.

Example

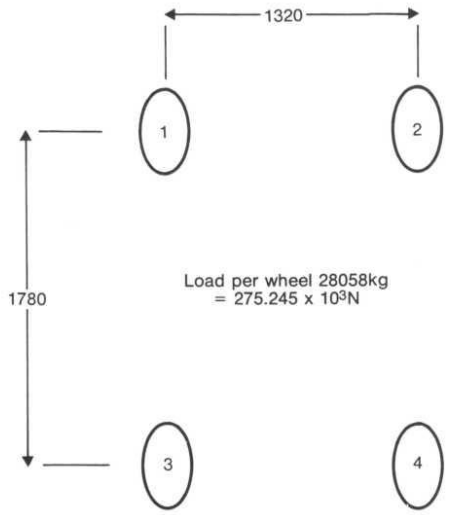


Figure 35 Example main wheel gear for pass-coverage ratio

Tyre Pressure = 1.43 N/mm^2

*Contact Area Width W_t = $0.878x\sqrt{\frac{275245}{1.43}} = 385mm$

*If the tyre width is not known it can be taken as
W_t=0.878√Tyre Contact Area
which is more accurate than assuming a circular contact area.

FUNCTION γ

Distance from Centre-line of Wheel 1 (mm)	Wheel 1	Wheel 2	Wheel 3	Wheel 4	TOTAL
0	0.516	0.120	0.516	0.120	1.272
100	0.512	0.149	0.512	0.149	1.322
200	0.499	0.180	0.499	0.180	1.358
300	0.479	0.216	0.479	0.216	1.390
400	0.451	0.254	0.451	0.254	1.410
500	0.412	0.294	0.412	0.294	1.412
600	0.382	0.334	0.385	0.334	1.432
660	0.357	0.357	0.357	0.357	1.428

Pass-to-Coverage Ratio = $\frac{\mathbf{1}}{\gamma W_t} = \frac{\mathbf{1}}{1.432x0.385}$ = 1.8

N.B. The distance to obtain γ for Wheel 2 is (1.32 – x_1) where x_1 is the distance from the centre-line of Wheel 1.

Figure 36 Graph of γ against the distance from the wheel centre-line

Appendix F The Pavement Design Models

F1 INTRODUCTION

F1.1 The design models used to produce the design and evaluation charts can be split into six parts as follows:

(i) The pavement behaviour and mode of failure.
(ii) The failure criteria.
(iii) The method of analysis.
(iv) The derivation of allowable stresses within the pavement and subgrade.
(v) The integration of the ACN-PCN method as the loading variable.
(vi) The pavement construction.

F1.2 To obtain the design thicknesses shown on the charts the pavement thickness is adjusted for each combination of ACN, subgrade strength and Frequency of Trafficking until the actual stresses in the pavement and/or subgrade equal the allowable values. (Figure 40 summarises this procedure for rigid pavements).

F1.3 The fundamental function of the pavement is to protect the subgrade by spreading the concentrated aircraft load. The guide covers two basic types of pavement:

(vii) Rigid, which spread the load by means of their high flexural stiffness. The principle layer of a rigid pavement is a concrete slab.
(viii) Flexible, which spread the load through the sheer strength of the pavement materials. Flexible pavements comprise bituminous surfacing on unbound granular material or bound material of low stiffness and/or flexural strength (bitumen-bound or weak cement-bound layers such as lean concrete).

F1.4 The pavement construction is assumed to be basically in accordance with Defence Estates' Specification, as described in Chapters 5 and 6 for rigid and flexible pavements respectively.

F2 RIGID PAVEMENTS

F2.1 Failure Mode

F2.1.1 The corollary of the high flexural stiffness of the concrete slab is that loading induces a high flexural stress in the slab. The failure mode is therefore cracking of the concrete. The basis of the rigid pavement design model is the principle that cracking of concrete can be controlled by limiting the flexural stress.

F2.2 Failure Criteria

F2.2.1 Corner or halving cracking on the surface of an unreinforced concrete slab is the first sign of structural failure (see Figure 37 and Figure 38); derivative forms of halving cracking include quartering and delta cracking (see Figure 39). Failure of an area of pavement is considered to be the point at which surface serviceability can no longer by realistically maintained by minor maintenance.

F2.2.2 The rate at which a failure condition is approached following the development of initial cracking depends on several factors:

(i) The level of support provided by the base and subgrade.
(ii) The form of failure. Corner cracking generally leads to a faster rate of deterioration than halving cracking. Small broken corners subject to repetitive loading can soon cause spalling, and over-stressing of the base and subgrade.
(iii) The frequency of trafficking and overload. High frequency of trafficking by the heaviest user aircraft will increase the rate of crack propagation. Overload movements will greatly accelerate this process (see Chapter 8).
(iv) The attention given to the sealing of cracks and the repair of edge spalls can considerably delay the need for rehabilitation.
(v) Freeze-thaw cycles will accelerate spalling at cracks.

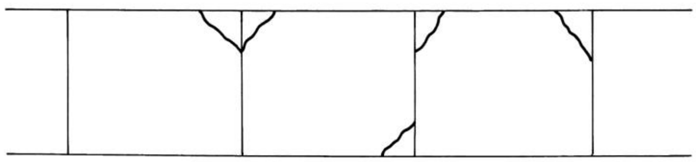


Figure 37 Corner cracking

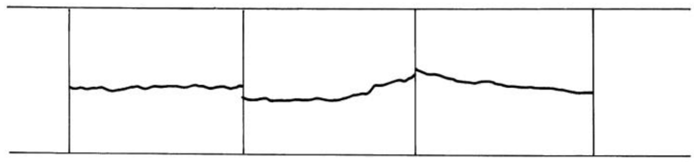


Figure 38 Halving cracking

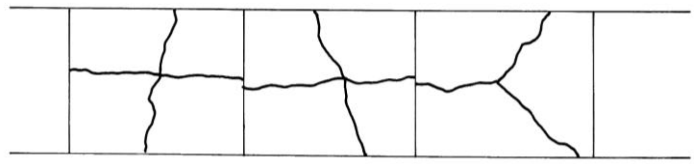


Figure 39 Quartering and Delta cracking

F2.2.3 The above factors clearly underline the difficulty of setting definitive failure criteria. The failure criterion used in this Guide is based on Defence Estates' experience of unreinforced rigid pavement performance. Failure of a rigid pavement is considered to have occurred when between a third and a half of the bays within the trafficked area have suffered cracking. Generally this would be corner, halving, quartering or delta cracking but some of the bays with weaker concrete or base support would have developed multiple cracking and some of these would have been replaced. (The replacement bays should be added to the cracked ones when counting the number of failed bays).

F2.2.4 The likely form of failure (i.e. corner or halving cracking) of the standard undowelled, unreinforced rigid pavement designs depends on the following factors:

(i) The pattern of trafficking in relation to bay layout. If wheel paths are channelised over the centre portions of bays, longitudinal halving cracking (i.e. in the direction of trafficking) is more likely. Similarly if wheel paths are concentrated along a line of bay corners, then corner cracking is more likely.
(ii) The larger by sizes normally adopted for thicker slabs tend to develop high centre warping stresses. In addition, high load transfer at the transverse joints of thick slabs substantially reduces the corner stresses. Halving cracking is therefore the most likely failure mode in thick slabs.
(iii) The critical aircraft operating on thick slabs generally incorporate multi-wheel main landing gear. Simple geometry shows that multi-wheel gears cannot easily be located at the extreme corner of a slab so as to induce a critical stress. The risk of corner cracking is therefore reduced.

F2.2.5 For fully dowelled PQC slabs (i.e. dowelled expansion, construction and contraction joints), longitudinal halving cracking in the wheel paths will usually be the failure mode. In the case of reinforced concrete the failure mode can either be longitudinal halving cracking in the wheel paths, or major spalling of the transverse cracks caused by shrinkage and warping.

F2.3 Method of Analysis

F2.3.1 To analyse the live load stresses in a rigid pavement the design model uses the Westergaard theory[32] for an elastic slab on a dense liquid subgrade. This assumes that at any point the reaction of the subgrade is directly proportional to the deflection of the slab. The constant of proportionality is the Modulus of Subgrade Reaction k.

F2.3.2 The actual solution used for stresses at the interior of a slab is that developed by Pickett, Raville, Jones and McCormack[33] and embodied in a computer program by Packard.[34] For corner case analysis the Pickett modification[35] of Westergaard's original equation is used, which allows for the loss of subgrade support caused by upward warping of the corner due to temperature differentials in the slab.

F2.3.3 The stress calculations are based on the static wheel load case. In line with the conclusions of References 35 and 36 no additional allowance is made for dynamic wheel load effects.

F2.3.4 The level of the load transfer at joints determines how critical corner stresses are compared to centre stresses. The design model incorporates a range of load transfers depending on factors such as slab thickness and subgrade support; and based on experience from pavement performance,[38,39] and plate bearing tests. Typical values are shown in Table 27.

Table 27 % Load Transfer at transverse joints in undowelled PQC in accordance with the Specification.

On a Strong Bound Base		On a Granular Base or Directly on the Subgrade	
Slab Thickness (mm)	% Load Transfer	Slab Thickness (mm)	% Load Transfer
150-225	15-20	150-225	5-10
>250	33	250-300	25-33
		>300	33

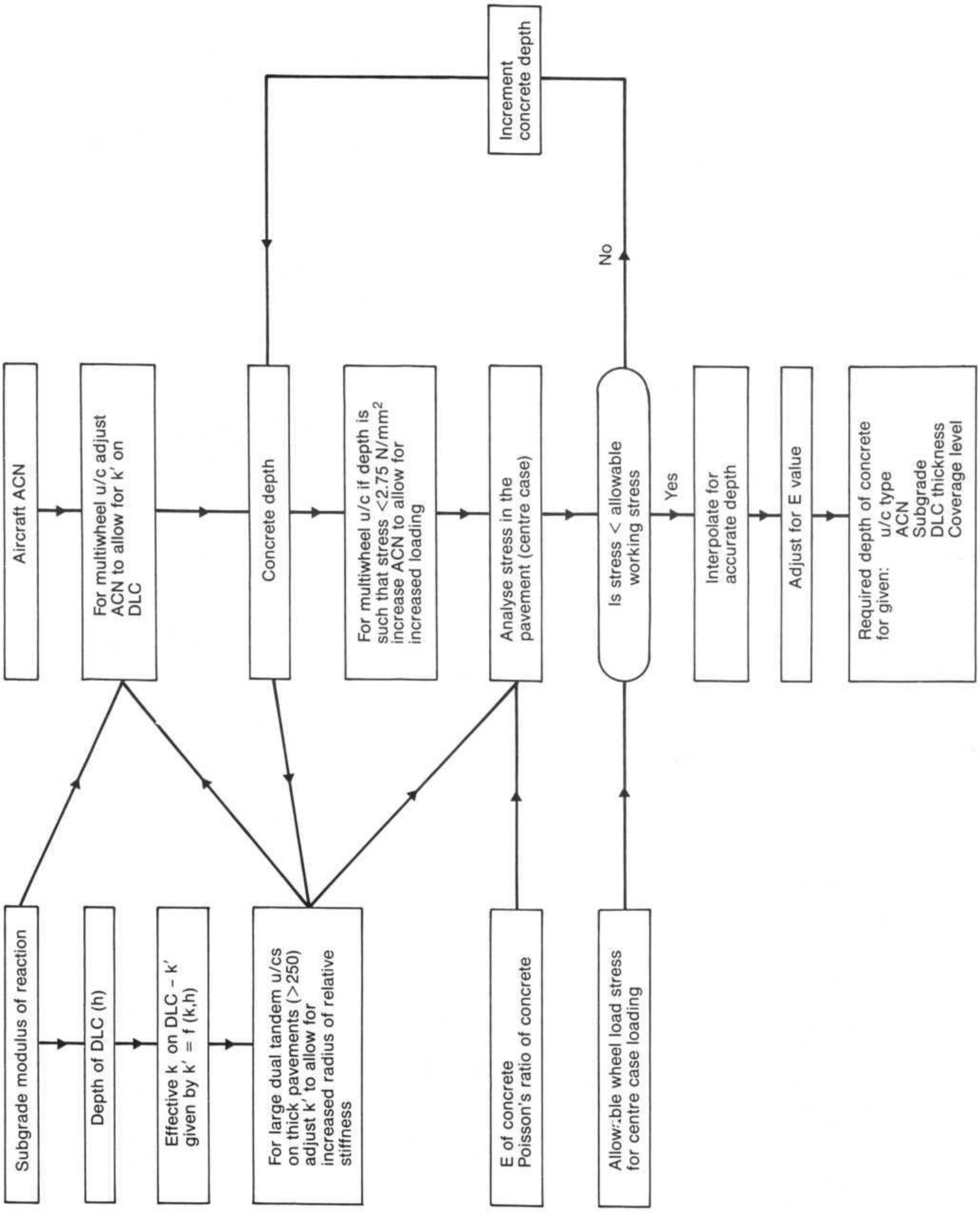


Figure 40 Flow diagram for computation of rigid pavement thickness

F.2.3.5 The structural effectiveness of a strong cement-bound base is accounted for in the design model by allowing for high long-term load transfer at transverse joints and an enhancement of the subgrade strength (k). The effective increase in k provided by the base varies depending on the actual subgrade strength, the thickness of base, the magnitude and configuration of wheel loads and whether the slab is loaded at its centre or edge. For heavy multiple wheel loads the effective increase in k provided by the base is reduced to allow for progressive cracking in the stiff cement-bound base a cracked modulus of elasticity of 8000 N/m^2 has been taken. The design concept has been established from a combination of elastic theory, plate bearing tests and experience of pavement performance.[40,41,42,43,44]

F2.4 Allowable Stresses

The rigid pavement design charts are based on an allowable wheel load stress at the interior of a concrete slab (centre case stress). If the stress at a slab corner is critical the allowable centre case wheel load stress is reduced to ensure that the corner is not overstressed. The allowable wheel load can be expressed as:

σ = F.R.

where σ is the allowable wheel load stress
R is the flexural strength of the concrete
F is the Design Factor.

In practice the allowable wheel load stress is derived directly and a value of F is not calculated. The difference between the flexural strength and the allowable wheel load stress is the allowance made for the fatigue effects of the live load and temperature induced stresses.

F2.4.2 The following variables are taken into account in determining the allowable stress:

(i) The flexural strength of the concrete.
(ii) Temperature induced stresses:
  a) End-restraint compressive stresses
  b) Restrained and partially restrained warping stresses.
(iii) Load transfer at joints.
(iv) The ratio of corner to centre stresses.
(v) Load repetitions (Coverages).

The calculation of allowable wheel load stresses is summarised in Figure 41.

F2.4.3 The allowance for differential temperature effects is based on References 17,44,45,46 and 47. A sinusoidal daily variation in differential temperature up to a maximum of 15°C is incorporated and account is taken of the maximum bay sizes allowed by the Specification. However, in countries with extreme climates it is recommended that a slab thickness calculated from Charts 1,2,3 and 5 be increased by 10% to allow for excessive warping stresses (refer to para 5.1.4).

F2.5 Integration of the ACN-PCN Method

F2.5.1 To minimise the number of steps in the design procedure the ACN-PCN method has been linked directly to the design method. In doing so, the following factors have been considered:

(i) An ACN represents a single wheel load that produces a maximum flexural stress of 2.75 N/mm^2 at the centre of a slab. Consequently in using the ACN as the loading criterion it was necessary to include correction factors to account for different allowable wheel load stresses.
(ii) The ACN pavement model considers a slab directly on a subgrade whereas the lean concrete base is an integral part of the standard rigid pavement design. As a reduced thickness of PQC slab is required when a DLC base is provided correction factors have been incorporated to modify the ACN single wheel load for dual and dual-tandem aircraft.
(iii) For heavy dual-tandem aircraft, allowance is made for wheel load interaction in the DLC base by limiting the effective k value on the DLC base.

(iv) A correction factor has also been used to allow for the difference in the elastic modulus for concrete used in the ACN pavement model and that used in the rigid pavement design method.

(v) The ACN computation has been done on the basis of one main gear assembly. For current aircraft types this provides adequate representation of the loading severity. Comparative stress analysis for rigid pavements shows that load interaction between adjacent main wheel gears does not produce higher stresses than those for one main wheel gear on its own.

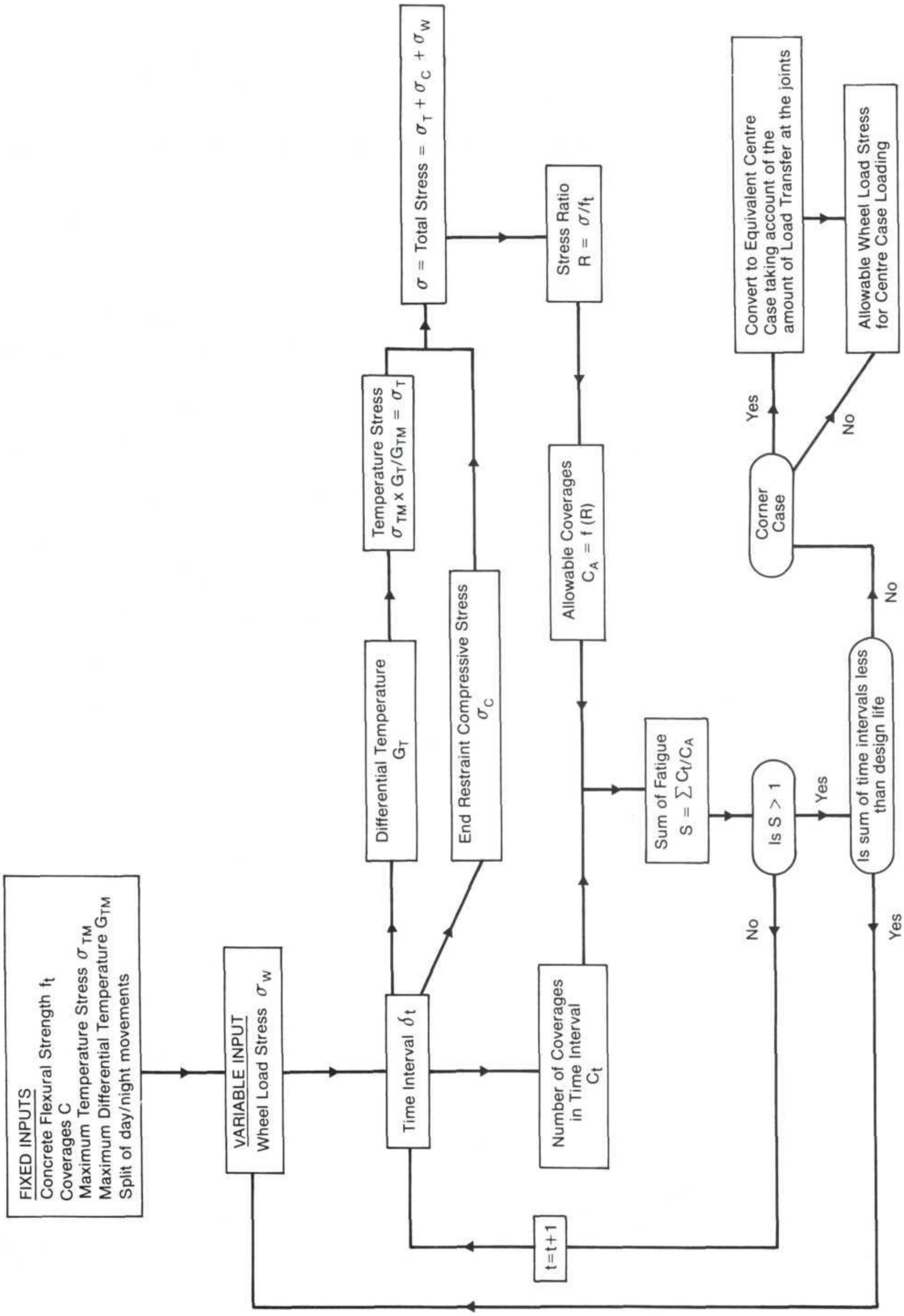


Figure 41 Flow diagram for the computation of allowable wheel load stress

F3 FLEXIBLE PAVEMENTS

F3.1 Failure Mode

F3.1.1 The stresses in the subgrade are high compared to allowable limits and the mode of failure is permanent deformation with the eventual sheer failure. A secondary mechanism which may lead to failure is full-depth fatigue cracking of the bituminous surfacing.

F3.2 Failure Criteria

F3.2.1 In Defence Estates' experience the primary failure mode for pavements with bound bases is likely to be rutting with associated heave due to shear failure of the subgrade; the high base stiffness considerably reduces the chance of fatigue cracking in the asphalt as an initial failure mode. Rut depth at failure, as assumed in the design method, depends on a number of factors, including the magnitude and configuration of the load, the subgrade characteristics, the pavement thickness and the lateral distribution of the aircraft wheel paths. Experience indicates that for pavements subject to channelised trafficking (e.g. a taxiway) by medium to heavy single wheel geared aircraft the acceptable rut at the end of the design life is unlikely to exceed 15 mm in depth and 1.5 metres in width. For pavements subject to less channelised trafficking (e.g. centre section of runway) by heavy multi-wheel geared aircraft the rut depth should be within the range of 20-40mm and the width may be in excess of 5 metres.

F3.3 The Method of Analysis and Allowable Stresses

F3.3.1 Design thicknesses for flexible pavements are obtained from the US Army Corps of Engineers CBR Equation.[49,12] This is a semi-empirical equation based on full-scale testing of pavements comprised of thin bituminous surfacings on unbound granular materials. As such the allowable stresses are automatically built into the design method.

