PAVEMENT DESIGN FOR PORT AREAS

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PAVEMENT DESIGN FOR PORT AREAS

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ABSTRACT

This thesis describes a design method for heavily loaded flexible pavements in port areas. The design method is based on existing pavement design principles which have been extended to cover the specific design constraints of these structures.

Existing pavement structures are surveyed and analysed, and the general problems of structural durability summarised. From this survey the most suitable paving materials are isolated. Characterisation techniques for paving materials and the development of existing empirical and semi-empirical designs are then reviewed. Since segmental block surfacing is being used increasingly in U.K. ports, current research and design and construction recommendations for this type of surfacing material are assessed.

The pavement design method uses the multi-layer elastic analysis model. A concept of relative damage is proposed for the assessment of vehicle loading, based on the distribution of gross weights of the containers carried. A suitable design unit,

the Port Area Wheel Load or PAWL, is defined. The method is presented in the form of a fully interactive computer program and graphical design charts. In both cases care has been taken to preserve the engineer's freedom in specifying material types and thicknesses, and controlling the actual design process.

Finally, two design examples are presented and areas of further extensions and refinements to the overall design method indicated.

CHAPTER ONE

PORT DEVELOPMENT AND OPERATION

Synopsis

This Chapter briefly outlines the principles of handling containerised cargo and the layout and operation of a modern port. The handling vehicles now in common use are described and general requirements of terminal pavements isolated. Finally, the development of a suitable design method for these structures is outlined and the contents of remaining Chapters summarised.

1.1. Introduction.

The recent growth of freight containerisation in all ports around the world has brought with it a serious problem for the port engineer. Charged with providing extensive areas of pavement capable of carrying container traffic's exceptionally high wheel loadings, the engineer can call on very little design guidance. The research reported in this thesis descibes the formulation and application of a complete design method for this class of heavy duty pavement.

In order to understand the problems associated with the design of heavy duty industrial pavements it is obviously necessary to apply the principles of pavement design, but it is also necessary to have a basic understanding of port operation and cargo handling requirements. This Chapter briefly outlines port operation and isolates the general requirements of an ideal pavement structure. Pavement design is divided into two distinct parts: an understanding of characterisation of the materials used and assessment of vehicular loading, and the application of these to a design method. Each Chapter of this thesis covers a specific topic or section on the development of a suitable design philosophy.

1.2. Development of the Modern Port.

Due to the development of steel construction and steam power throughout the nineteenth century, there was a dramatic increase in the size of ships, promoting a similar increase in world trade. However, the basic method of cargo handling in a port didn't change; all goods were treated as a form of general cargo with very little, if any, long distance shipments of bulk cargo. The small individual items were stored on the quayside, in transit sheds, until they were required for loading, when all cargo was either manhandled or lifted by simple craneage in itemised form. This was obviously a very lengthy process, ships were tied up at the quayside for many days for the relatively straight-forward process of transferring imported goods onto the quayside and export goods into the hold.

Mechanisation of this process obviously helped to decrease ship turn round times; pallets, deck cranes, large hatch covers, fork lift trucks were developed and widely used, but the handling system remained unchanged. Since a ship is designed for travel at sea, all time spent at the quayside was both wasteful of resources and extremely costly, thus if transportation costs were to be reduced, turn round times had to shortened. Also, as ships became larger, direct shipment to small ports became increasingly uneconomic and a system of feeder ships was introduced. However, transportation costs became prohibitively expensive if cargo had to be transferred between ships on route, often resulting in a doubling of storage time as well as handling costs. There were two ways of reducing through transport costs of cargo :-

(i) cargo handled in standard bulk form to make for rapid loading and unloading.

(ii) use of larger ships which run full, frequently and with minimum turn-round times at each port

The first moves towards this ideal were made in America in the late 1950's. Sealand Service Inc. and Matson Navigation Co. developed a system whereby a complete lorry was taken on board in a single lift. The benefits of quick handling and limited storage was almost immediately appreciated and specialised containers were built by the different shipping companies. Within a few years containers were being used on most trans-Atlantic shipping routes; containerisation had begun.

Although the system spread rapidly over the next decade, it did not overcome all fundamental problems. The aim had been solely to minimise a ship's berthing time, containers were invariably loaded, or stuffed, in transit sheds just prior to loading. Also, each company had its own containers of specific size, hence if there was no return cargo 50 percent of a container's travelling time was spent empty.

To achieve maximum benefit from containerisation each container should be loaded as near its source as possible, then transported and delivered complete to its destination, thus minimising costs at the land/sea interface. The complete system was brought to a single standard by the adoption of the International Standards Organisation (I.S.O.) specification for 20, 30 and 40 ft. containers with marking codes for all freight containers. High throughput at ports has also been greatly increased by the parallel development of specialised handling equipment.

With the development of bigger and faster ships thoughout the 1960's all types of trade increased dramatically. There are many forms of bulk handling, e.g. oil, ore, grain, gases, liquids, and all have called for the development of specialised port facilities and purpose built ships. Freight containerisation can be defined simply as the bulk handling of general cargo.

Perhaps the most striking visual aspect of a modern port is the abs~~c~~ence (JK) of ships. The specialised loading facilities now used to keep ships at sea for as long as possible makes multi-use of berths impracticable. Annual tonnage through a roll-on roll-off container berth is approximately ten times greater than that of a traditional break-bulk berth. It is also evident that large areas of storage space are required to build up a complete cargo prior to loading, however since containers are weathertight covered storage is not necessary. World investment in containerised handling has been very high and is expanding, there is a large potential for development of ports around the world which will be exploited over the next two decades.

1.3. Port Layout and Operation.

There has been a dramatic change in the use and layout of a port; the finger quay system with large areas of covered storage has been replaced by single berths, large areas of open parking and heavy investment in specialised machinery. Many modes of container handling are used, each particularly suited to a specific environment and degree of flexibility, but all require a heavy duty pavement to ensure economic operation.

With the current cost of transporting a container through a port at approximately £30, and large capital investment in handling machinery, a port cannot afford to have the pavement out of service for long periods for repair and replacement. The requirements of a pavement are directly related to the type of vehicle operating on the surface, hence it is necessary to consider the operation and, particularly, the range of equipment now in common use. The following discussion is not intended to be a technical appraisal of the different forms of handling, this is a long way beyond the scope of this research, and although the relative effects of different vehicles on the pavements is presented, the choice of a particular system will invariably depend on factors not evenly remotely connected with pavement design.