F.3.3.2 A set of failure criteria is also built into the CBR Equation. To give the failure criteria described above the thicknesses obtained from the CBR equation have been increased by an empirical factor of 1.05.

F3.3.3 To take account of the improved performance of bound base constructions, Equivalency Factors have been applied to the thicknesses determined from the CBR Equation. The Equivalency Factors are related to a number of parameters including the quality of the base material and the subgrade support value; they have been developed from an analysis of full-scale tests carried out by Defence Estates,[50] US Army Corps of Engineers**Error! Reference source ot found.** and the Royal Engineers.[51]

F3.3.4 The strength of DLC has increased considerably since DLC was introduced in the 1950s. For many years Defence Estates attempted to control the strength of DLC in flexible pavements (Type F DLC) by specifying a maximum compressive strength of 15N/mm^2. By the 1980s this maximum strength was only being achieved by the use of very high aggregate to cement ratios. Because of the difficulty in achieving uniform mixing at very low cement contents, the maximum strength was abandoned in 1989 and replaced with a minimum aggregate to cement ratio of 23:1 for DLC in flexible pavements. Modern DLC strengths can be significantly higher than 15N/mm^2, and a new DLC specification, Type FH has been introduced to take advantage of the higher strengths. Minimum specification requirements for Type FH and Type F DLC are described in Appendix C.

F3.3.5 To assess the behaviour and performance of Type FH DLC accelerated testing was undertaken in the Pavement Test Facility in TRL. The testing showed a continuous decrease in the elastic stiffness of the layer with trafficking, so that an analysis of performance before cracking occurs is not possible. Chart 5 has been derived by setting a minimum elastic stiffness for Type FH DLC, based on a condition where the layer is substantially cracked but has not started to disintegrate into a granular material. The layer thickness is then designed using analytical methods based on Multi-Layer Elastic Analysis, so that:

1) Subgrade Vertical Strain is controlled such that rutting in the subgrade does not occur. The limiting subgrade vertical strain was determined from analysis of the CBR curves used to derive Chart 8.

2) Flexural stresses in the material are controlled so that further cracking does not take place.

F3.3.6 A minimum in situ elastic stiffness of 3,000N/mm^2 has been adopted for the control of subgrade rutting. For conservatism the elastic stiffness of Type FH DLC used to control further cracking has been set at 5,000N/mm^2. Temperature stresses have been ignored as they are very low in a substantially cracked layer. The thickness of the Bound Base Material is the thicker of the requirements calculated by the two approaches.

F3.3.7 The approach is fundamentally conservative if Type FH DLC is used as Bound Base Material, as it ignores the period before the DLC reaches the minimum elastic stiffness.

F3.4 Integration of the ACN-PCN Method

F3.4.1 To minimise the number of steps in the design procedure, the ACN-PCN method (see Chapter 2) has been linked directly to the design method. In so doing, two factors have been considered:

(i) An ACN represents a single wheel load calculated at a specific coverage level. In using the ACN as the loading criterion, it was necessary to incorporate correction factors into the design model to allow for designs at other coverage levels.

(ii) The ACN is calculated for one main wheel gear. In virtually all circumstances this loading case gives the maximum stress induced by an aircraft at the pavement/subgrade interface.

Appendix G Conversion of LCN/LCGs to PCNs

G1 GENERAL

G1.1 Until 1981 the standard method of reporting airfield pavements strengths used in the United Kingdom was the LCN/LCG system.9 This was also one of the four systems quoted in Annex 14 to the Convention on International Civil Aviation[13] as recognised methods of reporting pavement strength.

G1.2 From 26 November 1981 the four methods have been superseded in civilian use by a single standardise system known as the Aircraft Classification Number – Pavement Classification Number method (ACN-PCN method). This Appendix sets out a method for the conversion of LCN/LCGs to PCNs.

G1.3 There is no direct equivalence between LCNs and PCNs and any method of conversion can only be approximate. However, within practical limits Figure 42 and Figure 43 have been developed for the purpose of equating pavements originally designed using the 1971 LCN/LCG system to the PCN system. In addition to using the graphs it is recommended that an appraisal of the aircraft using the pavement is made, particularly those with the highest ACNs, to ensure that no unreasonable penalty is incurred in the classification; this is because anomalies arise due to the LCN/LCG and PCN systems being incompatible.

G1.4 These graphs have been prepared in the interests of maintaining consistency and continuity of pavement strength reporting. In consequence no account has been taken of the updated design methodology incorporated in this document.

G2 USE OF THE CONVERSION GRAPHS

G2.1 The first stage of the conversion is to decide whether the pavement is rigid or flexible. This decision should be based on the method used for the original design. The second stage is to select the subgrade category.

G2.2 As can be seen from the graphs the subgrade category can make a substantial difference to the PCN and should be chosen with care, based on the available soil information. If the value on the surface of a sub-base/fill is taken as the subgrade category care should be taken to ensure that this does not invalidate the design methodology (i.e. lean concrete bases are an integral part of Defence Estates' rigid pavement designs).

G2.3 To determine the PCN choose the relevant Figure and project a line up from the horizontal axis (LCN/LCG) until it intersects the curve for the appropriate subgrade category. The PCN is then found by a horizontal projection to the vertical axis from the intersection.

G3 TYRE PRESSURES

G3.1 There need normally be no tyre pressure limit on concrete or Marshall asphalt surfacings. Some surfacing such as Hot Rolled Asphalt would usually need an X rating. Lower ratings can be used if the operator has evidence that a surfacing has particularly low stability.

G4 METHOD OF EVALUATION

G4.1 The final part of the reported PCN is the method of evaluation. In this case the classification is based on the original design and therefore the evaluation can be described as Technical (T).

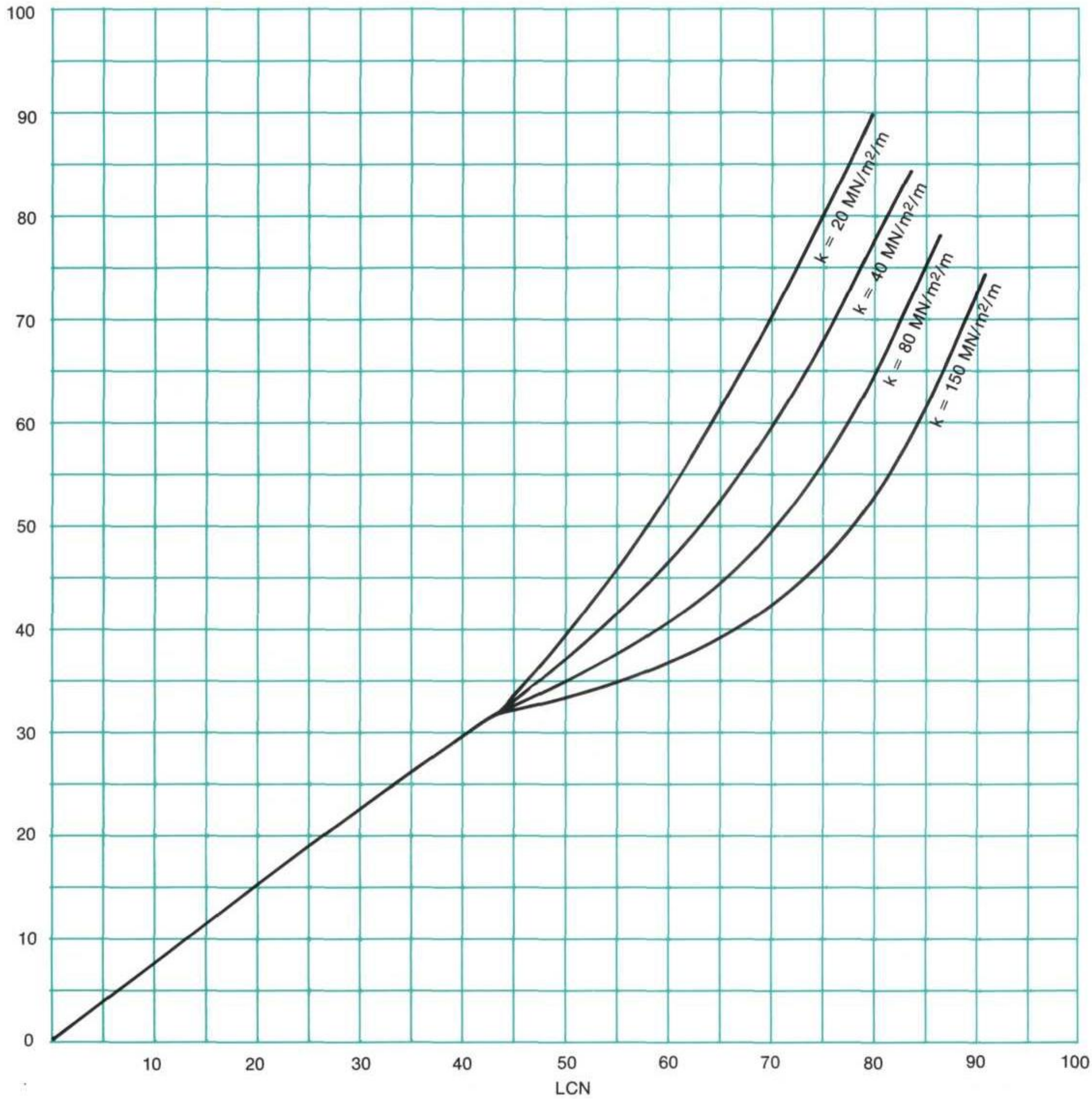


Figure 42 PCN/LCN Rigid conversion

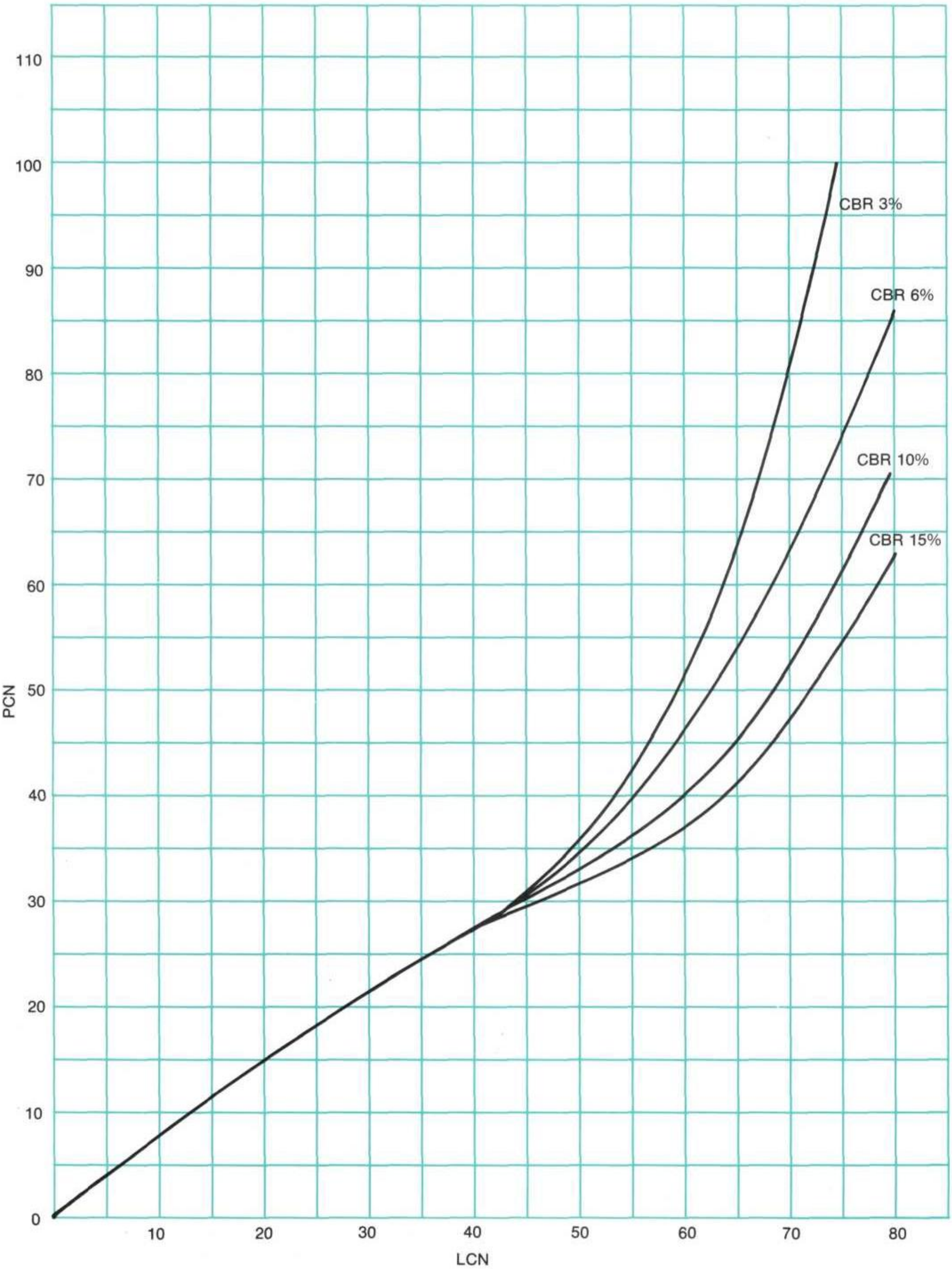


Figure 43 PCN/LCN Flexible conversion

Appendix H Evaluation Based on Experience of User Aircraft

H1 The ACN/PCN Classification System allows the aerodrome operator to assign a PCN to a pavement on the basis of experience of the pavement performance in use. As explained below pavement use is an unsatisfactory method of evaluating strength, but this Appendix provides some advice on obtaining a reasonably reliable estimate of the classification.

H2 Normal pavement failure is a fatigue process, and will occur after a relatively small number of load repetitions of a heavy load, or a larger number of repetitions by a light load. Therefore it is difficult to accurately assess the strength of a pavement from aircraft use since, although a pavement may have taken, for example, 10 movements of an aircraft with no problems, there is no way of knowing whether it will eventually fail after 100, 1000, 10000 or any other number of movements. The greater the number of movements that the pavement has had the more accurate the user classification, but the nature of the problem usually means that user classifications are based on a limited number of aircraft movements.

H3 One of the factors affecting the accuracy of the user evaluation is the effect of climatic conditions. This is especially significant in rigid pavements where temperature induced stresses in the concrete slabs may form a considerable proportion of the total stress in a pavement under load. A concrete pavement which accepts an aircraft at 08.00 hours on a dull day may fail when the same aircraft uses the pavement at 20.00 hours on a warm sunny day.

H4 It is invariably preferable to carry out a technical evaluation of a pavement, either by reverse design or in situ testing. However, if it is necessary to use a user evaluation a reasonably safe classification can be obtained by factoring the ACN of the user aircraft by the percentages given in Figure 44.

H5 In using Figure 44 the following conditions apply:

(i) The pavement must show no signs of structural distress.
(ii) The ACN appropriate to the actual operating weight of the aircraft should be used.
(iii) The scale used on the movement axis should be appropriate to the main wheel gear type of the aircraft.
(iv) The resulting classification will be appropriate for Low Frequency Trafficking.

H6 Although there are too many variables involved to allow a precise estimate of the reliability of a PCN obtained using Figure 44 there should be a 90-95% probability that the pavement life will be equal to or greater then the 10000 Coverages associated with Low Frequency Trafficking.

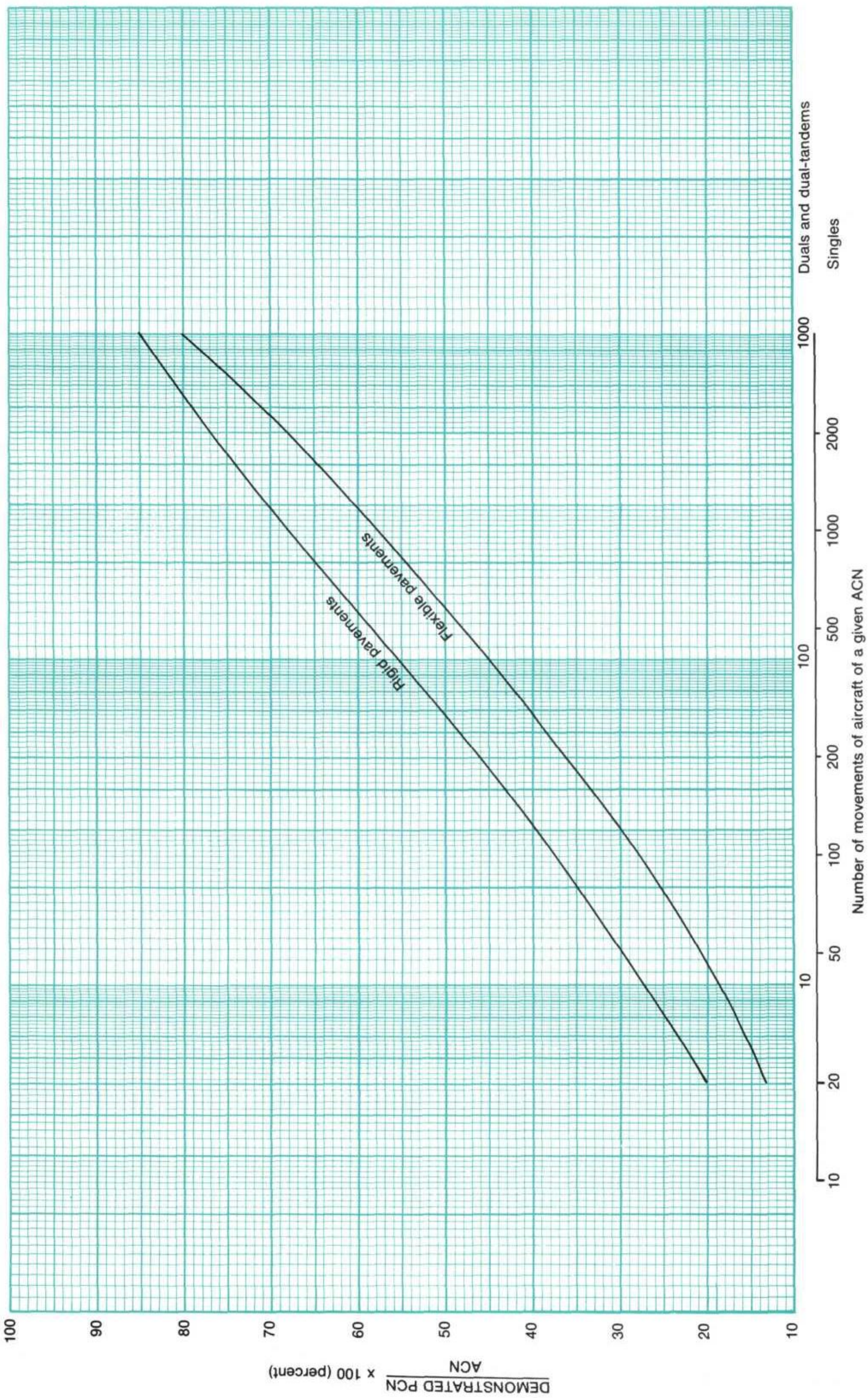


Figure 44 PCN by user aircraft evaluation

Appendix I Structural Investigations of Airfield Pavements

I1 INTRODUCTION

I1.1 General

I1.1.1 Chapter 7 describes the evaluation of airfield pavement strength by reverse design. This Appendix contains detailed guidance and advice on undertaking structural investigation on airfield pavements to provide the pavement and subgrade inputs to reverse design.

I1.1.2 Separate sections in this Appendix deal with the main activities that the pavement engineer will have to consider. Starting with planning the investigation, the Appendix goes on to describe the different types of surveys that are available providing guidance on when they should be used and how the results should be interpreted. This Appendix should be used when planning any form of structural maintenance on MOD airfield pavements.

I2 PLANNING THE INVESTIGATION

I2.1 The need for a pavement investigation

I2.1.1 Investigations to provide information for reverse design are carried out at "project" level, following a process of identifying projects at a network level (i.e. several airfields or a large number of pavements at a single airfield) (Figure 45). A structural investigation and pavement evaluation may be a project in its own right, e.g. to determine the residual life following a change in use, or may be a component of a much larger project, e.g. to strengthen a pavement.

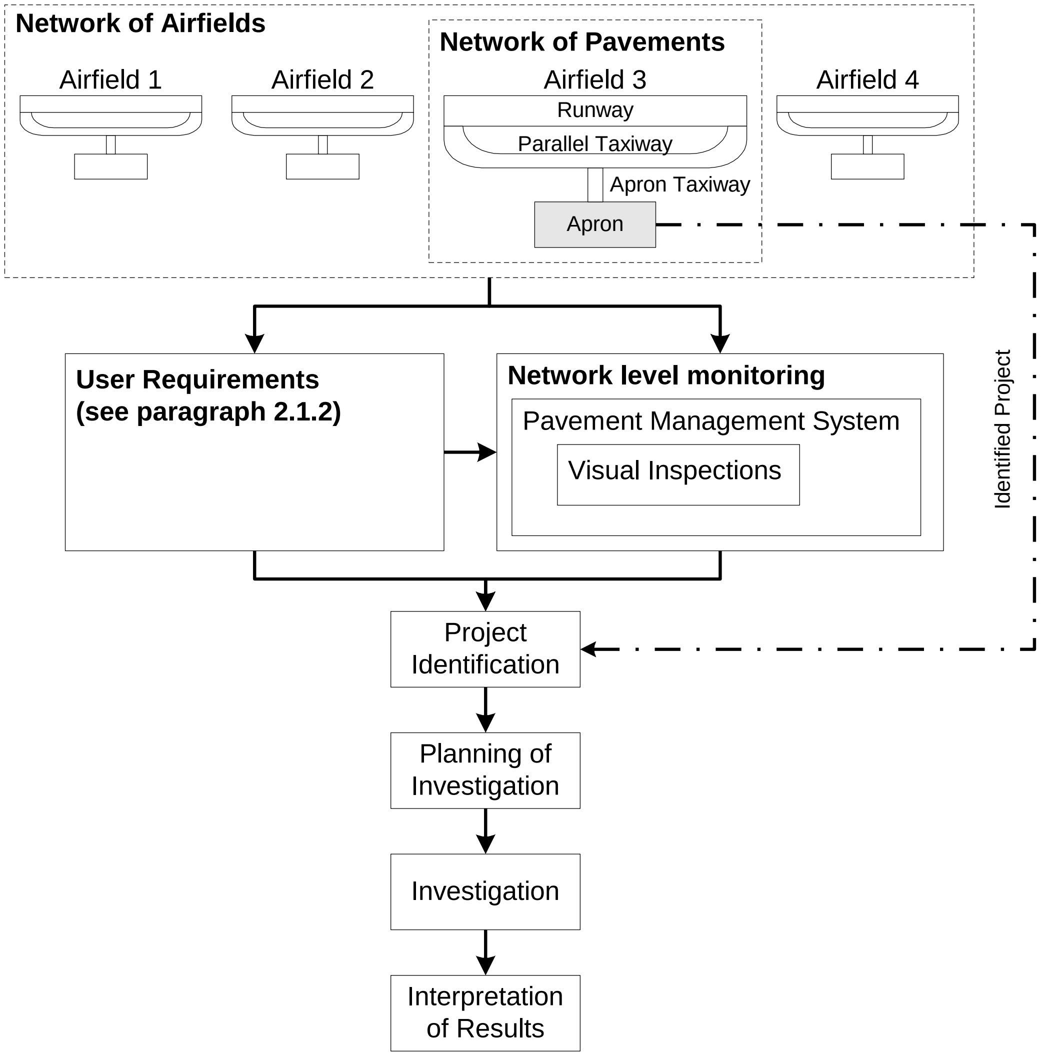


Figure 45 Identification of projects.

I2.1.2 The identification of a project may be based on:

(i) a user requirement, e.g. a change in use, a change in the frequency of use, or reinstatement of a disused pavement,

(ii) a method of network level monitoring, e.g. a Pavement Management System or regular investigations, to identify pavement maintenance or rehabilitation requirements.

I2.2 Components of a structural investigation

I2.2.1 Once a project has been identified it is good practice to undertake a structural investigation to obtain the information required for an accurate reverse design and design of any strengthening requirements (Figure 46).

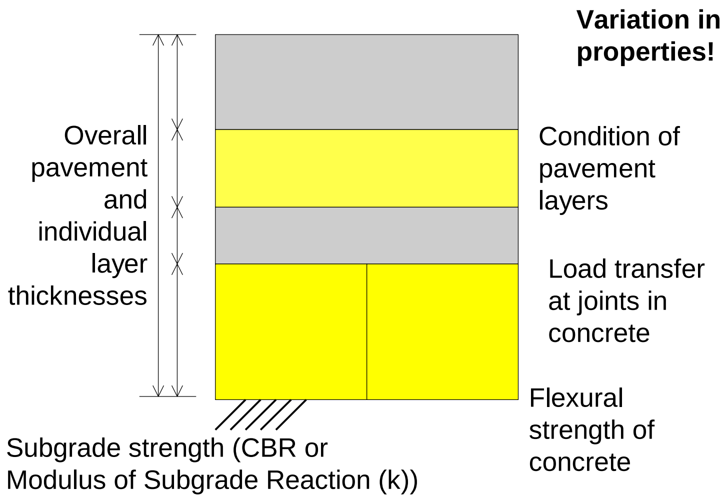


Figure 46 Information required from structural investigations.

I2.2.2 A structural investigation may comprise:

(i) Data collection.
(ii) A visual inspection (Section I3).
(iii) Destructive testing by coring or trial pits (Section I7).
(iv) Subgrade strength tests in core holes or trial pits, usually by Dynamic Cone Penetrometer (DCP) (Section I4).
(v) Non-destructive testing by Falling Weight Deflectometer (FWD) or Ground Penetrating Radar (GPR) (Sections I4 and I6).
(vi) Compressive strength tests on recovered concrete samples (Section I7.4.3).

I2.2.3 Possible components of a structural investigation and their relationship with pavement evaluation by reverse design are shown in Figure 53. The minimum requirements are cores or trial pits at regular intervals and subgrade strength tests in the core holes or trial pits. However, the more comprehensive the investigation, the lower the risk of unnecessary capital expenditure due to over-design, or early failure due to under-design.

I2.2.4 In planning an investigation the following points should be considered:

(i) Before undertaking a structural investigation all available data on the construction history, maintenance records and previous use should be collected.
(ii) A visual inspection must always be undertaken.
(iii) Destructive testing by coring or trial pits is always necessary, to obtain thicknesses for direct use or for calibration of GPR results, to allow subgrade strength tests, and to recover concrete samples for strength testing. Coring is the preferred option, but cores are not practicable in thick unbound constructions when trial pits are necessary.
(iv) Non-destructive testing cannot provide accurate measurements of subgrade strength, therefore subgrade strength tests through cores or trial pits are necessary.
(v) Destructive testing of operational pavements is difficult. Usually only a limited number of tests is practicable, which can give misleading results.
(vi) Airfield pavements and subgrades are typically very variable. In this situation, a large number of relatively accurate test results is more useful than a small number of very accurate results. Homogenous sections (see Section I2.3) can only be detected by frequent tests, such as FWD tests or a GPR survey. FWD and GPR tests can be used to reduce the required number of cores or trial pits.

(vii) The accuracy of destructive testing can be improved by selecting representative locations through statistical analysis of deflection measurements by FWD (see Section I4). However, this process requires adequate time for analysis of the FWD results before the core locations are selected, and if there are services under the pavement the optimum test locations may not be accessible.

(viii) If core locations have to be fixed before the investigation to allow for services clearances the best way of ensuring that the information on pavement thicknesses is representative is to carry out a GPR survey, and to use the core information to calibrate the results (see Section I6).

(ix) FWD testing is the only method of measuring load transfer at joints.

(x) Information from cores or trial pits, or FWD results, is too coarse or difficult to analyse for the accurate detection of hidden construction changes. GPR provides the best method of investigating varying construction.

(xi) The use of layer thicknesses determined from GPR measurements may significantly improve the accuracy of the analysis of FWD results.

(xii) The DCP is the preferred method of testing for the strength of unbound bases and sub-bases and the subgrade as it provides continuous readings with depth, allowing the thickness and strength of each layer to be established without the need to excavate the layers and test at discrete points.

I2.3 Homogenous sections

I2.3.1 A key part of the structural investigation is the identification of homogenous sections of pavement behaviour. A homogenous section is one that cannot be further sub-divided in sub-sections with significantly different means. Multiple homogenous sections may occur within a single Construction Location, as factors such as layer thicknesses and condition and subgrade strength vary (Figure 47).

I2.3.2 The planning of the investigation should take account of the expected variation and the need to detect homogenous sections; i.e. if the subgrade is expected to be very variable FWD testing is desirable to detect changes in the subgrade.

I2.3.3 Methods of determining homogenous sections are described in Annex C.

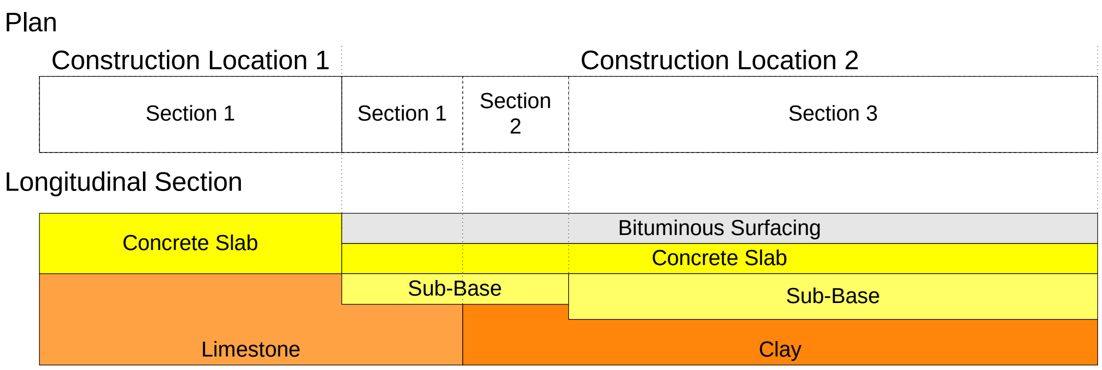


Figure 47 Homogenous Sections.

I2.4 The plan

I2.4.1 The plan for a structural investigation should take account of:

(i) The area to be investigated.
(ii) Operational restrictions and access periods.
(iii) The likelihood of services and requirements for service clearances.
(iv) The known information about the pavement.
(v) The likely variability of the pavement.
(vi) Whether concrete strength is required.
(vii) Whether the pavement is jointed concrete and if the load transfer at joints is to be measured.

I2.4.2 The plan should comprise:

(i) The type of testing.
(ii) The number of lines to be tested and their location.
(iii) The frequency of the tests.

An overview of recommended test locations and frequency of testing is shown in Figure 48.

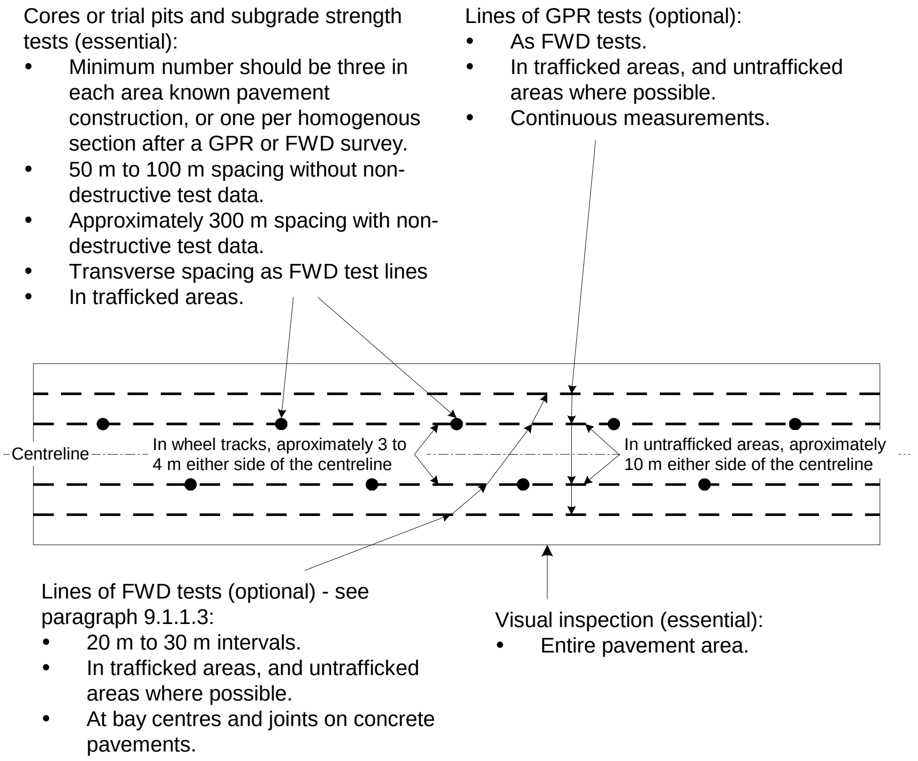


Figure 48 Test locations and frequency.

I2.5 Other tests methods

I2.5.1 There are several other test methods that may be used in some circumstances, including:

(i) Other deflection measuring methods, such as the Deflection (Benkleman) Beam, which may be used as a substitute for the FWD for a simple deflection survey to determine homogenous sections and representative test locations if access by FWD is not practicable (Section I5).
(ii) Alternative subgrade strength tests, including in situ CBR, plate tests and MEXE Penetrometer which may be used if a Dynamic Cone Penetrometer is not available (Section I7.2).
(iii) The use of GPR for the detection of cracks, voids or debonding (Section I6.3.5).
(iv) The use of trial pits to look for the location and cause of failures (Section I7.1.3).
(v) Laboratory tests for the properties of bituminous materials (Section I7.4.2).

I3 VISUAL INSPECTIONS

I3.1 General

I3.1.1 A detailed visual inspection is an essential component of a structural investigation.

I3.1.2 Before undertaking the inspection the following should be determined as far as possible to give an indication of what to look for and where to look for it:

(i) The nominal pavement construction and the age of the various layers.
(ii) The aircraft use to date, including the track dimensions of the major user aircraft.

If no details are available an inspection can still yield useful evidence on the construction.

I3.1.3 The most important areas of a pavement for a structural inspection are those regularly trafficked by aircraft. Although structural deterioration, due to ageing of asphalt or similar processes, can take place uniformly across a pavement, the most important information for design and evaluation purposes is how the pavement is behaving under aircraft loading, and what the condition is of the areas regularly trafficked. If there is a centreline or nose wheel marking on the pavement, 75% of load applications can be expected to occur within bands about 2m wide centred on the centreline of each main undercarriage strut. Without markings for the nose wheel to follow trafficking, may be much more uniformly distributed across the pavement and the inspection will have to cover a wider area in detail.

I3.1.4 The whole width of the pavement should be inspected to determine whether there are significant differences between trafficked and untrafficked areas. Distress features which occur uniformly across a pavement are unlikely to have been caused by aircraft use and some other reason should be looked for.

I3.1.5 Failure of a pavement as a whole is not caused by structural failure at one individual point in the pavement. The inspection should note the proportion of the area within the wheel tracks that shows signs of structural distress for comparison with the failure criterion of structural failure over approximately 30%-50% of the areas regularly trafficked by aircraft.

I3.2 Concrete surfaces

I3.2.1 The failure mechanism of a concrete pavement is cracking; followed by deterioration of the cracks due to movement or weathering. In situations where there is little weathering action (e.g. hangar floors) or where a very stiff support to the top concrete slabs restricts deflection (e.g. multiple slab construction) cracks may exist for long periods before they become a maintenance problem. For this reason it is helpful to have adequate information on historical aircraft use, since cracks could be present in the wheel tracks of an aircraft that no longer uses the pavement and was different in size to the existing user aircraft; these cracks can be difficult to explain without the relevant knowledge.

I3.2.1 The survey should note the presence and amount of the following:

(i) Corner cracks – and their relation to expansion joints or very open transverse joints.
(ii) Longitudinal halving cracks (i.e. parallel to the direction of the concrete lanes) – and whether they appear to start at joints or the bay centre.
(iii) Transverse halving cracks.
(iv) Quartering cracks.
(v) Delta cracks.
(vi) Multiple cracked bays.
(vii) The relation between aircraft wheel paths and the type of cracking.

I3.3 Bituminous surfaces

I3.3.1 Pavements with bituminous surfaces fall into two categories:

(i) Bituminous materials overlying concrete (composite pavements).
(ii) Bituminous materials overlying unbound granular bases and sub-bases or relatively weak cement-bound materials (e.g. Drylean Concrete or Cement-Stabilised Soil) or full depth bituminous constructions (flexible pavements).

I3.3.2 The failure mechanism in flexible pavements is rutting with associated heave, due to shear failure of the subgrade or unbound pavement layers, or full depth cracking of the bituminous surfacing. The heave will be accompanied by ‘alligator cracking'. A structural survey should check the following:

(i) Rut depth.
(ii) Height of heave.
(iii) The width of the rut (between the highest point of the heaved areas).
(iv) The presence of alligator cracking.
(v) Longitudinal cracking in the wheel paths.
(vi) Other forms of cracking, such as block cracking.

I3.3.3 A composite pavement with a relatively thick overlay of bituminous materials will, in the long-term, behave like a flexible pavement, and the distresses described above should be noted.

I3.3.4 A composite pavement with a relatively thin overlay of bituminous materials will, in the long-term, behave like a concrete pavement. The inspection should note the underlying bay layout and any of the crack features described in Section 0 if they are reflecting through the bituminous surfacing.

I3.3.5 It is sometimes difficult to tell the difference between asphalt lane joints and reflective cracks from longitudinal construction joints in concrete slabs. Bituminous overlays are usually constructed with an overlap of around 1m over the longitudinal concrete joints. The asphalt lane joints are likely to be straighter and more continuous than the reflective cracks.

I3.4 Other distresses

I3.4.1 Other structurally related distresses that should be noted are:

(i) Mud pumping, which may occur in all types of pavement. Mud pumping may be directly visible in certain conditions, or detectable by staining of the pavement surface around joints and cracks.

(ii) Blistering of bituminous surfaces - small transient domes on the surface of the pavement that form in hot weather conditions, or associated cracking in the form of a cross at the centre plus an additional crack around some or all of the circumference. Blistering or the presence of the symptomatic cracking should be noted.

I4 FALLING WEIGHT DEFLECTOMETER

I4.1 General

I4.1.1 This section gives guidance on the use of the Falling Weight Deflectometer (FWD) for assessing the structural condition of airfield pavements. It describes the principles of operation of the FWD and provides detailed guidance on how to analyse and interpret FWD data.

I4.1.2 The FWD is used primarily on airfield pavements for evaluating the elastic stiffness of various pavement layers which in turn can be used to indicate material condition, and for determining the load transfer efficiency at joints in rigid pavements. These two types of surveys require the FWD to be configured differently and require very different analysis and interpretation.

I4.2 Equipment and principle of operation

I4.2.1 The FWD was conceived at the Laboratoire Centrales Pontes et Chausses (LCPC) in France but was developed in Denmark into a sophisticated non-destructive test method for evaluating pavements. The FWD generates a load pulse by dropping a weight onto a damped spring system mounted onto a circular loading plate. The spring smoothes the load of the falling weight to produce a sinusoidal pulse. The mass, drop height and plate can all be adjusted to achieve the desired impact loading. The load pulse has a shape similar to that generated by a moving vehicle. The resultant deflection basin (peak deflection) is measured at the centre of the loading plate and at several radial positions by a series of geophones. The actual load applied to the pavement is partially dependent on the FWD-pavement interaction and is therefore measured by a load cell placed on the surface of the plate.

I4.2.2 FWDs vary in detail depending on manufacturer and model type. The majority of FWDs (and all those currently in use in the UK) are trailer-mounted devices, towed behind a vehicle as shown in Figure 49. Most FWDs typically have a loading range of 30 to 120kN. Some manufacturers have developed deflectometers that are capable of applying greater loads, typically up to 240kN. Very similar in operation and appearance to FWDs, these devices are often referred to as Heavy Weight Deflectometers (HWD). Although the HWD has been specifically developed for testing thick strong pavement structures such as airfield pavements, recent studies undertaken by TRL on a wide range of MOD airfields have shown that the results from FWDs and HWDs are extremely similar. Therefore, on the vast majority of airfields, there is little advantage to be gained from using the additional loading capacity of the HWD. Exceptions to this will include pavements built on exceptionally stiff subgrades, where errors in deflections could give misleading results, and testing the load transfer of very strong pavements where a FWD may not mobilise the joint. Throughout the rest of this note, the abbreviation FWD is used to indicate either a FWD or HWD.

Figure 49 Falling Weight Deflectometer

I4.3 Machine calibration and approval

I4.3.1 It is an essential requirement that all FWDs used for evaluating MOD airfields should be subject to machine calibration and approval in accordance with Chapter 5 and Annex 4 of HD29/94 *Design Manual for Roads and Bridges* (Highways Agency et al, 1999) including:

(i) Absolute annual calibration of the sensors, load cell and system processor.
(ii) Regular consistency checks of the dynamic response.
(iii) Annual correlation trials.

I4.4 Layer stiffness evaluation

I4.4.1 Test Procedure

I4.1.1.1 The vast majority of MOD airfields are of jointed rigid or composite construction (i.e. rigid overlaid with bituminous material). When using the FWD for the purpose of determining layer stiffnesses, testing should normally be undertaken following the guidance in Annex A.

I4.4.2 Deflection Profiles

I4.4.2.1 The adjustment (or *normalisation*) of FWD deflections to standard conditions makes the comparison of deflections more straightforward. Normalisation of FWD for load (or contact pressure) is achieved using linear extrapolation i.e. the measured deflections are multiplied by the factor ($Load_{target}/Load_{measured}$) and assumes that the deflection is linearly dependent on load. This assumption is reasonably correct as long as the measured load is not significantly different from the standard load level. The FWD deflection data, normalised to the standard test load level of 100kN, may be tabulated and plotted to show the variation of pavement response along each of the airfield test lines. Different parts of the deflection bowl are influenced by different pavement layers. The central deflection (d1) gives an indication of the overall pavement stiffness. The outer deflection measurements (e.g. d6 and d7) give an indication of the subgrade condition. The difference between deflection measured at two points close to the load (e.g. d1-d4) is mainly dependent on the stiffness of the upper bound layers. Deflection and deflection-difference plots are useful for showing relative differences in the condition of the layers, and enable delineation of the pavement into sections with similar behaviour, giving an indication of where structural weakness may be present.

I4.4.3 Surface Modulus

I4.4.3.1 Deflection measurements can be used to produce surface modulus plots. The surface modulus at a point, distance r from the centre of the loaded area, is roughly equal to the "weighted mean elastic stiffness" below a depth R on the load centre line. Note that the depth R is based on the "equivalent pavement thickness", where the thickness of the pavement layers is converted to an equivalent thickness of a material with an elastic stiffness equal to the subgrade stiffness. At a point sufficiently far from the loaded area, the deflection is not influenced by the upper pavement layers. Therefore the surface modulus calculated at the outer points on the deflection bowl is approximately equal to the subgrade modulus. Such plots give an indication of the stiffness of the pavement at different equivalent depths and can be used as guidance for the selection of further investigation and analysis methods. Further details of this method are given elsewhere (FEHRL, 1996).