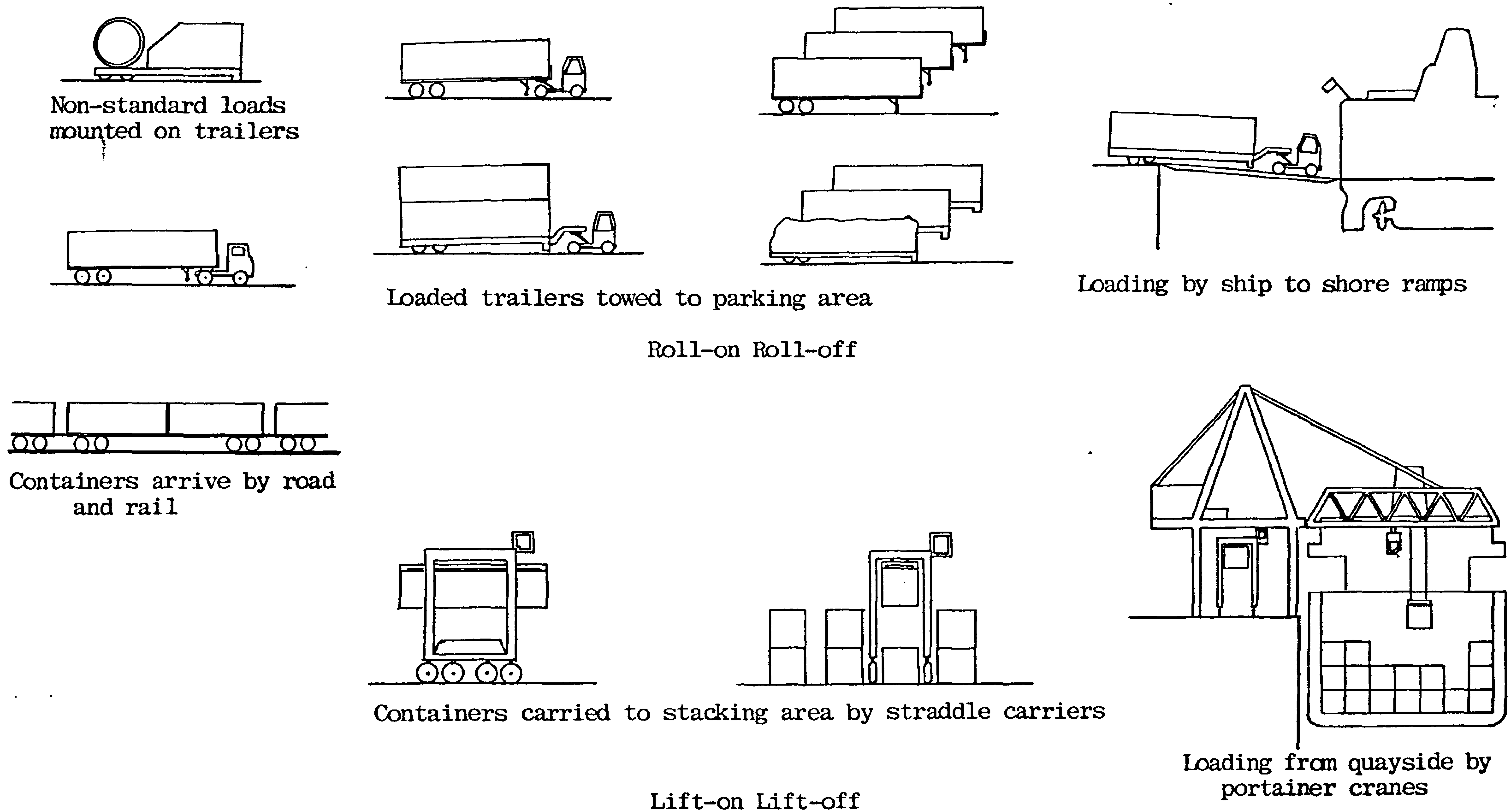
The means of transporting a cargo to and from a port has also changed; old ports had extensive rail systems running right up to the quayside, now the majority of cargo is carried by road, rail transport has not been eliminated completely, but it is usually restricted to an area away from the quayside. Again short turn round times are important and road vehicles invariably collect as well as deliver to the port on each trip.

There are two distinct forms of container handling, as shown in Figure 1.1, :-

(a) container handling for longer sea crossings - broadly described as lift-on lift-off (Lo-Lo).

(b) trailer handling for shorter sea crossings - Roll-on Roll-off (Ro-Ro).

Figure 1.1 Ro-Ro and Lo-Lo handling methods



With Lo-Lo the container units are handled separately, being stored or stacked until required for loading. There are several systems available to transport the units from the arrival bays, to the stacks and then to the quayside, e.g. trailers or straddle carriers. Special track mounted cranes, portainers, are usually provided on the quay which are specifically designed to give fast loading of container units into the hold.

For shorter sea crossings a much faster turn round time is required and Ro-Ro ships, with ramps, are used which enable loaded vehicles and trailers to be driven onto and off the ship. The loading ramps may be mounted on the ship, on the quayside or a floating pontoon, and the use of two ramps, usually at the bow and stern, obviates the need to turn vehicles within the ship and hence minimises loading times. With this system road trailers are not unloaded, the prime mover is detached upon arrival and specialised terminal tractors carry the trailer through the port. Containers and other non-unitised cargo can also be handled by the use of terminal trailer systems. These usually consist of low flat trailers, 20 ft or 40 ft long, towed by terminal tractors by removable "gooseneck" towing connections.

With trailer systems, stowage volume per tonne of cargo is increased, particularly where highway trailers are used, however total journey time is considerably reduced. Many operators find it more economical to use the shortest Ro-Ro crossing with a long road haulage than the longer Lo-Lo shipping routes. There is an increasing trend towards the carrying of both Lo-Lo and Ro-Ro cargo on medium and long sea crossings, containers are stacked on the upper deck whilst ramps give access to the covered holds.

Both handling systems require large areas of open space, a requirement many of the traditional ports could not satisfy even if old finger quays were filled in. There is also an economic necessity of having access to berths at all states of the tide. This has led to the construction of new facilities at deep water sites much nearer the open sea, requiring extensive land reclamation as well as the construction of quays and equipment. Many such developments of major ports all over the world have been described in detail elsewhere.