The surface modulus at the top of the pavement (equivalent depth = 0mm) is calculated as:

$$E_o = 2(1-<^2)\Phi_o a/\ast_o \qquad (1)$$

The surface modulus at the equivalent depth R (valid for r>2a) can be calculated from:

$$E_o\,(r) = (1-<^2)\Phi_o a^2/(r.\ast_r) \qquad (2)$$

Where

E_o	= the surface modulus at the centre of the loading plate (MPa)
$E_o(r)$	= the surface modulus at a distance r (MPa)
$<$	= Poisson's ratio
Φ_o	= contact pressure under the loading plate (kPa)
a	= radius of the loading plate (mm)
r	= distance from sensor to loading centre (µm)
$\ast_r$	= deflection at a distance r (m)

I4.4.3.2 For surface modulus analysis, it is normal to assume a value of 0.35 for Poisson's ratio. There are five common examples of surface modulus plots (Figure 50):

(i) A continuously decreasing value of surface modulus with increasing distance. This indicates that the outermost deflection measurement points were not far enough away from the load.

(ii) A decreasing value which becomes constant. This indicates a normal pavement structure overlying a linear elastic subgrade.

(iii) A decreasing value which starts to gradually increase for the outer deflection measurement points. This indicates a normal pavement structure on a non-linear elastic subgrade, or a layered subgrade which increases in stiffness with depth.

(iv) A decreasing value with a sudden large increase for the outermost measurement points. This indicates that a very stiff subgrade layer underlies the pavement (e.g. rock).

(v) A minimum value close to the surface. This indicates a weak interlayer somewhere in the upper bound layers.

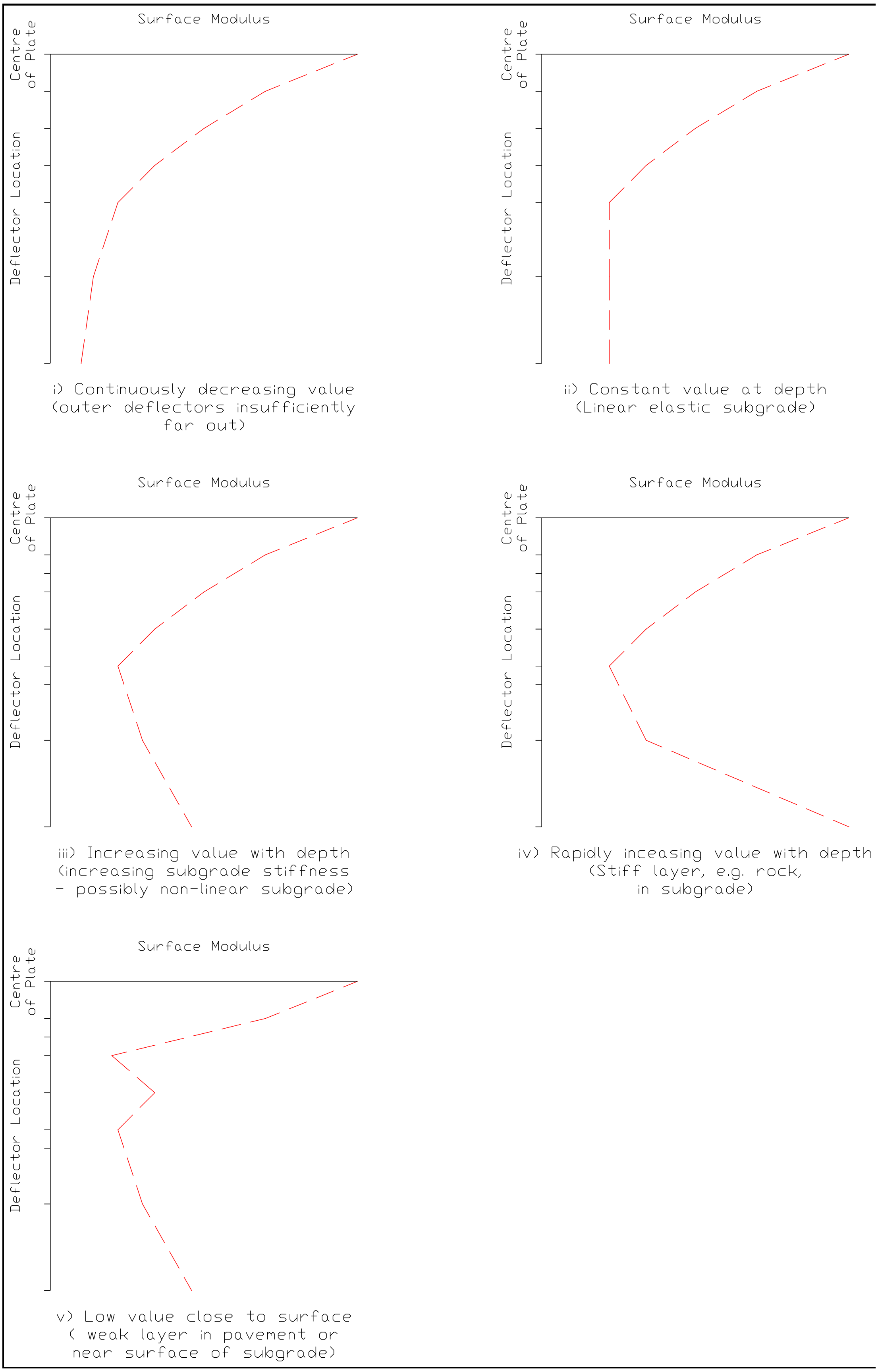


Figure 50 Typical Surface Modulus Plots.

I4.4.4 Standard Back-analysis Procedure

I4.4.4.1 The shape and magnitude of the deflection bowl generated by an FWD impact load depend on the type, thickness and stiffness of each of the pavement layers. Computer programs using linear elastic multi-layered analysis can be used to model the pavement structure. The analysis is based on a mathematical model of the pavement structure that predicts the surface deflection under a given applied load. Using an iterative process, the program adjusts the stiffnesses of the pavement layers in order to match the computed deflections to the measured values. The layer stiffnesses are adjusted until a satisfactory match is obtained. This process is known as "back-analysis".

I4.4.4.2 Layer stiffness results from back-analysis are extremely sensitive to the layer thicknesses assumed for the analysis. It is therefore essential that accurate and reliable thickness information be obtained prior to analysis. Coring can provide information of sufficient accuracy. However, coring of all test points is usually impracticable and GPR, in conjunction with cores, is an effective means of obtaining the necessary information (Section I6).

I4.4.4.3 Recent research has also shown that small changes in temperature gradients in concrete slabs may significantly influence absolute deflections and the shape of the deflection basin. Back-analysis of the elastic stiffnesses of layers in rigid pavements may therefore be misleading. Possible indicators are very high elastic stiffnesses for the concrete layer, very low stiffnesses for bound bases and high stiffnesses for subgrades when compared with strengths obtained from in situ soil strength tests.

I4.4.4.4 There are many different programs available for performing FWD back-analysis. The results produced can vary quite significantly depending on how the pavement is modelled, what layer thicknesses are assumed and the program used. Therefore a standard procedure is specified in this Guidance Note which should be used when performing back-analysis on all MOD airfield pavements. The use of this procedure should produce reasonably consistent results independent of who is performing the analysis. There will be occasions where alternative back-analysis procedures may be required but their use must be clearly justified and they must be used *in addition* to the standard procedure.

I4.4.5 Back-analysis Program

I4.4.5.1 At present there is no approved standard FWD back-analysis program for use on MOD Airfield Pavements. As a minimum, the program to be used for back-analysis needs to comply with the following criteria:

(i) It shall model the pavement structure as a number of horizontally infinite linear elastic layers.
(ii) It shall employ elastic multi-layer analysis based on Burmister's equations (Burmister, 1945) with all layers modelled linearly including an infinite depth subgrade and no slip between layers.
(iii) It shall be capable of modelling at least three independent layers.
(iv) It shall be capable of handling at least seven geophone sensors.
(v) It shall be capable of reporting the computed deflection values.

I4.4.6 Pavement Model

I4.4.6.1The model used for back-analysis i.e. the number and thickness of layers, can significantly affect the results obtained. The following rules should be applied when determining how to model the pavement:

(i) The minimum thickness of any single pavement layer shall be 75mm.
(ii) The maximum number of independent layers (including the subgrade) shall be four.
(iii) Bituminous layers should normally be combined and modelled as a single layer.
(iv) Where a layer of concrete overlies another concrete layer, these should normally be modelled separately (subject to constraint (ii).
(v) The unbound layers shall normally be modelled as a single layer of infinite depth*.
(vi) Poisson's ratios for each layer shall be as shown in Table 28.
(vii) Bituminous layer stiffnesses should be calculated at the test temperature and then adjusted to the reference temperature of 20˚C using equation 3.

(viii) The "goodness of fit" parameters defined in equations 4 to 7 should be calculated for each bowl.

*If the surface modulus plot indicates the presence of a non-linear subgrade or bedrock then an additional analysis using a layered subgrade model or a stiff layer at depth may be appropriate.

Adjustment of bituminous layer stiffness to 20°C:

$$E_{20} = E_T . 10^{(0.0003 \text{ x } (20-T)^2 - 0.022 \text{ x } (20-T))} \qquad (3)$$

Where:

E_{20} = Stiffness at 20°C

E_T = Stiffness at temperature T

T = Temperature of the bituminous material at the time of testing (normally measured at 100mm depth).

Table 28 Poisson's ratios for use in back-analysis.

Material	Poisson's Ratio
Bituminous Bound	0.35
PQ Concrete	0.15
Weak Cement Bound	0.20
Crushed Stone	0.40
Soils (fine-grained)	0.45

I4.4.7 Goodness of Fit

I4.4.7.1 There are two "goodness of fit" parameters commonly used for indicating how well the program has matched the data. These are the Absolute Mean Deviation (AMD) and the Root Mean squared Deviation (RMS) which can be expressed either in absolute terms (microns) or in percentage terms. The AMD parameters indicate whether or not there is an overall bias to the calculated bowl relative to the measured bowl. The RMS parameters indicate how well, on average, the calculated bowl matches the measured bowl. Although a good fit does not in itself indicate that a correct solution has been obtained, a poor fit does indicate that the solution found is suspect. The parameters are defined as follows:

$$\text{AMD} = \left| \sum (d_{ci} - d_{mi})/n \right| \qquad (4)$$

$$\text{RMS} = \sqrt{(\sum (d_{ci} - d_{mi})^2/n)} \qquad (5)$$

$$\text{AMD(\%)} = \left| \sum ((d_{ci} - d_{mi})/ d_{mi})/n \right| .100 \qquad (6)$$

$$\text{RMS(\%)} = (\sqrt{(\sum ((d_{ci} - d_{mi}) / d_{mi})^2/n)}).100 \qquad (7)$$

Where d_{mi} are the measured and d_{ci} the calculated deflections at positions i =1 to n, respectively in microns, where n is the total number of sensor positions used in the analysis (normally seven).

I4.4.7.2 Different back-analysis programs vary in their ability to match calculated to measured deflections. However, poor fits can also be obtained where cracks or other discontinuities are present in the pavement, where incorrect assumptions about layer thicknesses or material types are made, or where layer debonding is present. In addition, increasing the number of layers improves the level of fit. Table 29 contains guidance values based on AMD and RMS for three layer models. Bowls for which the AMD or RMS exceed these values should be treated with caution. Isolated results which exceed these limits should be discounted when assessing the overall condition of a section.

Table 29 Guide values for goodness of fit.

Parameter	Maximum Acceptable Value (microns)
AMD	2.0
RMS	10

I4.4.8 Material Threshold Stiffnesses

I4.4.8.1 Layer stiffnesses produced from back-analysis can be related to material condition using the reference values listed in Table I8.1. However, this comparison should normally be done on representative mean values for homogenous pavement sections.

I4.5 Load transfer evaluation

I4.5.1 Test Procedure

I4.5.1.1 The relative degree of load transfer at joints may be assessed by loading the slab on one side of the joint whilst measuring deflections on each side of the joint. Details of the procedure to be adopted when using the FWD for the determining load transfer are given in Annex A.

I4.5.2 Assessment of Load Transfer

I4.5.2.1 The Load Transfer Efficiency (LTE) in deflection is calculated from:

$$LTE = (d_{300}/(d_{200})).100 \qquad (8)$$

Where

LTE = load transfer efficiency (%)

d_{300} = the deflection on the unloaded slab (300mm from the centre of the load plate)

d_{200} = the deflection on the loaded slab (200mm from the centre of the load plate)

I4.6 Reporting of results

I4.6.1 When reporting the results of a FWD survey, the following minimum information should be included:

For all surveys:

(i) Make, model and serial number of the FWD.
(ii) Date of the survey.
(iii) Temperatures (and depths) measured during the survey.
(iv) FWD set-up used including load/s, plate sizes and geophone positions.

Stiffness evaluation:

(i) Details of the FWD back-analysis program used (name and version).
(ii) Details of the model used (thicknesses assumed for each layer, number of layers and details on any deviation from the standard model described in paragraph I4.4.6.1).
(iii) Tabulated deflections.
(iv) Deflection profiles (paragraph I4.4.2).
(v) Back-calculated stiffnesses (including "as-measured" and "adjusted to 20°C" for bituminous layers).
(vi) Goodness of fit parameters for each bowl (paragraph I4.4.7).

Load transfer evaluation

(i) Tabulated deflections.
(ii) Load transfer efficiencies (paragraph I4.5).

I5 OTHER DEFLECTION MEASURING DEVICES

I5.1 General

I5.1.1 The FWD is the most popular and versatile tool for measuring the deflection of airfield pavements. However other devices are available to measure deflection, notably the Deflectograph and the Benkleman Beam. Although it is possible to use these devices to record a form of deflection bowl, the primary use of this equipment is to measure the peak deflection which can be used to identify relatively weak and strong sections and to split the site into homogeneous lengths (see Appendix C). The devices are also limited because the relatively limited loads that they can apply make it difficult to obtain realistically measurable deflections on strong airfield pavements. As a consequence of this, measurements will be feasible on many flexible pavements but are unlikely to be usable on any but the thinnest rigid pavements.

I5.1.2 This section briefly describes the equipment and principle of operation. Further information on the design, specification, calibration and operation of the devices is contained in TRRL Report LR834 (TRRL, 1978).

I5.2 Benkleman beam

I5.2.1 The Benkleman Beam (or Deflection Beam) was developed in the 1950s as a method of measuring flexible pavement deflections caused by vehicle loading. For road use, systems have been developed which use deflection as a measure of pavement strength, condition and rehabilitation requirements. It is a simple, manually operated device. It consists of a reference beam and dial gauge, which is positioned between the twin wheels of the rear axle of a loaded lorry. The transient deflection is measured as the lorry travels slowly along the line of the beam. The measurement is not an absolute value of surface deflection since the reference beam is itself influenced by the load.

I5.2.2 Results from the Benkleman Beam are very sensitive to operator technique and to the type of lorry used for testing.

I5.3 Deflectograph

I5.3.1 The Deflectograph is an automated deflection measuring system based on the Deflection Beam principle. It is a fully self-contained lorry-mounted system capable of measuring deflection in the two wheelpaths of the lorry. Measurements are taken while the lorry is in motion (2.5km/hour) with measurements approximately 4m apart. It is therefore considerably faster in operation than the Benkleman Beam but measurements are less easily aligned to particular precise locations, which may be important on jointed rigid pavements.

I5.3.2 As with the Benkleman Beam, this is an empirical measurement technique and it is essential that the equipment used is regularly checked and calibrated. Details of the requirements for these checks are given in Annex 2 of HD29/94 *Design Manual for Roads and Bridges* (Highways Agency et al, 1994). Deflectographs should also have taken part in and passed the most recent UK Deflectograph correlation trial. These are currently held annually and are organised by TRL on behalf of the Highways Agency.

I5.4 Test procedures

I5.4.1 Test procedures for using the Deflectograph on road pavements are described in HD 29/94. Further detailed guidance on the use of the Deflectograph and the Benkleman Beam is given in Annex B.

I6 GROUND PENETRATING RADAR

I6.1 Introduction

I6.1.1 Ground Penetrating Radar (GPR) is a non-destructive tool that can be used to obtain information about the construction and condition of pavements. The primary role of GPR on airfield pavements is the determination of layer thicknesses and changes in construction. However, GPR can also provide additional information about defects and features within pavements.

I6.1.2 There are many different types of GPR system available operating at different radar frequencies, with different antenna types and coupling methods (i.e. air coupled or ground coupled). This section provides a brief description of the principles of operation and includes guidance on the appropriate use of GPR on airfield pavements. However, for more detailed guidance and background information on the use of GPR on pavements, see Chapter 6 and Annexes 5, 6 and 7 of HD29/94 *Design Manual for Roads and Bridges* (Highways Agency et al, 2001).

I6.2 General principle of operation

I6.2.1 GPR operates by transmitting pulses of electromagnetic radiation from an antenna down into the pavement. As the electromagnetic waves penetrate the pavement, their velocities are changed and their strength is attenuated. In addition, some of the electromagnetic radiation will be reflected back at interfaces between different materials (such as changes in pavement layers). The GPR receives, via its antenna, some of the reflected wave and records its amplitude, phase, frequency and arrival time (relative to when the pulse was transmitted). The reflected signal amplitude, plotted against time, is referred to as a radar "waveform".

I6.2.2 In operation, the GPR apparatus is moved along the pavement and pulses are transmitted at fixed time or distance intervals. The individual reflected waveforms are recorded and a waveform graph is built up. The waveform graph can be interpreted to reveal information about the pavement structure.

The quality of information obtained from GPR surveys is dependent on three main factors:

(i) The electrical (dielectric) properties of the pavement materials.
(ii) The type of GPR equipment used.
(iii) The processing and analysis methodology used, including calibration procedures.

I6.2.3 GPR surveys should not be carried out when it is raining or when standing water is present on the surface of the pavement. This is because a thick film of surface water may affect the radar signal making interpretation of data more difficult. Calibration of the GPR may also be less certain in such conditions.

I6.2.4 Multi-channel GPR systems allow a wide range of data collection options. These can range from one measuring line being scanned with antennas operating at different frequencies in a single run, to a number of parallel measuring lines being scanned with antennas operating at the same frequency in a single run. The first option is useful for surveys where data is only required for a single line and the second option is useful where large areas need to be surveyed in detail.

I6.2.5 The sampling rate in the direction of travel controls the plan size of the pavement features that GPR can detect and depends on the survey speed, the firing rate of the radar pulses, the number of points used to define the radar waveform and the number of channels being used. Typically, a single channel GPR with a firing rate of 90 pulses per second, 512 points per waveform surveying at 80km/hr will make a measurement every 250mm of forward travel.

I6.2.6 There are two types of antenna design, dipoles and horns. Dipoles generally provide greater penetration and horn antennas operate with a larger air gap and so are more easily adapted to surveys at high speed. The most important factors defining the various types of GPR system is given in Table 30 (reproduced in full from HD29/94).

Table 30 Typical values of penetration and resolution for various types of GPR

Factors	Radar frequency				
	400 / 500 MHz	**900 MHz**	**1 GHz**	**1.5 GHz**	**2.0 - 2.5 GHz**
Antenna type	**Dipole**	**Dipole**	**Horn**	**Dipole**	**Horn**
Coupling: air coupling gives slightly better resolution of the surface layer but slightly less penetration than ground coupling.	Ground	Ground	Air	Ground	Air
Resolution: minimum thickness of surface layer which radar can resolve.	200mm	100mm	50mm	70mm	25mm
Penetration: practical depth to which the radar can provide information.	2m depending on subgrade material and moisture content	800mm	600mm	500mm	300mm
Sampling rate: effective spacing along road at which radar pulse is fired.	At 80 km/h a sampling rate of 4 measurements every metre is achievable on some radar systems which means that features and defects less than 250 mm long may be missed by the radar scan. At lower survey speeds smaller features and defects will be detected. Sampling rate is dependent on the type of ground radar equipment being used.				

I6.3 Recommended uses

I6.3.1 General

I6.3.1.1 With the GPR systems that are currently available, not all pavement features can be identified with the accuracy and reliability needed for assessment purposes. However, GPR technology and methodology are fast developing and it is likely that improvements will allow more features to be detected accurately and reliably in the future.

I6.3.1.2 The primary recommended applications for GPR on airfield pavements are:

(i) Determination of layer thickness.
(ii) Detecting changes in construction.

In addition, GPR can be used for:

(i) Locating reinforcement in concrete slabs.
(ii) Detecting large voids below unreinforced slabs.

I6.3.2 Layer Thickness

I6.3.2.1 Accurate layer thicknesses are essential for correct FWD back-analysis (see Section I4.4 of this Note). GPR can, in the right circumstances, determine layer thicknesses to a high degree of accuracy. However, it is important that the GPR survey is carried out in conjunction with coring. Cores are required to identify the materials present in each type of construction and, for some GPR systems, are also required to determine layer thicknesses at specific locations within the survey site in order to calibrate the GPR. Cores taken for the purposes of GPR calibration need to be carefully referenced to the GPR survey. Methods for calibrating GPR systems for layer thicknesses are given in Annex 6 of HD 29/94 *Design Manual for Roads and Bridges* (Highways Agency et al, 2001). Typical accuracy of layer thickness measurement is ±10%.

I6.3.2.2 GPR will, in general, detect all adjacent layers constructed of the same basic material as a single layer e.g. bituminous surfacing and base will usually appear as a single layer.

I6.3.3 Changes in Construction

I6.3.3.1 GPR surveys can be used for detecting changes in pavement construction along a site. For example, changes from concrete to bituminous bases can be detected. Cores will be required to identify the materials present in each type of construction.

I6.3.3.2 Any unusual variations in construction indicated by a GPR survey should be confirmed by coring.

I6.3.4 Detecting Reinforcement

I6.3.4.1 GPR surveys can be used to detect the presence of reinforcement, and other steel features such as dowel bars, in rigid pavements. Low speed surveys can also provide further details about the reinforcement such as depth and spacing. GPR surveys are unlikely to be able to detect the condition of the steel. However, if the steel is badly corroded and damaged the surrounding concrete, the survey may detect the damage.

I6.3.5 Void Detection

I6.3.5.1 GPR can provide estimates of the size of large air-filled voids and smaller water-filled voids under reinforced concrete slabs. The survey can also indicate the position and relative (plan) size of such voids. Surveys using multiple antennas are recommended, as these will give the greatest coverage with the minimum passes along the site. The presence of reinforcement makes such detection far more problematical and unreliable.

I6.3.5.2 Air-filled voids need to be greater than 80mm in height to be measured with reasonable accuracy with GPR. Water-filled voids and wet patches are more easily detected and can be measured where they are more than 25mm in height.

I6.3.6 Other Applications

I6.3.6.1 GPR surveys can be undertaken to investigate other defects relevant to airfield pavements. These include:

(i) Debonding of pavement layers.
(ii) Crack detection.

I6.3.6.2 Debonding between different bound layers can sometimes be detected but it probably requires water to be present in order to be detected. Special GPR systems have been developed which are designed to investigate cracks in pavements. Crack-depth determination may be possible for surface cracks.

I7 DESTRUCTIVE TESTING

I7.1 Coring and trial pits

I7.1.2 Cores

I7.1.2.1 Coring is one of the most useful tools for investigating structural condition in airfield pavements. As well as providing important information about layer thicknesses, necessary for the correct interpretation of FWD and GPR surveys (see Sections I4 and I6), cores provide an opportunity to examine the visual condition of the various materials in the pavement. Coring also provides material that can be subjected to further testing in the laboratory, where appropriate, and it effectively opens a window in the pavement through which Dynamic Cone Penetrometer tests can be carried out to reveal further information about the condition of the unbound layers.

I7.1.2.2 Cores shall normally be extracted to the full depth of the bound material present in the pavement and shall generally be 150mm diameter. If cores are taken from concrete slabs, where the slab thickness is less than 150mm, then 100mm diameter cores will be sufficient although they may not provide enough material for laboratory testing.

I7.1.2.3 After removal of the core, water should be removed from the core hole, the total depth of the bound material recorded and the sides inspected, as far as possible, for voids and cracks. The high shear forces induced by coring may cause some bound materials, in particular old tarmacadams, to break up in the core. The sides of the core hole should be inspected to see if materials that have disintegrated have deteriorated in the pavement.

I7.1.2.4 All cores should be logged to record the layer thickness, material type and condition. The following should also be noted:

(i) Aggregate type (e.g. crushed rock, gravel, slag), nominal size and shape.
(ii) The presence and position of any reinforcement.
(iii) The relative quality/density/void content of the material.
(iv) The bond between layers.
(v) The presence of detritus where there is a lack of bond between layers.
(vi) The presence and depth of any cracking or loose material.
(vii) Any missing layers (i.e. not recovered by coring).
(viii) Any unusual features.

When measuring layer thicknesses, the following points should be noted:

(i) Individual layer thicknesses should be recorded to the nearest 1mm.
(ii) Thicknesses should be measured at not less than 2 points diametrically opposite each other on the surface of the core, and an average taken.
(iii) The total thickness of each group of combined asphalt layers should be recorded in addition to the individual layer thicknesses.

I7.1.2.5 Each core should be photographed with a scale strip and with the core reference number clearly visible. A core log should include all of the above information and a colour photograph of the core, ideally 150mm by 100mm or greater.

I7.1.2.6 DCP tests should be carried out through the core hole to the maximum depth possible. If other forms of subgrade strength tests are used, any unbound materials must be tested after the bound materials have been removed, and then the unbound layers must be removed with an auger or other tools to establish their depth and allow strength tests on the subgrade.

I7.1.3 Trial Pits

I7.1.3.1 In certain situations, it may be advantageous to open a trial pit. In particular:

(i) Cores are not practicable in thick unbound constructions,
(ii) A carefully excavated trial pit can help provide information about an unexplained or unexpected pavement failure.

I7.1.3.2 However, due to the length of time needed to correctly excavate a trial pit plus the time for a suitable reinstatement to be completed, cores are generally preferred to trial pits. The location of trial pits will depend on the sort of deterioration present at the surface.

I7.1.3.3 When excavating trial pits, the following points should be noted:

(i) The normal plan size of a trial pit is 1.5m by 1m. Each side of the rectangle should be marked on the pavement surface prior to excavation.
(ii) If samples of bituminous material are required for laboratory testing, or if cracks in lower layers are being sought, the bituminous layers should be removed layer by layer.
(iii) If the pavement has a thick bituminous surfacing layer, and if deformation is present, it may be possible to determine which layers have deformed. In order to do this, a rotary saw needs to be used to obtain a clean cut face, which can be inspected, measured and photographed. A steel straight edge across the width of the pit can be used as a datum line.
(iv) The surface of each layer shall be closely examined before excavation is continued and the general appearance of each layer shall be noted. An air line is useful for clearing away detritus.
(v) DCP tests should be done through the full depth of any unbound materials into the subgrade, after removal of bound layers. For other forms of subgrade strength test the bound layers and each layer of unbound material must be carefully removed to expose the underlying layer down to the subgrade, to allow a strength test on each layer.
(vi) The condition of the unbound layers should also be noted and tests undertaken as appropriate and samples of material, including unbound materials, should be retained for future testing (see Section I7.4).

I7.2 Dynamic Cone Penetrometer

I7.2.1 The TRL Dynamic Cone Penetrometer (DCP) is a device used for the rapid in situ measurement of the strength of unbound granular pavement layers including subgrades. Layer thicknesses can also be determined if the layers are of different strengths. A typical test will take less than 10 minutes and measurements can be made to a depth of 0.8m or 1.2m if an extension rod is fitted. Further details on the device and its operation are given elsewhere.

I7.2.2 The DCP is shown diagrammatically in Figure 51. It has a 60° cone, which is driven through the pavement by an 8kg weight dropping through a distance of 575mm. The penetration of the cone is recorded using a steel rule attached to the base of the device. The DCP needs three operators: one to hold the instrument vertical, one to raise and drop the weight and one to record the number of blows and the depth of penetration.

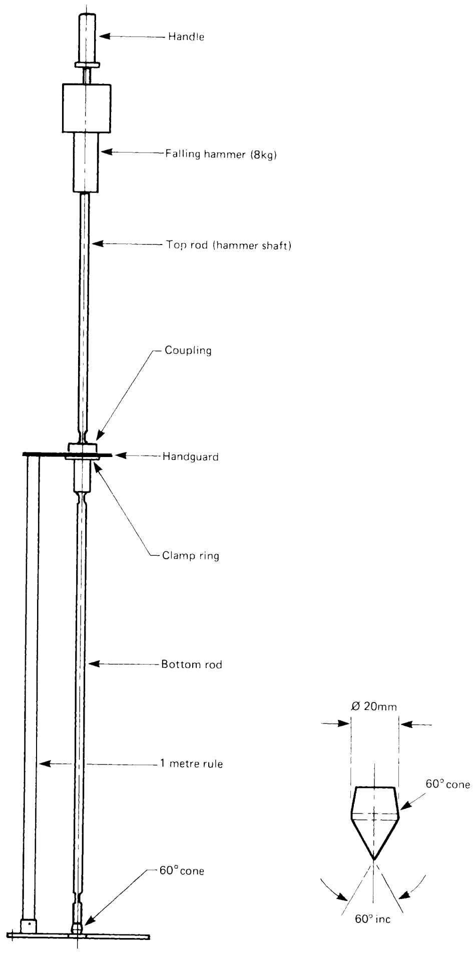


Figure 51 Dynamic Cone Penetrometer

I7.2.3 Pavement layers are identified by changes in the rate of penetration and the strength of each layer calculated from the gradient of the best-fit line through the points. The rate of penetration (per blow) can be related, through empirical relationships, to the in situ CBR of the material. Several different relationships exist which relate the mm/blow to CBR. The relationships produce very similar results over most of the range but differences are apparent at low values of CBR, especially for fine-grained materials. For these materials, it is expected that the DCP/CBR relationship will depend on material state. For tests on soils in the UK, it is recommended that the TRL relationship is used:

$$\mathrm{Log}_{10}\,(\mathrm{CBR}) = 2.48 - 1.057 \times \mathrm{Log}_{10}\,(\mathrm{mm/blow}) \qquad (10)$$

A typical Dynamic Cone Penetrometer test result, plotting the penetration per blow with depth and the CBR calculated for various homogenous layers, is shown in Figure 52.

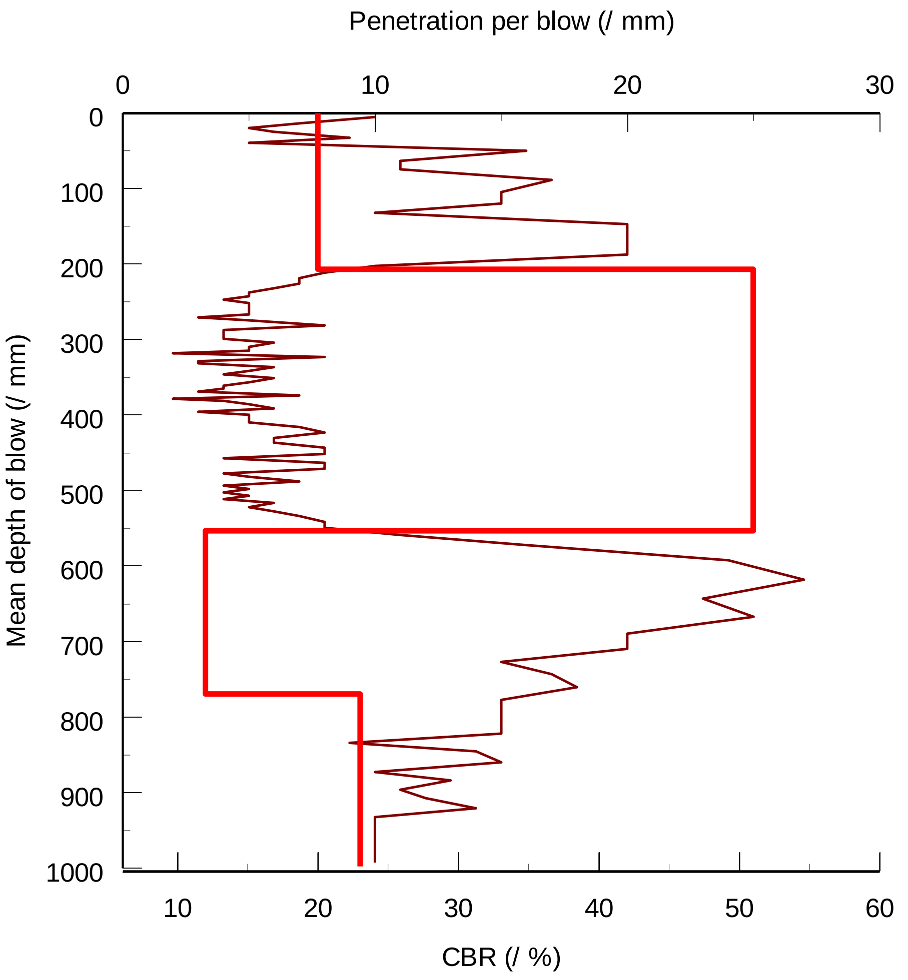


Figure 52 Typical Dynamic Cone Penetrometer test result.

I7.2.4 Although the DCP can be driven through very thin bituminous layers, tests on airfields will normally be undertaken through core holes or in trial pits.

I7.3 Other in situ soil tests

I7.3.1 Introduction

I7.3.1.1 There are a number of other techniques available for testing soils in situ. These include tests for determining density as well as alternative techniques to the DCP for determining strength. Details of these tests are given elsewhere and some of them are described in Chapter 3. The following section presents a brief summary of the main methods available.

I7.3.2 Density

I7.3.2.1 There are two main methods available for determining the density of unbound layers: the sand replacement test and the nuclear density gauge. The sand replacement test involves excavating and weighing material removed from a small hole and then refilling the hole with sand. The volume of the hole is calculated from the mass of the sand used. Although the test is time consuming, it is very accurate. A quicker method is the nuclear density gauge. However, the device does require careful calibration for each soil or aggregate tested.

I7.3.3 Strength Tests

I7.3.3.1 The CBR test (California Bearing Ratio) involves the insertion of a small plunger into the ground surface at a rate of 1mm per minute, whilst the load is recorded. The stress at penetrations of 2.5 and 5mm is compared with the result for a standard aggregate and the ratio expressed as a percentage. The test is not suitable for coarse aggregates because the plunger and aggregate particles are of similar size. There are several disadvantages to using the CBR test in situ and it is recommended that alternative in situ methods (e.g. DCP) are used to determine estimates of CBR.

I7.3.3.2 The Clegg Hammer test uses a hammer/plunger to dynamically load the soil, the deceleration on impact being recorded to give a "Clegg Impact Value". The Clegg Impact Value can, for soils dry of the optimum moisture content, be related approximately to CBR. The device is also useful for detecting soft spots on a subgrade and for differentiating between material types.

I7.3.3.3 Cone penetrometers can be used to rapidly estimate CBR in soft and medium fine grained subgrades. Various sizes of these instruments exist although they are only applicable up to CBR values of 5-6%.

Static plate bearing tests can provide an excellent indication of soil strength and can be used directly to determine the modulus of subgrade reaction (k) using a 762mm diameter plate. The plate is loaded to give increments of deflection of 0.25mm. The pressure on the plate is plotted against settlement and the k value is taken as the slope of the line passing through the origin and the point on the curve corresponding to 1.27mm deflection. However, because of the large diameter of the plates needed for this test (circa 300 to 762mm), it is only carried out in very large diameter core holes or in trial pits.

I7.3.3.4 Dynamic plate bearing tests can also be used to derive estimates of soil strength. These tests involve dropping a weight onto a plate and then measuring either the peak deflection under the plate or, as in the case of the Dynaplaque, the rebound of spring weights. An example of this type of device is the FWD (as described in Section 4 of this Note) but there are a number of smaller, hand held devices, which are specifically designed for testing unbound materials.

I7.4 Laboratory tests

I7.4.1 Introduction

I7.4.1.1 In certain situations, it may be necessary to carry out detailed evaluation of materials recovered from site. Laboratory tests on pavement materials can be divided into three categories according to the type of material being evaluated i.e. bituminous materials, cement bound materials and unbound materials. This Section provides a brief summary of the main tests that are available for each of these material types.

I7.4.2 Bituminous Materials

I7.4.2.1 The elastic stiffness of bituminous materials can be determined using the Indirect Tensile Stiffness Modulus test (ITSM) using the Nottingham Asphalt Tester (NAT). ITSM values can be used to help judge the quality of material as well as to assess the load spreading ability of the material. The values can also be used to confirm or explain FWD-derived layer stiffnesses. As the stiffness of bituminous materials depends on loading time, the shorter pulse of the FWD results in stiffnesses greater than ITSM values. Therefore ITSM values at 20°C should be multiplied by 1.5 to allow comparison with FWD-derived bituminous layer stiffnesses at 20°C.

I7.4.2.2 The Wheel Tracking Test and the Repeated Load Axial Test (RLAT) can be used to provide a relative indication of the rut susceptibility of bituminous materials.

I7.4.2.3 Inadequate compaction of bituminous materials can often give rise to problems such as deformation, high permeability and low stiffness. The bulk density of bituminous materials can be measured by weighing in air and water. Used in combination with the maximum theoretical density, volumetric proportions may be calculated. For DBM materials, the Percentage Refusal Density (PRD) test can be used to compare the achieved density with the maximum achievable density.

I7.4.2.4 Further information about the composition of bituminous materials can be obtained by carrying out a particle size distribution, binder content and by determining the properties of the recovered bituminous binder. These tests are likely to be useful when helping to explain pavement failures.

I7.4.3 Cement Bound Materials

I7.4.3.1 The most important property of concrete in relation to its performance in airfield pavements is the flexural strength. This is most easily deduced from compressive strength tests undertaken on cylindrical samples recovered from cores. Details of this test, and other methods for testing concrete strength, are described in detail in BS 1881. Other properties that can be measured include density, elastic stiffness and compositional analysis of the concrete including cement content, aggregate content and fine/coarse aggregate ratio. More information about these specialist tests is given elsewhere (Neville, 1996).

I7.4.4 Unbound Materials

I7.4.4.1 Laboratory testing of the unbound layers is not generally necessary as the in situ tests should normally be sufficient to indicate condition. However, it may sometimes be advantageous to determine the properties of unbound materials present, for example, to help to explain the reasons for particularly high or low stiffness. The most useful tests are:

(i) Grading
(ii) Liquid limit
(iii) Plastic limit
(iv) Moisture content
(v) CBR
(vi) Density.

I8 INTERPRETATION OF RESULTS FROM SITE INVESTIGATIONS

I8.1 General

I8.1.1 This section describes how the results of a site investigation should be interpreted to determine the inputs required for an evaluation of the pavement strength and residual life by reverse design, and for the design of rehabilitation or strengthening measures.

I8.1.2 The process is summarised in Figure 53

I8.2 Visual inspection

I8.2.1 General

I8.1.1.1 The results of the visual inspection should be used to:

(i) Assist in the assessment of condition factors for evaluation and the design of strengthening requirements.
(ii) Assess material and pavement condition
(iii) Provide a reality check for the reverse design.

In assessing the structural condition reference should be made to the failure criteria described in Appendix F (paragraphs F2.2 and F3.2).

I8.2.2 Condition factors

I8.2.2.1 The type and amount of cracking in concrete pavements can be related to the Condition Factors for evaluation and overlay design of composite and multiple slab pavements described in Chapter 7 (Sections 7.9 and 7.10, as described in paragraph 7.3.1).

I8.2.3 Pavement and material condition

I8.2.3.1 A number of distresses recorded by a visual inspection are an indication of the structural condition or behaviour of the pavement or individual layers, including the following:

(i) Cracks in rigid pavements and rutting and/or cracking in flexible pavements in the wheel track areas indicate the onset of structural failure (see paragraphs 7.3.1.5 and 7.3.1.6).

(ii) Measurement of rutting and associated heave can be used to assess the condition of flexible pavements (paragraph 7.3.1.6).

(iii) Rut width compared to the track of the most damaging user aircraft may provide an indication of where failure is occurring in a flexible pavement with unbound granular layers. The narrower the rut width the higher in the pavement the problem is likely to be.

(iv) Significant rutting without associated heave suggests densification of bituminous layers, unbound granular materials or the subgrade due to inadequate compaction for the loading. With time the densification will cease as the relative density of the layer becomes adequate for the loading, but until that point there is no cure other than to remove the layer, or put on a thick overlay to reduce stresses in the layer to a more acceptable level.

(v) Cracking in a bituminous surfacing which has an underlying concrete construction may be a reflection of the bay pattern of the concrete or may indicate that the underlying bays have undergone some structural cracking.

(vi) Some forms of cracking can occur in bituminous surfaces for non-structural reasons. These can usually be found randomly distributed over the pavement area, not just in the wheel tracks. The most common of these is “block” or “age” cracking, which produces a block pattern formed by short longitudinal and transverse cracks. These are caused by a combination of the binder stiffening with age and temperature effects in the bituminous layers. Although not related to structural behaviour they will reduce the structural capacity of the bituminous layers.

(vii) Mud-pumping indicates the presence of a saturated fine grained subgrade and relatively large deflections under loading causing fine material to pump up through joints or cracks in the pavement. In a flexible pavement the mud-pumping will indicate full-depth cracking of bituminous layers. If unbound granular layers are present mud-pumping indicates contamination by fine materials with a probable loss of structural capacity. If a bound base/sub-base is supposed to be present in the pavement mud-pumping will indicate severe deterioration of that layer.

(viii) Blisters are probably formed by the heating of water vapour trapped in asphalt layers with a high void content, which have been overlaid with impervious asphalt layers. If the voids are large and interconnected (e.g. open macadams) blistering does not occur as the water vapour has room for expansion; and once the overlay becomes thick enough blistering will be reduced because of the insulating properties of the overlaying asphalt. Once the crack occurs the blistering ceases to occur, but may recommence if the surface is overlaid, sealing the cracks. Unless the layer causing the blistering is removed there is no proven cure other than to puncture blisters as they occur. It may be possible to prevent the recurrence of blistering by laying an open textured material on top of the existing surface before overlaying in dense bituminous materials. Although unsightly there is no evidence that most blisters or the associated cracking significantly affect serviceability or pavement life. In the event of this defect occurring, the need or otherwise to carry out major remedial/refurbishment work will depend on its extent and severity and also the nature of aircraft operations.

I8.2.4 Reality check

I8.2.4.1 When a pavement structural investigation and evaluation has been undertaken, a reality check should be carried out against the visual inspection. If there is an obvious mismatch the information on use should be re-assessed, and then if necessary the interpretation of the structural testing and the reverse design should be re-done.

I8.2.4.2 Pavement condition must be related to previous use. For instance, if a pavement classified as PCN 50/F/A fails under regular ACN 40 (Flexible High Subgrade) use only, then the classification is obviously wrong, and the pavement will need strengthening as well as restoration of fatigue life if future PCN 50/F/A use is intended.