The Port of Ipswich is typical of many that have developed with the expansion of Ro-Ro and containerised traffic. An area on the south bank of the River Orwell was reclaimed in 1972 and is now the site of a single Ro-Ro berth with container backup. Since the main dock facilities are on the north bank of the river, this quay is self-contained and provides an excellent example of operation and layout, see Figure 1.2.

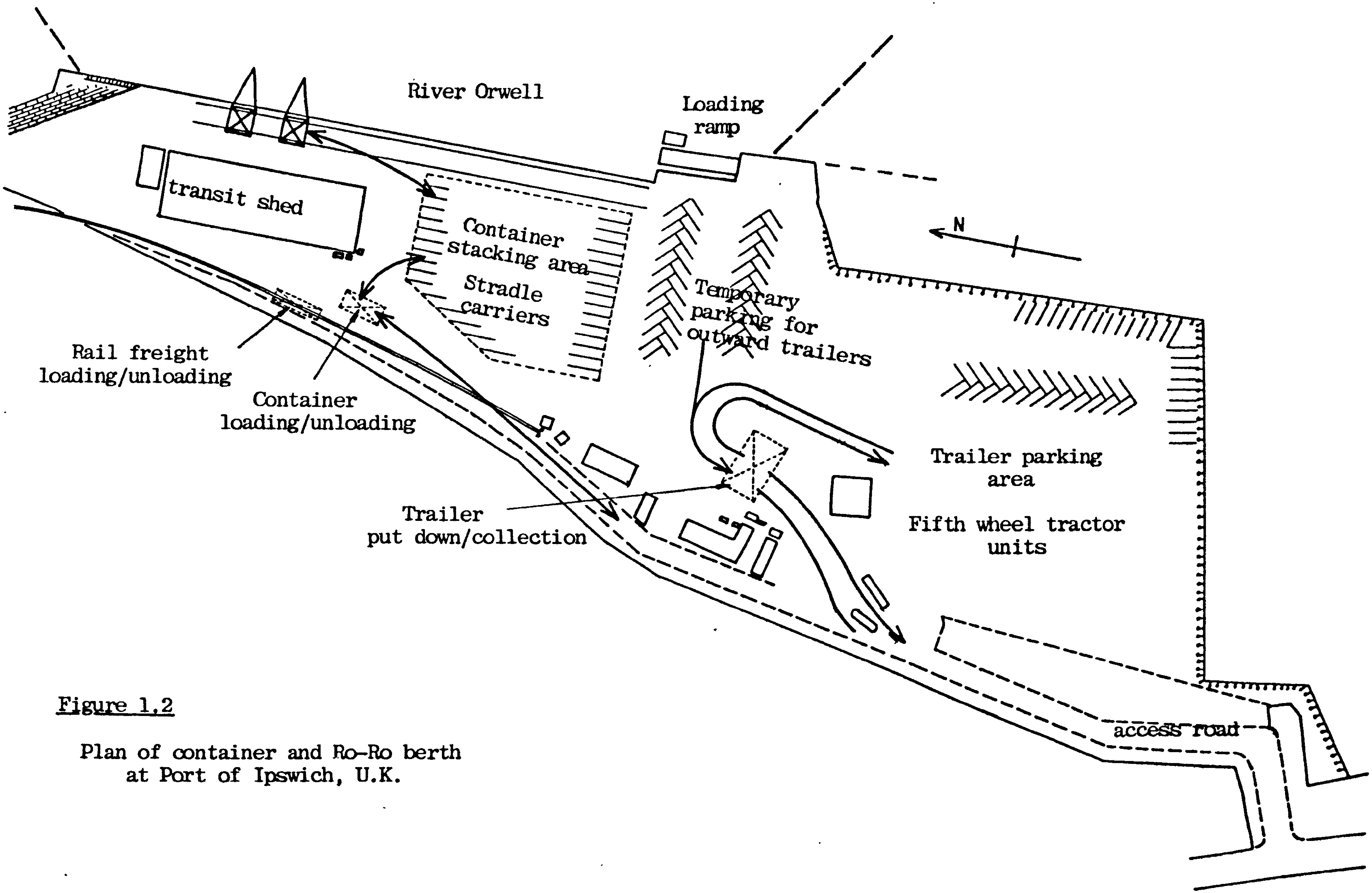


Figure 1.2

Plan of container and Ro-Ro berth
at Port of Ipswich, U.K.

1.4. Cargo Storage.

The amount of time the cargo spends within a port should be kept to a minimum. Long dwell times not only take up valuable storage space, but can become a major hindrance if they have to be continually moved and sorted. The cargo is divided into two groups, imports and exports. Imports require a relatively free access, or high selectivity, since the order of collection is not known, whereas the handling of exports can be planned more easily in advance. One obvious method of increasing capacity is to stack higher, but this reduces accessibilty and increases retrieval times. For this reason, most ports using straddle carriers stack export containers two high and imports "one and a half" high. Late, heavy or non-standard loads cause several problems, often calling for a specific handling system of their own.

For Ro-Ro traffic these problems are not so great as Lo-Lo since dwell times are shorter and, by definition, the operation allows much greater flexibility to accommodate the problem load.

1.5. The Handling Equipment.

The choice of a particular handling and operating system is depend~~a~~ent on many factors, not least the preference of a port to an existing and proven system. However, the differences between the various types of handling equipment can be covered by the following categories :-

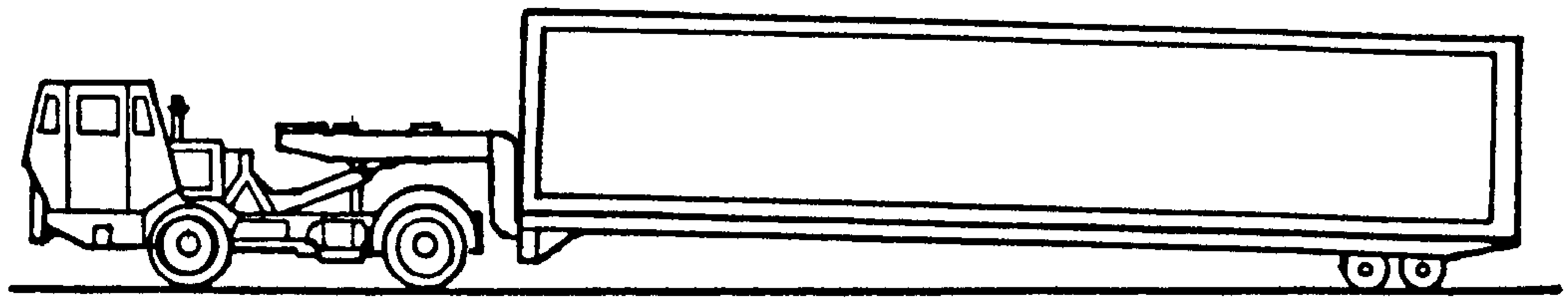
1. tractor and trailer systems
2. straddle carriers
3. sidelifts or sideloaders.
4. front lift trucks
5. rubber tyred gantry cranes
6. rail mounted gantry cranes

All handling vehicles, except gantry cranes, are illustrated in Figure 1.3.