I8.3 Coring or trial pits

I8.3.1 Layer thicknesses measured in cores or trial pits should used to determine the pavement thicknesses for reverse design, as follows:

(i) If there are several cores or trial pits in a section of pavement, calculate a design thickness from the mean plus one half standard deviation of the thicknesses for each layer (if several bituminous layers are grouped in the pavement construction, the overall thickness of the group should be calculated).
(ii) If the core or trial pit location has been selected as representative of a section by analysis of FWD results, use the layer thicknesses from that core or trial pit for the whole of the section.
(iii) If the core or trial pit measurements have been used to calibrate GPR results, use the GPR results.

I8.4 Subgrade strength tests

I8.4.1 Dynamic Cone Penetrometer

I8.4.1.1 A Dynamic Cone Penetrometer test can be used to:

(i) Assess regions of homogenous behaviour with depth.
(ii) Assess the depth and strength of unbound bases and sub-bases.
(iii) Assess the strength of the subgrade, and detect weak layers within the depth of the test.

I8.4.1.2 The CBR should generally be taken as the lowest in a layer. When several results in a pavement section are available the design value may be calculated as the mean minus one half standard deviation of the results for a layer. Care should be taken to eliminate outliers, i.e. high or very low strengths. If there are a few tests with a wide scatter, a simple visual assessment will be safer than a statistical analysis; generally selecting the lowest result or a result close to the lowest.

I8.4.1.3 The thickness of unbound bases and sub-bases should be determined by plotting CBR against depth to find the thickness of materials with CBRs greater than the minimum values required for Granular Base (100%) and Granular Sub-base (30%)

I8.4.1.4 For flexible pavements, if there is a significant depth of subgrade above the depth at which the lowest CBR occurs, with a strength greater than the design CBR, but not great enough to be treated as Granular Sub-base, it may be treated as subgrade improvement in accordance with Chapter 3 (Section 3.8).

I8.4.2 Other tests

I8.4.2.1 Methods of testing other than the Dynamic Cone Penetrometer will give a single strength measurement. If there are unbound bases and sub-bases, separate tests will be required on each layer as well as the subgrade.

I8.4.2.2 The design strength of each layer should be taken as the mean minus one half standard deviation of the tests in a section of pavement. Care should be taken to eliminate outliers, i.e. very high or very low strengths. If there are a few tests with a wide scatter, a simple visual assessment will be safer than a statistical analysis; generally selecting the lowest result or a result close to the lowest.

I8.4.2.3 The thickness of unbound bases and sub-bases should be determined as described in Section I8.3.

I8..5 Compressive strength tests

I8.5.1 The flexural strength of concrete should be based on the mean estimated cube compressive strength of the recovered core samples, for all concrete of the same age in the pavements being tested.

I8.5.2 To assess the flexural strength of the concrete, an approximate but expedient method is to a flexural to compressive strength relationship of 1:10. The ratio actually varies depending on the coarse aggregate type and concrete strength, decreasing with less elongated and more rounded aggregates (e.g. gravels) and with increasing compressive strength. Where evidence exists to show that an alternative cube strength to flexural strength ratio is appropriate this should be used.

I8.5.3 When using Charts 1 to 4 concerte strengths measured more than 2 years after construction should be divided by 1.18 to allow for the growth in strength build into the design graphs.

I8.6 Falling Weight Deflectometer results

I8.6.1 General

I8.6.1.1 FWD testing can be used in three ways:

(i) To determine homogenous pavement sections and select representative core or trial pit locations.
(ii) To measure load transfer at joints (Sections I4.4.8.1 and I3.5.1).

To estimate layer elastic stiffnesses, giving an assessment of material condition (Sections I4.4.8 I3.5.2).

I8.6.2 Homogenous pavement sections

I8.6.2.1 Statistical techniques for the determination of homogenous sections and representative test locations are described in Appendix C.

I8.6.3 Load transfer

I8.6.3.1 The design load transfer is the mean plus one half standard deviation of the measurements in a homogenous pavement section. Outliers, such as expansion joints, should be excluded from the calculation.

I8.6.3.2 Chapter 7 does not cover the effect of load transfer at joints on the pavement strength. If the measured load transfers fall below the values given in Appendix F (Table 27) then the pavement is unlikely to reach its design life and an additional overlay thickness should be allowed when strengthening.

I8.6.4 Back-analysis of layer elastic stiffnesses

I8.6.4.1 Layer elastic stiffnesses back-analysed from FWD deflection basin measurements may be used to assess material condition, including Condition Factors using the reference values for material condition listed in Table 7. This comparison should normally be done on representative mean values for homogenous pavement sections. When evaluating rigid pavements the effect of temperature gradients must be considered (see paragraph I4.4.4.3).

I8.6.4.2 The Condition Factors given in Table I8.1 relate to the Condition Factors described in Chapter 7.

	Material Type									
Condition	Pavement Quality Concrete		Drylean Concrete		Bituminous bound (20°C)		Sub-base (interlayer)		Subgrade	
	Condition Factor	Stiffness Range (MPa)	Condition Factor	Stiffness Range (MPa)	Condition Factor	Stiffness Range (MPa)	Condition Factor	Stiffness Range (MPa)	Condition Factor	Stiffness Range (MPa)
Poor	-> Drylean Concrete	<10,000	-> Granular Sub-base	<3000	-> Granular Base Course	<1000	Use CBR / k for evaluation	<100	Use CBR / k for evaluation	<100
Average	C_t = 0.85 C_i = 0.75	10,000 - 20,000	1.0	3000 - 8000	1.0	1000 - 4000		-		-
Good	C_t = 1.0 C_i = 1.0	20,000 - 30,000		8000 - 15,000	1.0	4000 - 7000		100 - 200		100 - 200
Excellent		>30,000		>15,000		>7000		>200		>200

Table I1 Stiffness values to be used for assessing material quality.

I8.7 Ground penetrating radar results

I8.7.1 GPR results can be used in two ways:

(i) As discrete results at FWD test locations for the back-analysis of the FWD results.
(ii) To give a design thickness for each layer in a homogenous section of the pavement, calculated as the mean minus one half standard deviation of all measurements in the section.

I8.8 Integration of results

I8.8.1 A flow diagram showing how the interpreted test results are integrated to provide information for the evaluation of the pavement and calculation of rehabilitation or strengthening requirements is given in Figure 53.

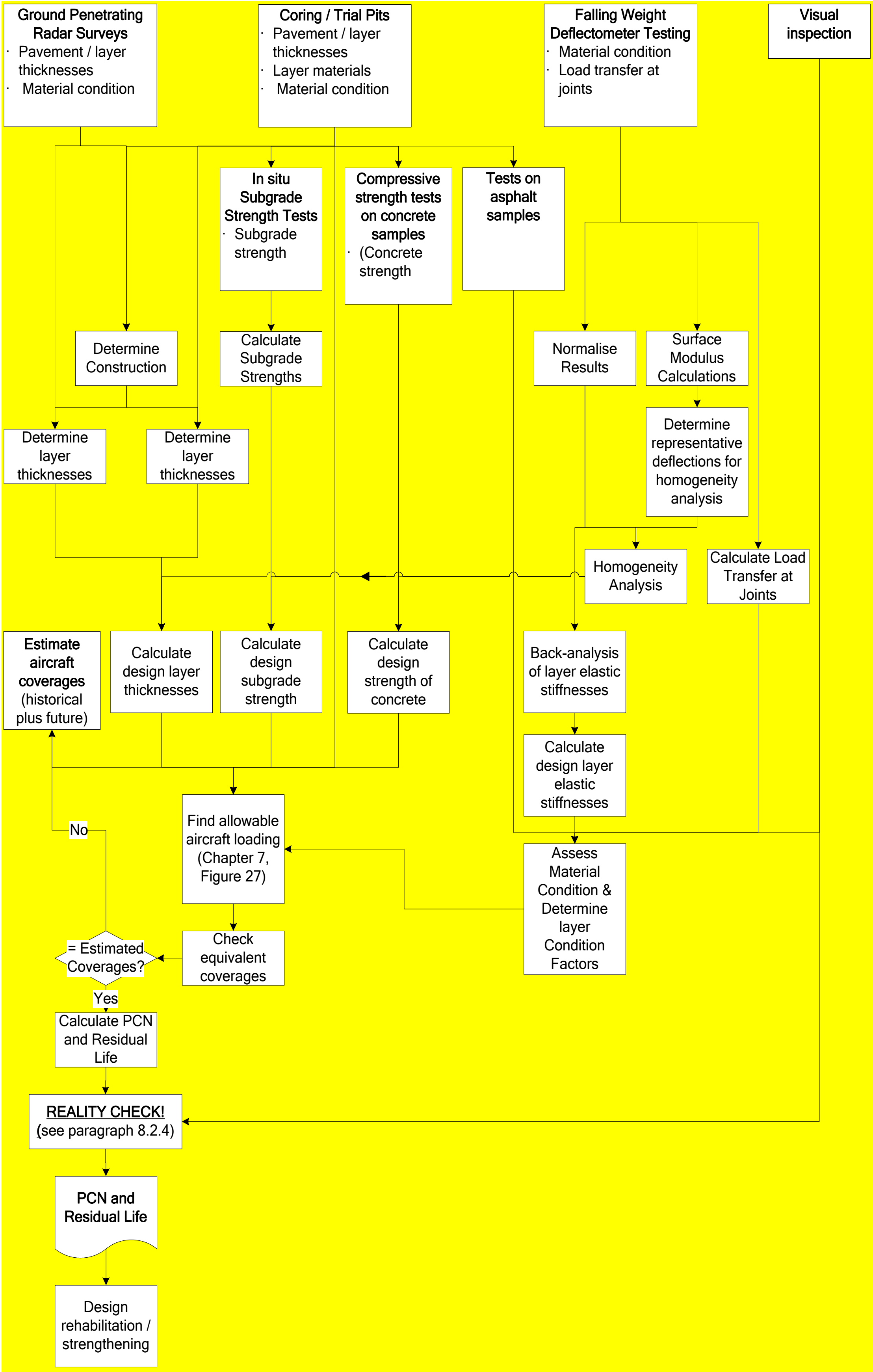


Figure 53 Standard techniques for structural investigations and interpretation and integration of test results.

I8.9 Reporting

I8.9.1 A report on a pavement investigation should include the following:

(i) The results of prior data collection.
(ii) A description of the investigation, including the pavements tested, the components of the investigation and test frequencies.
(iii) The results of the visual inspection.
(iv) A summary of the results, including the Construction Locations and homogenous pavement sections determined, the design values for the layer thicknesses, layer elastic stiffnesses, subgrade strengths, concrete strengths and load transfers at joints.
(v) A commentary on any features of the testing, including material conditions in cores and subgrade types.

I8.9.2 Appendices should be included for relevant tests giving:

(i) A description of the test methods, methods of statistical analysis for homogeneity and back-analysis procedures including a description of back-analysis software and its compliance with paragraph I4.4.5.1.
(ii) Previous construction records, if known.
(iii) FWD test measurements.
(iv) Results of any statistical analysis for determination of homogenous pavement sections.
(v) Results of any back-analysis of FWD measurements, including layer elastic stiffnesses, measured and calculated deflection basins and Absolute Mean Deviation and Root Mean Square deviation for each test.
(vi) Results of load transfers calculated from FWD tests.
(vii) Core logs.
(viii) Results of compressive strength tests on cores.
(ix) Dynamic Cone Penetrometer tests measurements, or measurements from other test methods.
(x) Subgrade strength results determined from the test measurements.
(xi) Construction records as determined by the investigation.
(xii) Drawings showing the pavements tested, test locations, Construction Locations and Homogenous Sections.

I9 REFERENCES AND BIBLIOGRAPHY

1. BSI. *In situ tests*. BS1377; Part 9. BSI. London.
2. BSI. *Methods of testing concrete.* BS 1881. BSI, London.
3. BURMISTER. *The general theory of stresses and displacements in layered soil systems*. Journal of applied physics, Volume 16. 1945.
4. FEHRL. *Harmonisation of the Use of the Falling Weight Deflectometer on Pavements Part 1 – Harmonisation of FWD Measurements and Data Processing for Flexible Road Pavement Evaluation.* FEHRL Report No. 1996/1.TRL, Crowthorne, 1996.
5. HIGHWAYS AGENCY et al. *The Design Manual for Roads and Bridges - Volume 7 Pavement Design and Maintenance.* The Stationary Office Limited. London.
6. KENNEDY C K, FEVRE P and CLARKE C S. *Pavement deflection: equipment for measurement in the United Kingdom.* LR 834. TRRL, Crowthorne. 1978.
7. NEVILLE M A. *Properties of Concrete - 4th Edition*. Longman Scientific & Technical, Harlow, 1996.
8. PSA. *A Guide to Airfield Pavement Design and Evaluation*. PSA, 1989.
9. TRL. *A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries*. Overseas Road Note 31. TRL, Crowthorne. 1993.
10. TRL. *Dynamic Cone Penetrometer (DCP)*. Overseas Technical Advice Note 3. TRL, Crowthorne. 1998.

I10 ANNEX A - FWD TEST PROCEDURE

I10.1 Machine configuration and test locations

I10.1.1 General

I10.1.1.1 Testing should normally be undertaken at a nominal load of 100kN using a 300mm-diameter plate. Most FWDs in the UK have a 60Hz "smoothing" filter option. The use of this filter has been shown to improve the agreement between machines and should be activated. The peak contact pressure should be recorded to a resolution of 1kPa or better. Peak deflections should be measured to a resolution of 1 micron or better.

I10.1.1.2 There should be no standing water on the pavement surface and care should be taken to ensure that the whole area of the plate is in contact with the surface. A segmented plate can assist with this on uneven or rutted surfaces. At each test location, three drops, plus a small initial drop to settle the loading plate, should be made. The results for all three drops should be measured and stored for later analysis. In addition, the FWD operator should check that the results at each test location are reasonably self-consistent.

I10.1.1.3 Tests on rigid and composite pavements should normally be carried out at 25-30m intervals either on mid-slab positions (for stiffness evaluation) or at joints (load transfer). Where the pavement is wide enough, measurements should be taken in and outside the wheel track areas, either side of the centreline, to detect any structural deterioration due to trafficking. If the pavement is too narrow for reliable results to be achieved in untrafficked areas then only two lines are required. On flexible pavements, measurements should normally be taken at nominally 20m to 30m intervals.

I10.1.2 Stiffness Evaluation

I10.1.2.1 On rigid or composite pavements, tests should be undertaken with the loading plate located in the centre of the slab and away from cracks. Peak deflections should be measured at a minimum of seven radial positions, as specified in Table 31. The deflections at each of these positions are usually referred to as d1 to d7.

Table 31 Recommended FWD geophone positions for stiffness evaluation testing (7 Sensors)

Geophone Number	d1	D2	d3	d4	d5	d6	d7
Radial Position (mm)	0	300	600	900	1350	1800	2250

I10.1.2.2 On thinner, weaker pavements, the peak FWD testing load may need to be reduced to 50kN (or less) to avoid permanent damage to the pavement.

I10.1.3 Load Transfer

I10.1.3.1 The preferred testing arrangement for evaluating load transfer is to position the 300mm diameter plate with its centre 250mm from the joint and with geophones located 50mm either side of the joint i.e. 200mm and 300mm from the centre of the loading plate. This may be accomplished by rearranging the positions of the geophones in front of the load plate, by using additional geophones in front of the plate or by using an extension bar which has geophones mounted on it, behind the plate. Care needs to be taken to avoid spalled material and geophone positions may need to be adjusted to allow for this.

I10.1.3.2 In trafficked areas, ideally, the slab downstream of the joint should be loaded as the downstream side of the joint is generally the weaker side. If testing in the direction of trafficking, this may be achieved using an extension bar.

I10.2 Temperature

I10.2.1 General

I10.2.1.1 The temperature of the pavement should be measured at the time of testing using an electronic thermometer accurate to 0.5°C and with a resolution of 0.1°C. Temperature measurement holes should be pre-drilled some time before the measurement so that the heat created by drilling has had time to dissipate. Glycerol, or similar liquid, in the bottom of the hole will ensure good thermal contact between the thermometer and the bound material. Temperatures should be measured at the start of testing and at least every 30 minutes during testing. Measurements should also be recorded when passing into or out of continuously shaded areas and when moving between sections with different surface materials (e.g. asphalt to concrete).

I10.2.1.2 Temperature should normally be recorded at a depth of 100mm. For pavements where the thickness of the bituminous bound layer exceeds 200mm, it is recommended that additional temperature measurements are made at nominally 2/3 the bituminous layer depth. It is important that depths are measured and recorded along with the times of the measurements. Care should be taken during and after prolonged spells of cold whether as ice may be present in the unbound materials which can significantly affect results.

I10.2.2 Stiffness Evaluation

I10.2.2.1 On rigid pavements, composite pavements and flexible pavements with a strong cement-bound base, where the condition of the concrete layer is the primary concern, it is recommended that testing be carried out below 15°C to allow any cracks or joints that may be present in the concrete layer to open up.

I10.2.2.2 The stiffness of bituminous materials depends on temperature and should be adjusted to the reference temperature of 20°C to allow comparison to be made with reference values. However this temperature adjustment becomes increasingly uncertain at temperatures significantly above or below 20°C. Therefore on pavements with thick bituminous layers and/or where the survey is primarily targeted at determining the condition of the bituminous layer, testing should ideally be undertaken between 15 and 25°C.

I10.2.3 Load Transfer

I10.2.3.1 Temperature can have a significant effect on the measured load transfer efficiency. Testing should be undertaken at relatively low temperatures (<15°C) where the joints are likely to be more open and the degree of movement more severe.

I11 ANNEX B - BENKLEMAN BEAM TEST PROCEDURES

I11.1 Operation

I11.1.1 The Benkleman Beam can be operated by one very hard working person, plus the vehicle driver. However, it is probably most efficiently operated by a team of four people; one to set-up and operate the beam, one to measure out the test locations, the third to help with both jobs, and the driver

I11.1.2 Apart from the Deflection Beam the only major item of equipment required is a vehicle. For comparative measurements of deflection the actual axle load is not important, although it is vital that the load is the same throughout the testing of a particular pavement e.g. sand/gravel ballast should be sheeted to prevent it getting wet. However, it is important that the load is sufficient to cause large enough deflections. A mean reading of 0.015mm should be acceptable, and this should be achievable with a 10 tonne axle load on flexible pavements with PCN values less than 20. It is known that aircraft have been used to provide the loading for Deflection Beam tests, but obtaining an aircraft for the duration of a test programme could be difficult and possibly expensive.

I11.1.3 The Deflection Beam designed to be used by placing it between the two wheels of a closely spaced dual wheel assembly, as found on the rear axle of a normal lorry. This is because the maximum deflection will occur on the centreline of closely spaced dual wheels and can therefore be measured. However, such a set-up may not be obtainable on vehicles with a sufficiently high axle load. In this case perfectly acceptable results can be obtained by placing the Deflection Beam alongside or behind a single wheel. Results will be consistent as long as the position of the beam is consistent.

I11.1.4 There are two methods used for taking the actual measurements:

(i) Measurement of the deflection as the load moves past the measurement point.
(ii) Measurement of the rebound as a parked load moves away from the measurement point.

Method i) is the standard method used in the UK. Method ii) is standard practice on the Continent, and is easier and faster to carry out.

I11.1.5 Temperature measurements should be undertaken in accordance with the guidance for the FWD given in Appendix A. The effects of temperature can be minimised by carrying out surveys at a time when temperature variations are at a minimum e.g. at night or on overcast winter days. Temperature variations will have the largest effect on pavements with considerable thicknesses of bituminous materials. To obtain consistent results the time for which the vehicle is left standing at each test point should be kept as constant as possible.

I12 ANNEX C – DETERMINING HOMOGENEOUS SECTIONS

I12.1 Introduction

I12.1.1 A homogeneous section is a length of pavement in which the measured parameter (e.g. deflection, stiffness, layer thickness) is reasonably consistent. There are a number of different techniques that can be used to split a site into homogeneous sections. These range from the visual assessment of results through to complicated statistical analysis. This appendix describes a relatively quick and straightforward method based on the cumulative sum of variables (CUSUMS), which is recommended for use on MOD airfield pavements.

I12.2 Cumulative sums method

I12.2.1 The cumulative sum is calculated using the following formulae:

$$S_1 = x_1 - x_m \qquad (10)$$

$$S_2 = x_2 - x_m + S_1 \qquad (11)$$

$$S_i = x_i - x_m + S_{i-1} \qquad (12)$$

Where

x_i = parameter value (e.g. deflection) measured at test point i

x_m = mean parameter value (e.g. deflection) of each main section

S_i = cumulative sum of the deviations from the mean deflection at test point i.

I12.2.2 The cumulative sums should be plotted against chainage (or bay number) on a chart known as a Cusum chart. Changes in slope on the Cusum chart indicate in homogeneity. A positive gradient indicates a section with values greater than the mean and a negative gradient indicates a section with values below the mean. A horizontal sequence of points indicates that the mean value of the parameter over that sequence is equal to the overall mean. The Cusum chart may be inspected visually to determine changes in slope and to split the site into sections.

I12.2.2.3 Figure 54 shows FWD central deflections (d1) along a 2.85km stretch of pavement. The corresponding Cusum chart is shown in Figure 55. Using the changes in gradients from the chart, the pavement has been split into 12 distinct sections.

I12.2.2.4 After delineation into homogeneous sections, it is advisable to check if there is a statistically significant difference between the sections. If there is not, they can be considered as a single section. Student's t-test can be used to check for statistical differences.