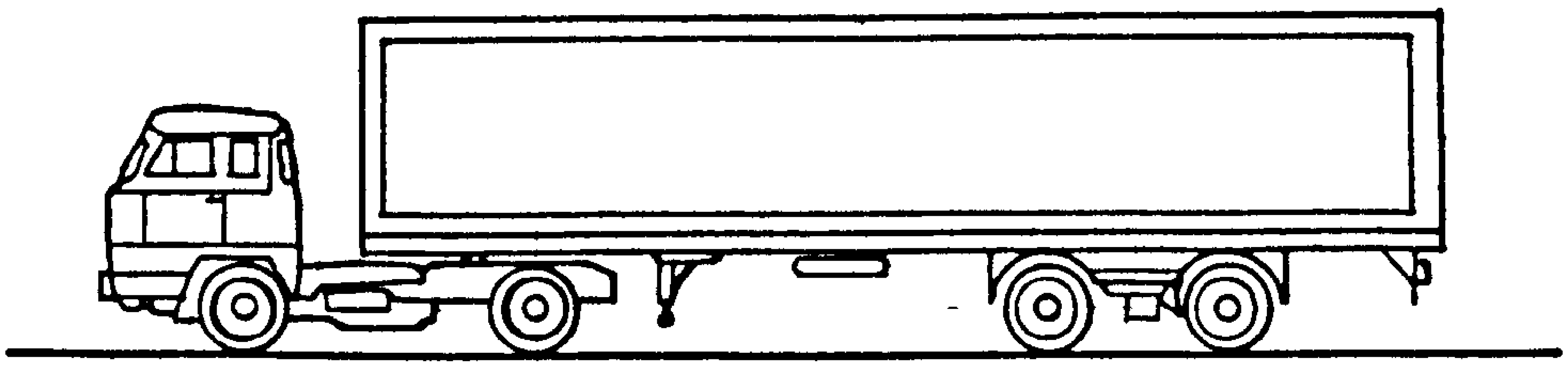
1.5.1. Tractor and trailer systems.

Trailer systems are widely used on all Ro-Ro berths giving very fast loading and unloading rate over ship to shore ramps. Highway trailers are taken on most short sea crossings, these being towed by terminal tractors with elevating fifth wheels. Highway trailers usually have two rear axles, fitted with pneumatic twin tyres, and steel leg supports at the front. Terminal trailers are used to carry containers or non-standard

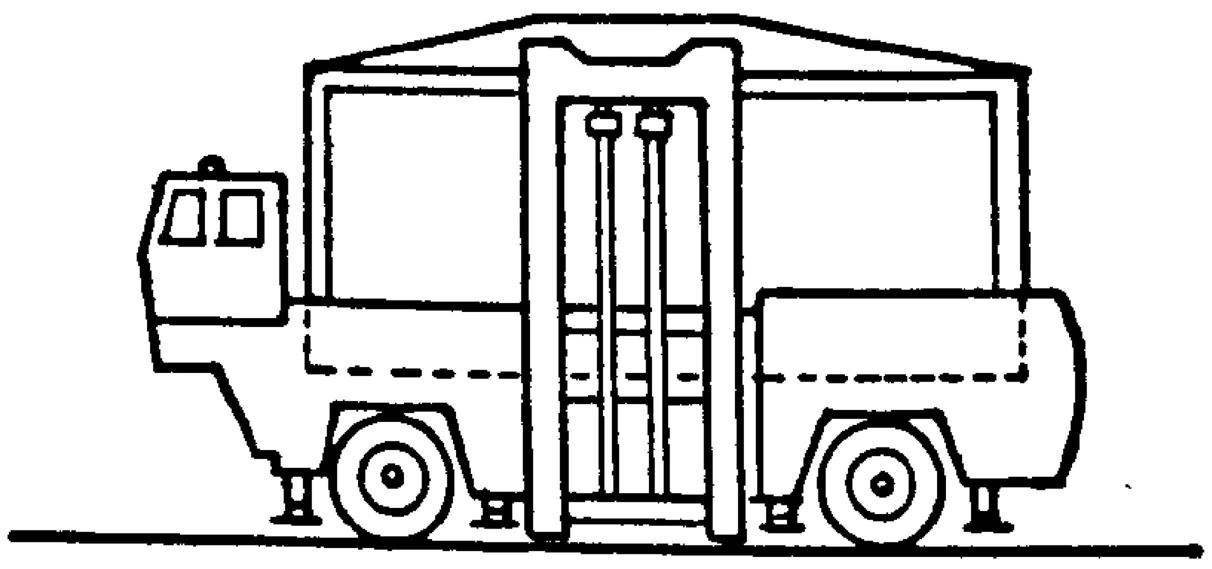
Figure 1.3 Container handling vehicles



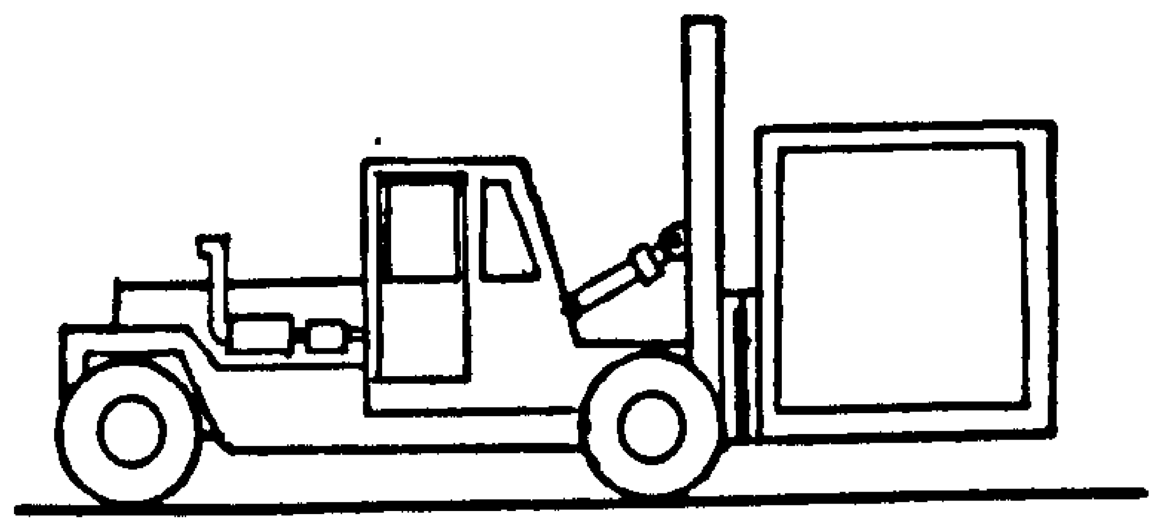
(a) terminal tractor/trailer



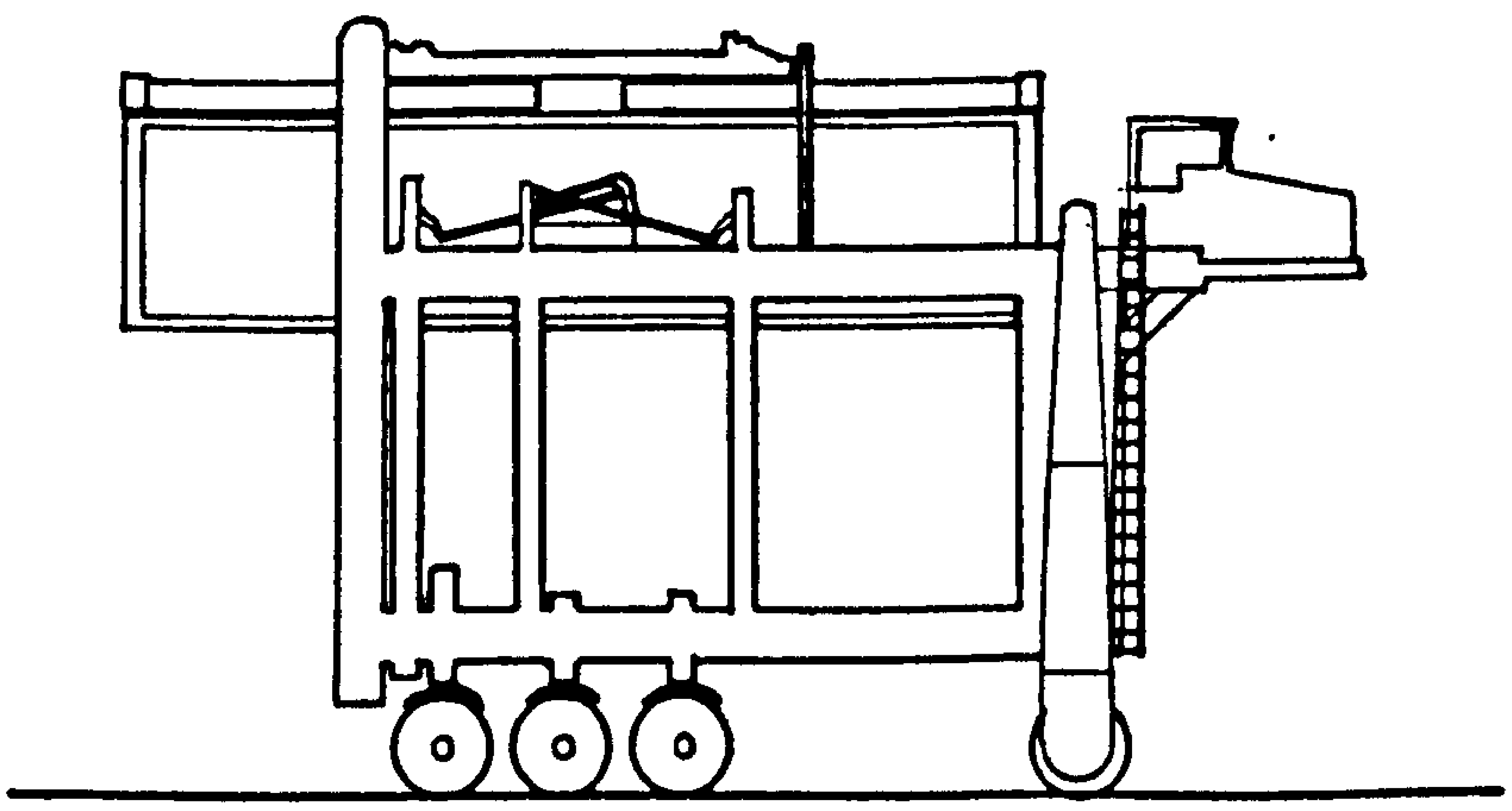
(b) highway trailer



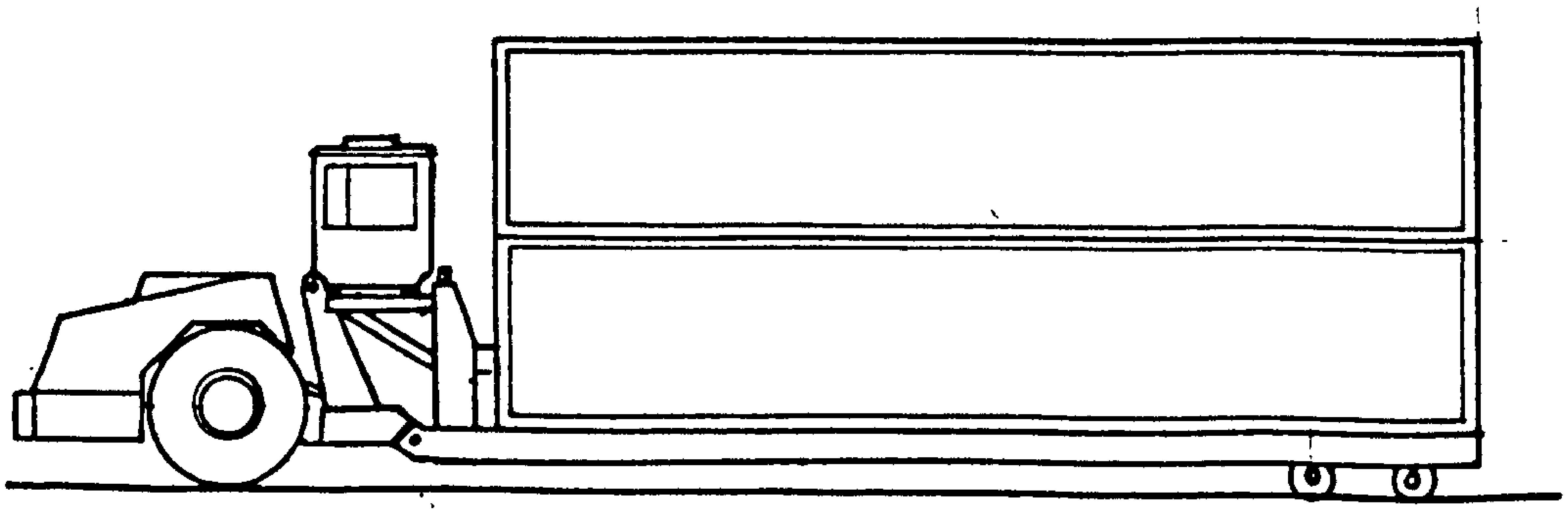
(c) side loader



(d) front lift truck



(e) straddle carrier



(f) lift unit frame (LUF) system

loads on Ro-Ro operations. These trailers have a very low profile, either 20 ft or 40 ft long, 8 ft wide, with a single or double axle having solid rubber, small diameter wheels at the rear. There is no towing attachment, the unit rests on a steel bar and is picked up by the tractor unit with a "gooseneck" connection.

This system has a relatively low capital cost, high availability, low maintenance and virtually infinite flexibility. Apart from the obvious Ro-Ro application, trailers are commonly used to haul containers between the quayside and the gantry operated stacks, particularly if the intermediate distances are large.

1.5.2. Straddle carriers.

Straddle carriers have been widely used in large container berths over the past 20 years. The early machines were relatively unreliable, with high maintenance costs, but these have improved with the development of better hydraulic and electrical systems.

A fleet of straddle carriers can operate a complete container park, carrying units between the delivery point, the stacks and the quayside loading crane. Most carriers allow a "one over two" operation, a maximum stacking height of two to allow for access. The containers are stacked in rows, with an aisle width of approximately 1.75 metres, the length of each row is restricted to about ten 40 ft units to keep travelling time down the aisles to an acceptable level. These machines are not ideally suited to either long travelling distances or, if the site is small and a high throughput required, gantry cranes can be a more economic choice. However, the straddle system is reasonably flexible and favoured at many ports.

High capital costs, maintenance and low availability have been the principal problems with straddle carriers. Oil spillage from damaged hydraulics is a very common complaint, as much as 50 litres can be spilt in a single fracture and since hydraulic oil is a potent solvent of bitumen, asphaltic surfacings have been severely affected. Availability of straddle carriers is estimated to be between 60 and 65 %, of which 90 % is attributable to impact damage (particularly exposed ladders and hydraulics) and punctures. Most ports also operate a pavement sweeper to reduce the frequency of punctures.

1.5.3. Sidelifts or sideloaders.

A more recent alternative to straddle carriers, sideloaders offer several operational advantages for confined sites. The vehicle operates from a single aisle, allowing containers to be stacked in double rows. Since the boxes are lifted out of the stack, they can be stacked up to three high without severely restricting selectivity of other boxes, compared to straddle carriers which are restricted to a maximum of two high. Sideloaders give increased visibility, are easier to drive and the load is carried evenly on the chassis, not slung from an hydraulic system. The containers are lifted from a point outside the vehicle's trackwidth, hence hydraulic stabilising jacks have to be lowered for loading and unloading. This increases the handling time for each unit, but with a lower centre of gravity, they can operate on more uneven surfaces and steeper gradients.

1.5.4. Front lift trucks.