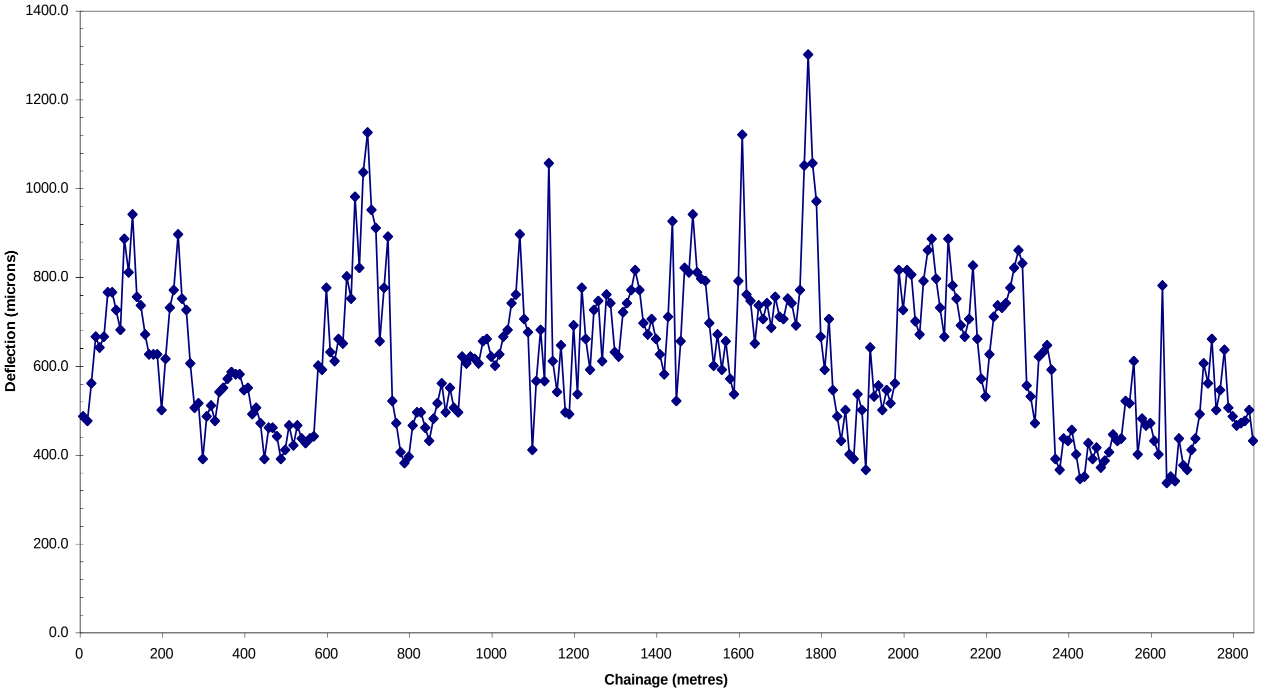


Figure 54 Example Deflection Profile

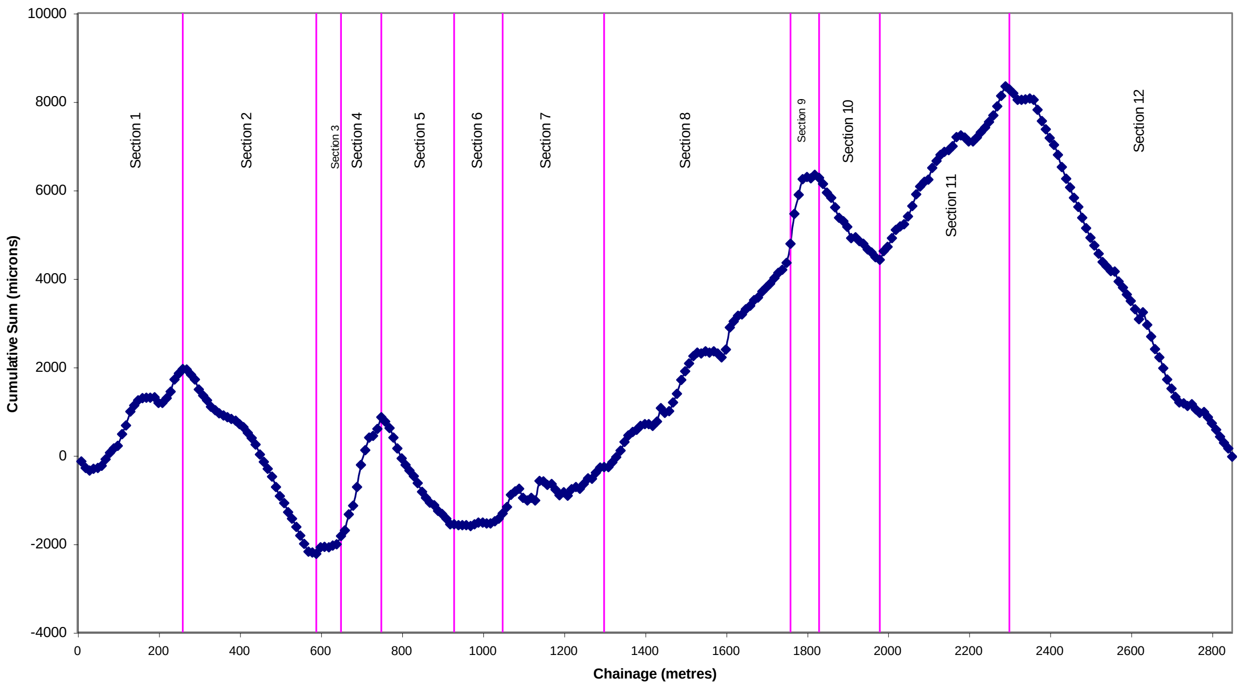


Figure 55 Cusum Chart (from **Figure 54** profile)

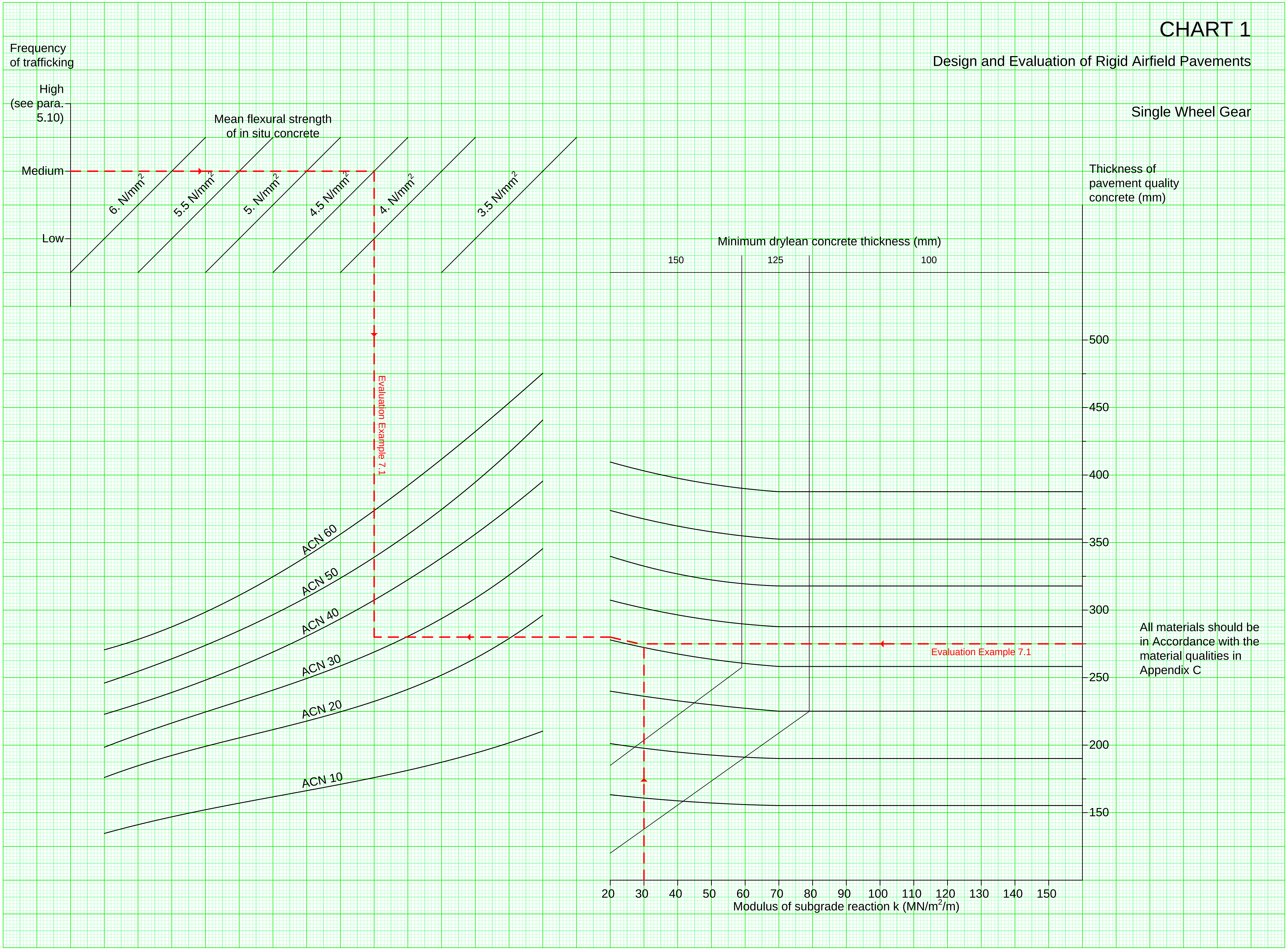


CHART 1
Design and Evaluation of Rigid Airfield Pavements
Single Wheel Gear
Frequency
of trafficking
High
(see para.
5.10)
Medium
Low
Mean flexural strength
of in situ concrete
6. N/mm²
5.5 N/mm²
5. N/mm²
4.5 N/mm²
4. N/mm²
3.5 N/mm²
Evaluation Example 7.1
ACN 60
ACN 50
ACN 40
ACN 30
ACN 20
ACN 10
Minimum drylean concrete thickness (mm)
150
125
100
Thickness of
pavement quality
concrete (mm)
500
450
400
350
300
250
200
150
Evaluation Example 7.1
All materials should be
in Accordance with the
material qualities in
Appendix C
20 30 40 50 60 70 80 90 100 110 120 130 140 150
Modulus of subgrade reaction k (MN/m²/m)

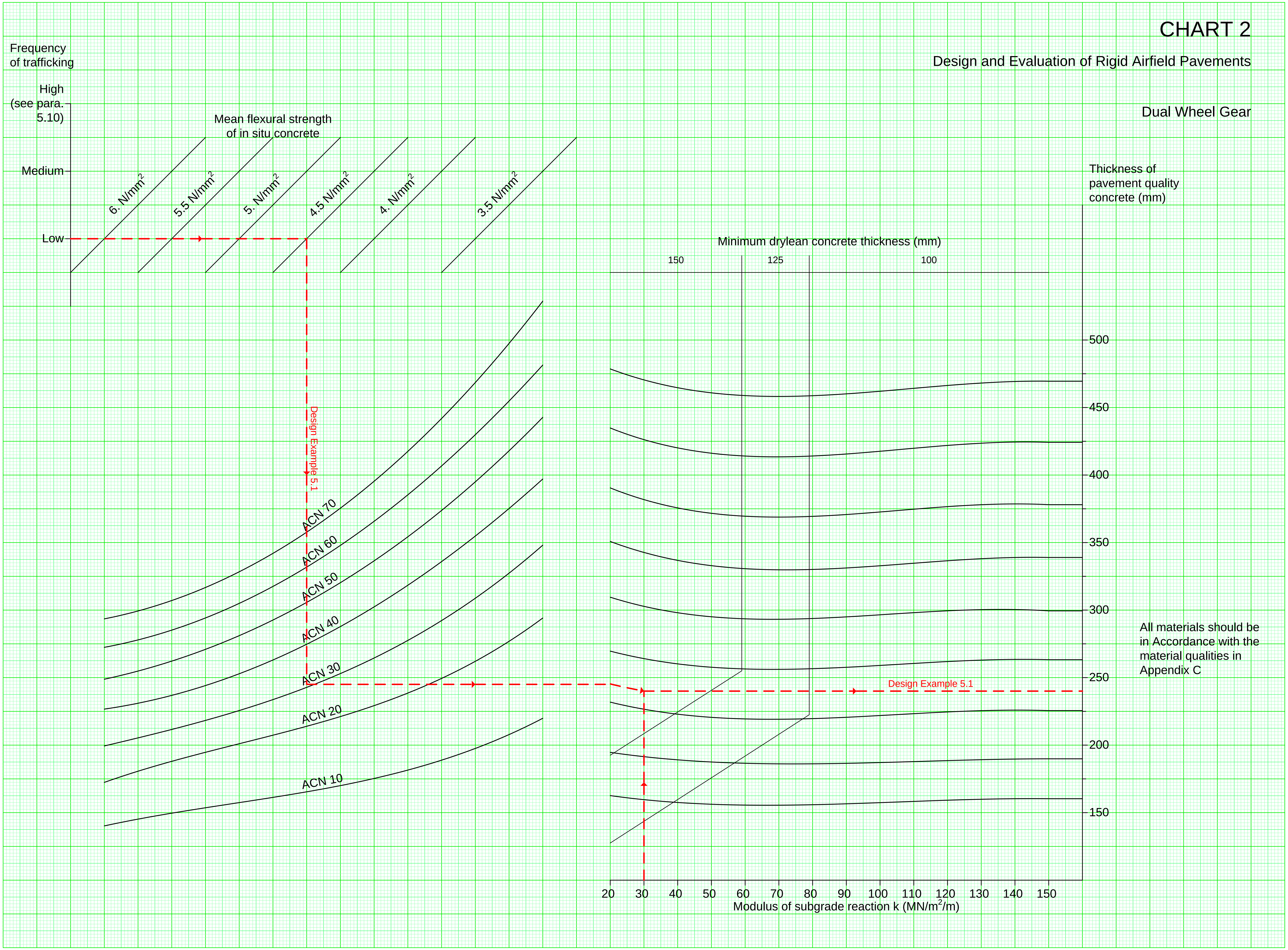


CHART 2
Design and Evaluation of Rigid Airfield Pavements
Dual Wheel Gear
Frequency
of trafficking
High
(see para.
5.10)
Medium
Low
Mean flexural strength
of in situ concrete
6. N/mm2
5.5 N/mm2
5. N/mm2
4.5 N/mm2
4. N/mm2
3.5 N/mm2
Design Example 5.1
ACN 70
ACN 60
ACN 50
ACN 40
ACN 30
ACN 20
ACN 10
Minimum drylean concrete thickness (mm)
150
125
100
Thickness of
pavement quality
concrete (mm)
500
450
400
350
300
250
200
150
Design Example 5.1
All materials should be
in Accordance with the
material qualities in
Appendix C
20 30 40 50 60 70 80 90 100 110 120 130 140 150
Modulus of subgrade reaction k (MN/m2/m)

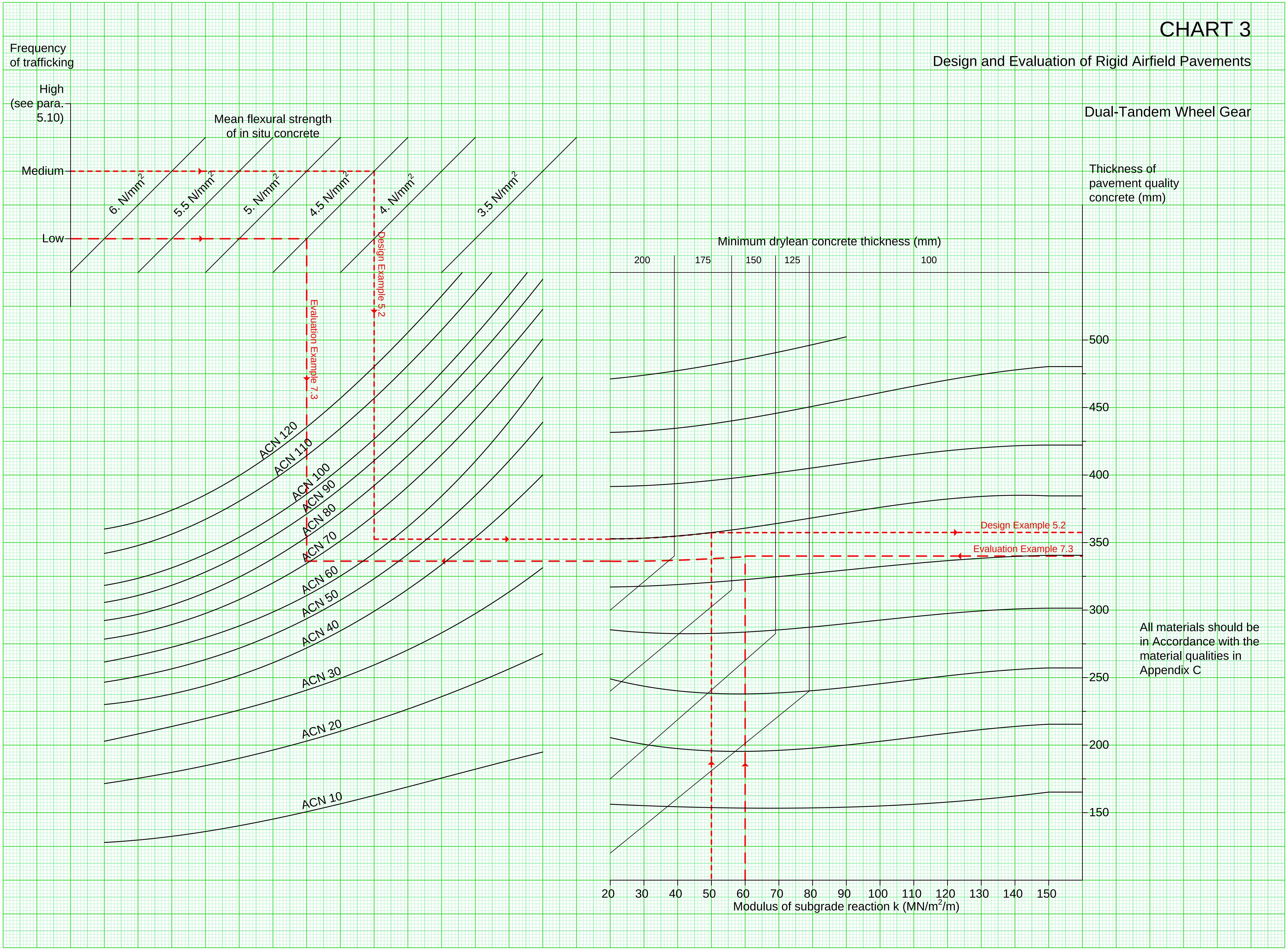


CHART 3
Design and Evaluation of Rigid Airfield Pavements
Dual-Tandem Wheel Gear
Frequency of trafficking
High (see para. 5.10)
Medium
Low
Mean flexural strength of in situ concrete
6. N/mm2
5.5 N/mm2
5. N/mm2
4.5 N/mm2
4. N/mm2
3.5 N/mm2
Design Example 5.2
Evaluation Example 7.3
ACN 120
ACN 110
ACN 100
ACN 90
ACN 80
ACN 70
ACN 60
ACN 50
ACN 40
ACN 30
ACN 20
ACN 10
Minimum drylean concrete thickness (mm)
200
175
150
125
100
Thickness of pavement quality concrete (mm)
500
450
400
350
300
250
200
150
Design Example 5.2
Evaluation Example 7.3
20 30 40 50 60 70 80 90 100 110 120 130 140 150
Modulus of subgrade reaction k (MN/m2/m)
All materials should be in Accordance with the material qualities in Appendix C