The front lift truck is the traditional form of cargo handling and a large port will have a number of these vehicles of varying capacities. However, they are not generally regarded as suitable in large container terminals because of restricted access to stacked containers; the units have to be either block stacked (very low selectivity) or stacked in double rows with wide aisles (very low density). Not all containers are fitted with pockets for the forks and suitable toplift beams have to be fitted. Despite the poor visibility and high axle loads, front lift trucks are well suited to some Ro-Ro applications and low silhouette masts are available for between deck operation. If containers are carried, the width of the roadway and loading ramp must be greater than the length of the unit, 20 or 40 ft, but full use of deck volume can be made by stacking since no overhead clearance is necessary.

1.5.5. Rubber tyred and rail mounted gantries.

Collectively known as transtainers, the gantry span, or effective span if two cantilevered arms are added, ranges from 8 m up to approximately 100 m. The system offers an increased density of container stacking compared to straddle carrier operation and is ideally suited to an automated location / positioning system. The Port of Felixstowe has recently installed a series of transtainers to replace its straddle carrier fleet, each gantry spans six rows of boxes plus an access roadway and is capable of stacking five high. It has already been noted that as stack height increases selectivity decreases, however here the maximum is five high rather than three high as in the case of straddle carriers.

The rubber tyred gantries usually run along specially constructed carrying beams with automatic steering. Gantry operation is not as flexible as straddle carrier operation, but higher throughput can be achieved easily with proper co-ordination. Transtainers do not normally deliver containers directly to the quayside, the units are usually ferried by trailers operating a "merry go round" system. Gantry cranes cannot operate on large gradients and excessive settlement could be a problem.

1.5.6. Other types of handling equipment.

Other types of container handling have been developed, but they are relatively inflexible, suited to a particular environment, and not widely used.

One system not descibed above is the Lift Unit Frame, or LUF, system. Best suited to Ro-Ro operation, the 20 ft or 40 ft trailer carries up to four containers at a time; the tractor unit is detachable to keep stowage volume to a minimum. Apart from the large size when laden, gross weight and axle loads are very high and not all ship to shore ramps are capable of carrying such loads.

1.6. Pavement Requirments.

The general requirement of large areas of open paving suitable for container stacking and vehicle operation has already been outlined. Modern ports are constructed near the open sea, usually on reclaimed land with little or no time available for trial constructions or natural settlement of fill material. All trafficked areas must be surfaced in some form to keep vehicle maintenance to a reasonable level and to maintain all year operation. The handling equipment does not satisfy the normal highway axle weight restrictions and, in addition to the high magnitude, the wheels are often channelled into narrow lanes. There are also high, concentrated static surface loads from stored cargo. These surface contact stresses can be much higher than those under pneumatic tyres, even though the total loads are smaller.

The requirement is, therefore, for an easily and rapidly laid pavement capable of withstanding very heavy and channelled wheel traffic as well as high static contact stresses. It should also be able to accommodate substantial settlement without distress, withstand surface pollutants and accept any possible future changes in use without the need for major reconstruction. Further, with such large capital investments in port facilities and equipment a high cargo throughput is essential, thus a pavement which has to be taken out of service for repair is critical since large financial losses accumulate very quickly. The pavement structure, therefore, must also be reliable and maintenance free.

Many different forms of pavement construction have been developed and tried by various port authorities. There has been little comparison between the various types and, almost without exception, no fundamental design procedures have been developed or applied. No one form of paving has proved to satisfy all the requirements, which partly explains the large diversity of contruction techniques. Each port has evolved, or tried to evolve, a form of construction that satisfies its own requirements, pricipally based on previous experience. Many authorities have come to expect high annual maintenance costs, sometimes starting within a few months of completion. This has led to a reluctance to try more costly forms of construction or to increase pavement strength where there is no clear evidence that maintenance will be reduced.

There is an obvious need, therefore, for a rationilisation of design procedures for these types of pavement. The aim of this research project is to present a critical appraisal of the various types of pavement construction used in port areas and to propose a suitable basis for a design philosophy.

1.7. Outline of the Remaining Chapters.

The development of a design method for heavily loaded pavements in container handling areas has been divided into seven sections, each forming a separate Chapter. The first Chapter presents a study of existing port pavement structures, this includes an appraisal of the problems encountered and compares the success and suitability of various construction techniques that have been applied. Most forms of construction are described, including proprietory surfacing materials, from this study the most suitable paving materials are isolated.

The characterisation of paving materials commonly used in terminal pavements is presented and the development of empirical and semi-empirical design methods briefly summarised. Each of these occupy a separate Chapter, the information is drawn from the large amount of research into pavement structures carried out over the last twenty years. The information presented is far from exhaustive, however it covers all topics referred to later, and full references are included.

The use of concrete blocks as a form of surfacing material is a relatively recent innovation to the U.K. and they are widely used for surfacing residential roads and pedestrian areas, as well as heavy duty industrial paving. Since the use of blocks for heavily loaded pavements is increasing rapidly and they are claimed to be the nearest solution to the "ideal material", Chapter 5 summarises the current research on the characterisation of this form of surfacing.

The final three chapters describe the development of a design method, each chapter covering one section of the complete method. Chapter 6 presents a comprehensive assessment of pavement loading, both static and dynamic, and shows how a range of wheel loads are assessed in a suitable format for the design analysis. Chapter 7 describes the development of a design program based on the multi-layer elastic model. The fundamental data used for this analysis is taken from the summarised details in Chapters 3 and 4. The analysis is compiled in the form of a fully interactive computer program allowing a pavement structure to be specified, checked and modified to give a final design. The design is also presented in chart form, using the same analysis program, thus making it possible to use the design method without the need of a computer. The final chapter, Chapter 8, summarises the design method, giving examples on its use, and analyses several existing structures to compare results obtained from the theoretical analysis to those on site.

CHAPTER TWO

BEHAVIOUR OF EXISTING PORT PAVEMENTS

Synopsis

This Chapter presents a study of existing port pavement structures, including an appraisal of the problems encountered, and compares the success and suitability of various construction techniques that have been tried. Most forms of construction are described, and from this study the most suitable paving materials are isolated.

2.1. Introduction.

Since no universal pavement recommendations or design methods have been proposed over the years, different ports have developed their own means of solving their particular paving problems. Neither existing highway design nor airfield design methods were directly applicable and, with short construction times available, long term research or exploratory design was not plausible. This has led to a very wide variety of construction materials and techniques, it is almost true to say that no two pavement structures are alike. This makes any orderly presentation on pavement performance virtually impossible, particularly since a port may have several totally different constructions laid within the space of a few years. This Chapter is arranged under headings of type of paving material for both the wearing course and the structural base course.

In order to make comparisons on pavement damage meaningful, details of subgrade, material availability, vehicle loading and port operation are included where possible for the particular ports. Information on pavement design and cost of construction have been established in some cases and these are given where appropriate, but it should be remembered that the effects of inflation and geographical location make detailed comparisons difficult.

Papers and articles on pavement performance in port areas did not appear before about 1975. Since then two large scale surveys have been published, Refs. 2.1 and 2.2, and several general articles or descriptions of specific forms of construction have also appeared, Refs. 2.3 to 2.10. The information presented in this Chapter is assembled from these sources as well as from a series of site visits undertaken by the author in the summer of 1978.

2.2. Types of Pavement Construction.

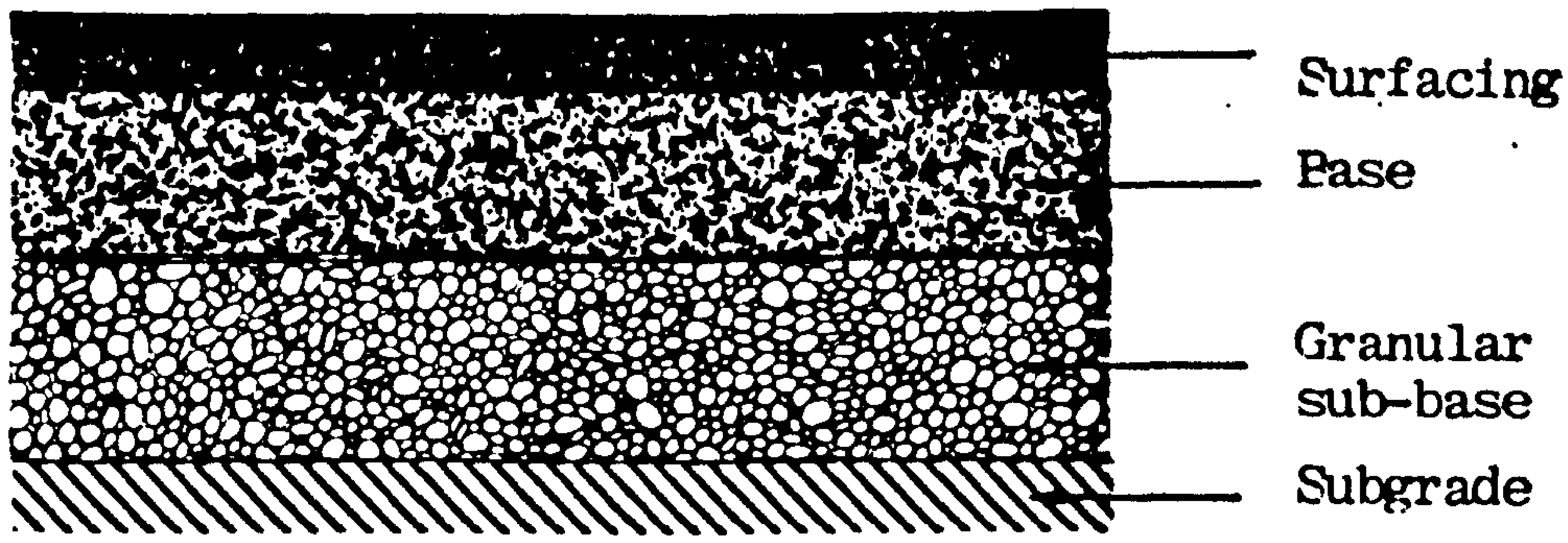
Despite the diversity of paving materials and construction techniques, most pavement structures can be divided into one of seven broad categories shown in Figure 2.1, (a) to (g). There are two principal elements in a pavement :-