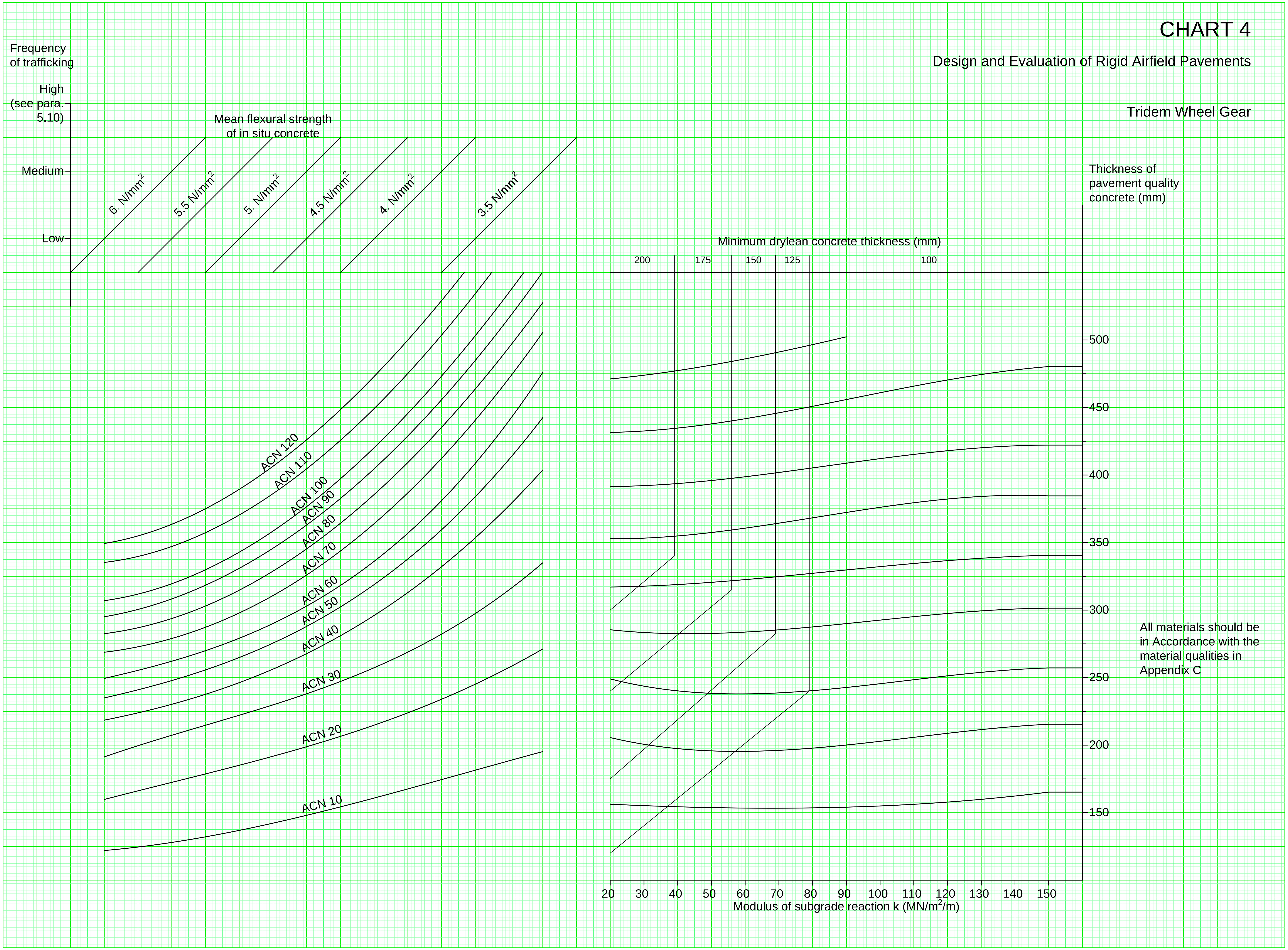


CHART 4
Design and Evaluation of Rigid Airfield Pavements
Tridem Wheel Gear
Frequency
of trafficking
High
(see para.
5.10)
Medium
Low
Mean flexural strength
of in situ concrete
6. N/mm2
5.5 N/mm2
5. N/mm2
4.5 N/mm2
4. N/mm2
3.5 N/mm2
ACN 120
ACN 110
ACN 100
ACN 90
ACN 80
ACN 70
ACN 60
ACN 50
ACN 40
ACN 30
ACN 20
ACN 10
Minimum drylean concrete thickness (mm)
200
175
150
125
100
Thickness of
pavement quality
concrete (mm)
500
450
400
350
300
250
200
150
20 30 40 50 60 70 80 90 100 110 120 130 140 150
Modulus of subgrade reaction k (MN/m2/m)
All materials should be
in Accordance with the
material qualities in
Appendix C

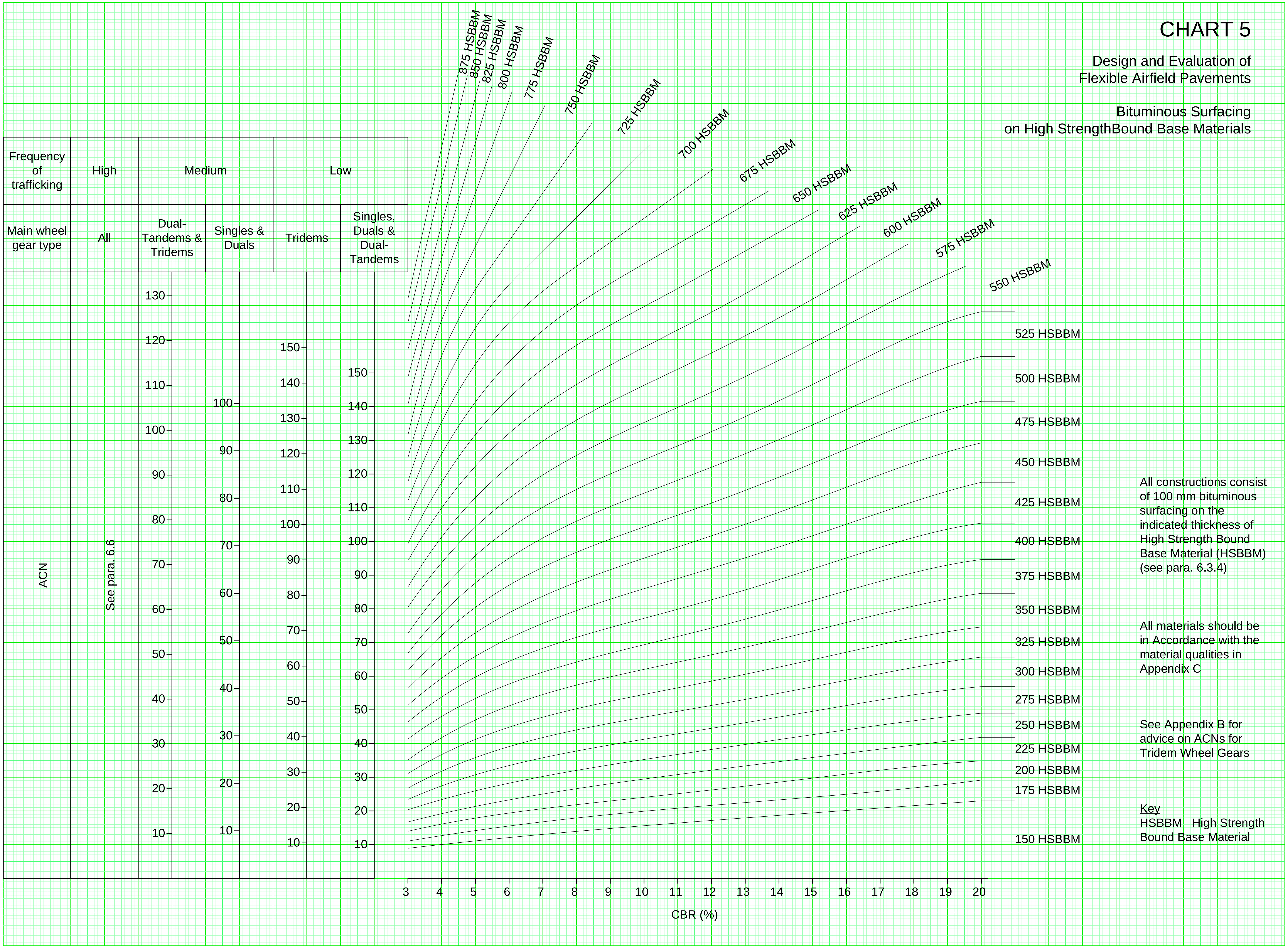


CHART 5
Design and Evaluation of
Flexible Airfield Pavements
Bituminous Surfacing
on High StrengthBound Base Materials
Frequency of trafficking
High
Medium
Low
Main wheel gear type
All
Dual-Tandems & Tridems
Singles & Duals
Tridems
Singles, Duals & Dual-Tandems
ACN
See para. 6.6
875 HSBBM
850 HSBBM
825 HSBBM
800 HSBBM
775 HSBBM
750 HSBBM
725 HSBBM
700 HSBBM
675 HSBBM
650 HSBBM
625 HSBBM
600 HSBBM
575 HSBBM
550 HSBBM
525 HSBBM
500 HSBBM
475 HSBBM
450 HSBBM
425 HSBBM
400 HSBBM
375 HSBBM
350 HSBBM
325 HSBBM
300 HSBBM
275 HSBBM
250 HSBBM
225 HSBBM
200 HSBBM
175 HSBBM
150 HSBBM
CBR (%)
All constructions consist of 100 mm bituminous surfacing on the indicated thickness of High Strength Bound Base Material (HSBBM) (see para. 6.3.4)
All materials should be in Accordance with the material qualities in Appendix C
See Appendix B for advice on ACNs for Tridem Wheel Gears
Key
HSBBM High Strength Bound Base Material

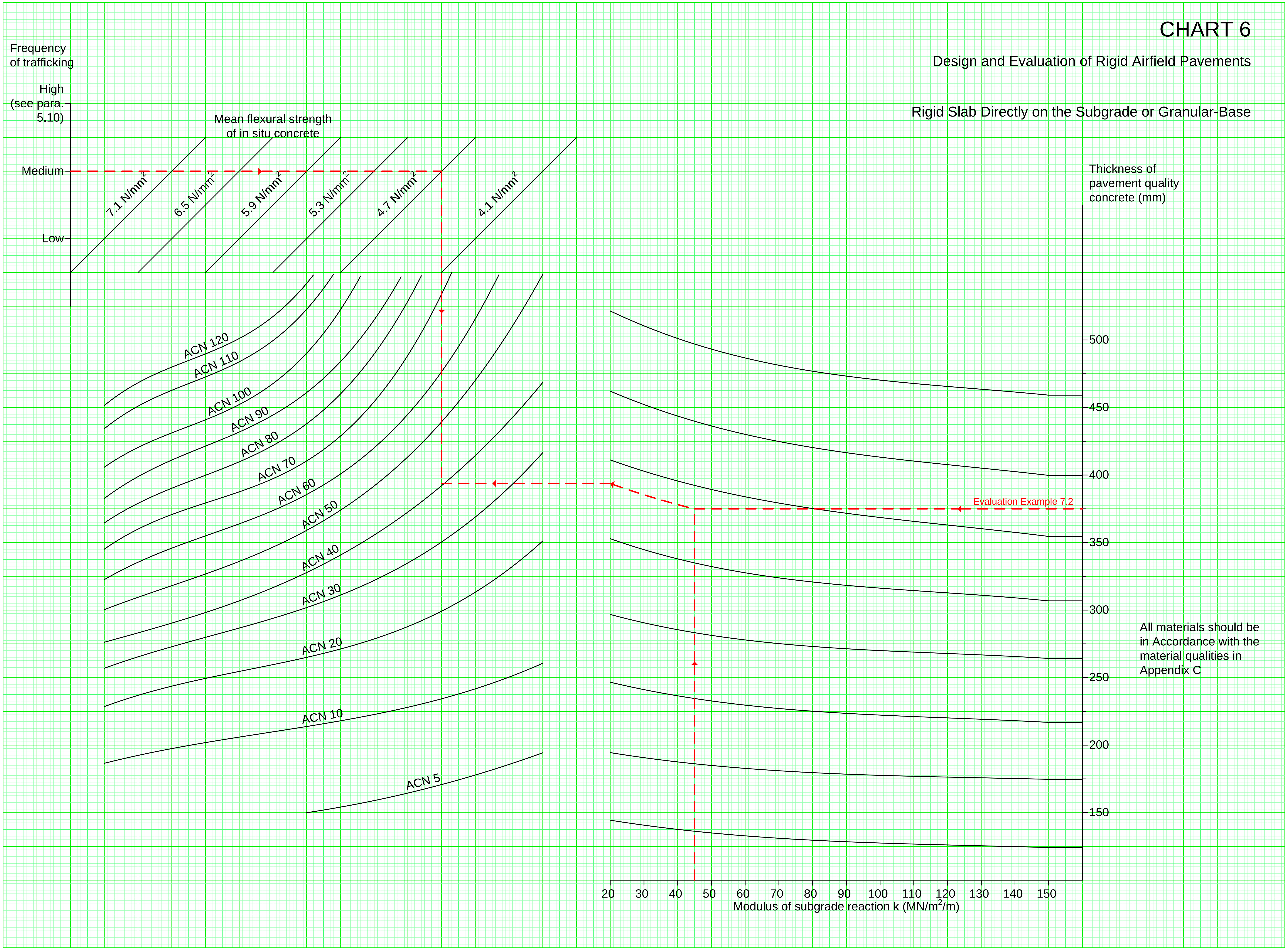


CHART 6
Design and Evaluation of Rigid Airfield Pavements
Rigid Slab Directly on the Subgrade or Granular-Base
Frequency of trafficking
High (see para. 5.10)
Medium
Low
Mean flexural strength of in situ concrete
7.1 N/mm2
6.5 N/mm2
5.9 N/mm2
5.3 N/mm2
4.7 N/mm2
4.1 N/mm2
ACN 120
ACN 110
ACN 100
ACN 90
ACN 80
ACN 70
ACN 60
ACN 50
ACN 40
ACN 30
ACN 20
ACN 10
ACN 5
Thickness of pavement quality concrete (mm)
500
450
400
350
300
250
200
150
Evaluation Example 7.2
20 30 40 50 60 70 80 90 100 110 120 130 140 150
Modulus of subgrade reaction k (MN/m2/m)
All materials should be in Accordance with the material qualities in Appendix C

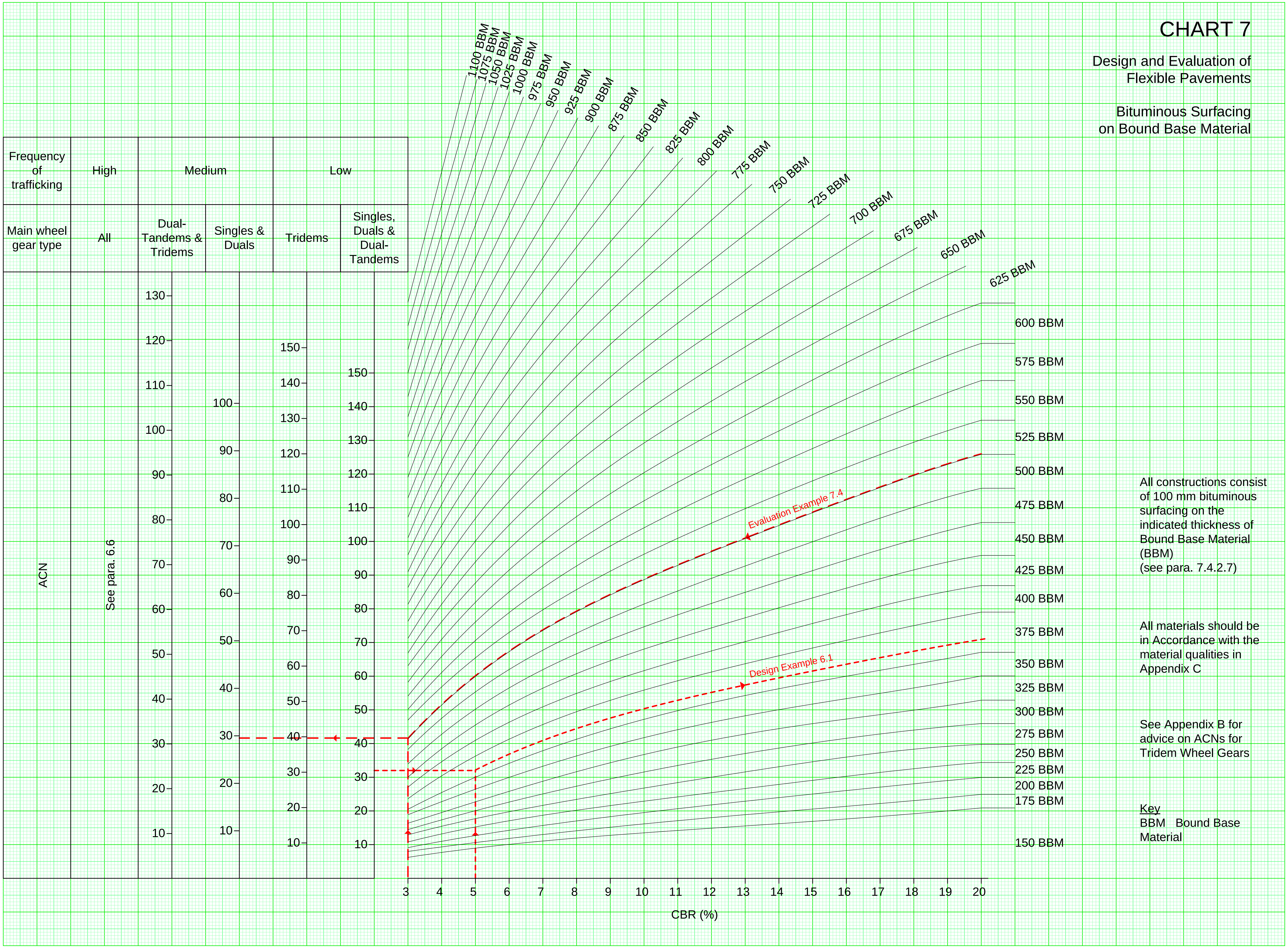
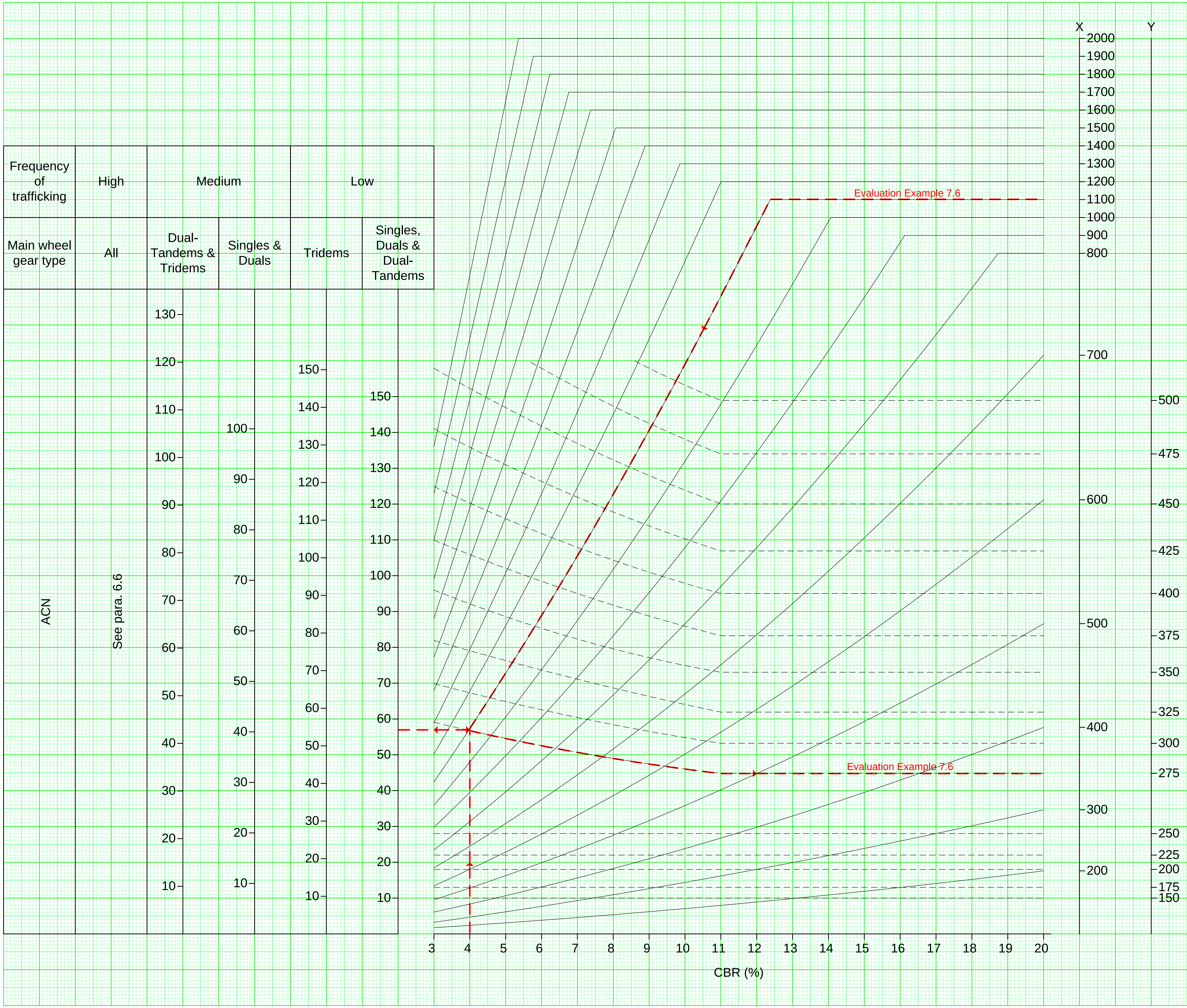


CHART 7
Design and Evaluation of Flexible Pavements
Bituminous Surfacing on Bound Base Material
Frequency of trafficking
High
Medium
Low
Main wheel gear type
All
Dual-Tandems & Tridems
Singles & Duals
Tridems
Singles, Duals & Dual-Tandems
ACN
See para. 6.6
CBR (%)
Evaluation Example 7.4
Design Example 6.1
1100 BBM
1075 BBM
1050 BBM
1025 BBM
1000 BBM
975 BBM
950 BBM
925 BBM
900 BBM
875 BBM
850 BBM
825 BBM
800 BBM
775 BBM
750 BBM
725 BBM
700 BBM
675 BBM
650 BBM
625 BBM
600 BBM
575 BBM
550 BBM
525 BBM
500 BBM
475 BBM
450 BBM
425 BBM
400 BBM
375 BBM
350 BBM
325 BBM
300 BBM
275 BBM
250 BBM
225 BBM
200 BBM
175 BBM
150 BBM
All constructions consist of 100 mm bituminous surfacing on the indicated thickness of Bound Base Material (BBM) (see para. 7.4.2.7)
All materials should be in Accordance with the material qualities in Appendix C
See Appendix B for advice on ACNs for Tridem Wheel Gears
Key
BBM Bound Base Material

CHART 8

Evaluation of Conventional Flexible Pavements



NOTE 1. X-AXIS
The X-Axis represents the total pavement thickness inncluding surfacing base and sub-base. The thickness of surfacing should be not less than 125 mm for aircraft with ACN > 50

NOTE 2. Y-AXIS
The Y-Axis represents the combined thickness of surfacing and granular base

All materials should be in Accordance with the material qualities in Appendix C

See Appendix B for advice on ACNs for Tridem Wheel Gears