SURFACING - must be of a high durability and strength to resist severe surface loads and contact stresses.

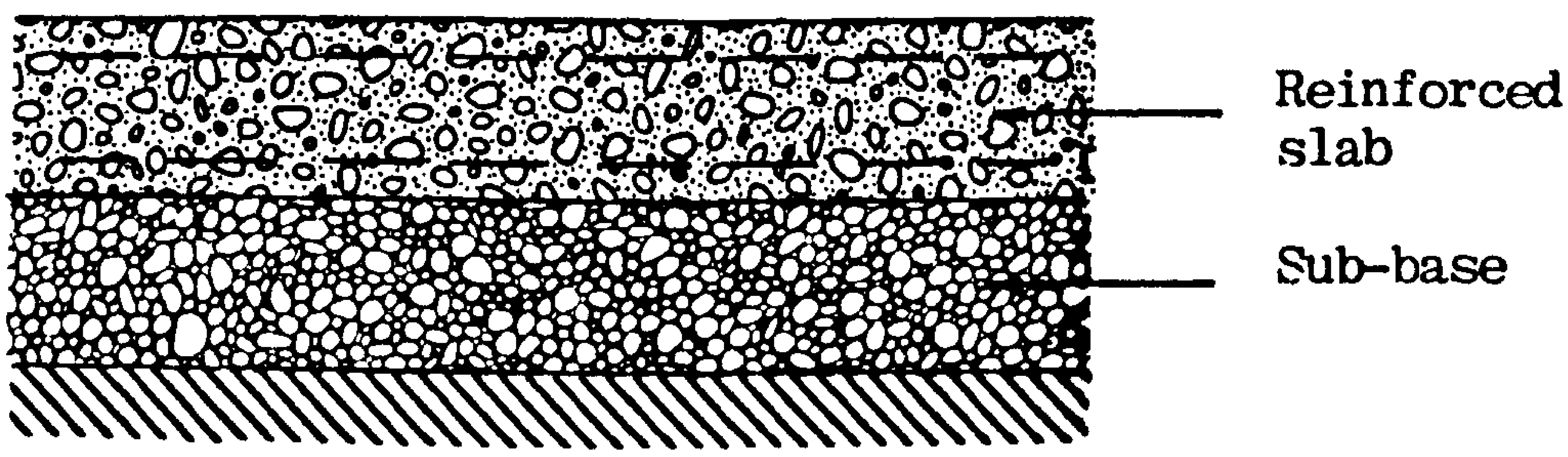
BASE - the structural component of the pavement which spreads the load so the subgrade is not overstressed.

Figure 2.1 Types of pavement construction

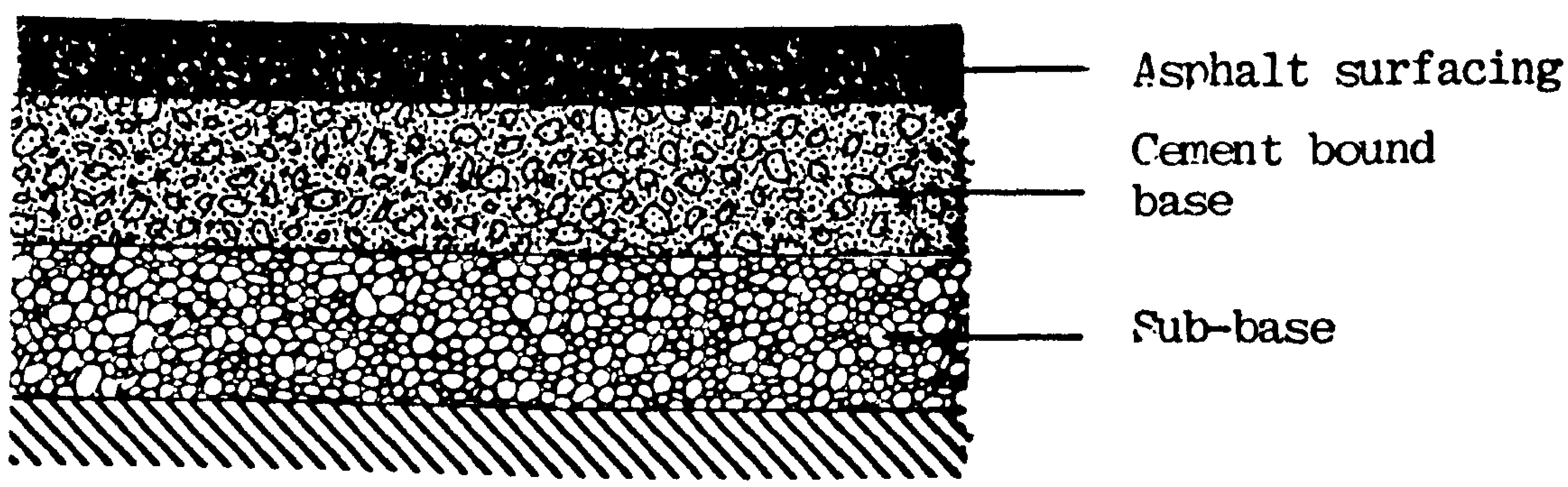
(a) All asphalt construction



(b) Rigid construction



(c) Composite construction



(d) Precast raft construction

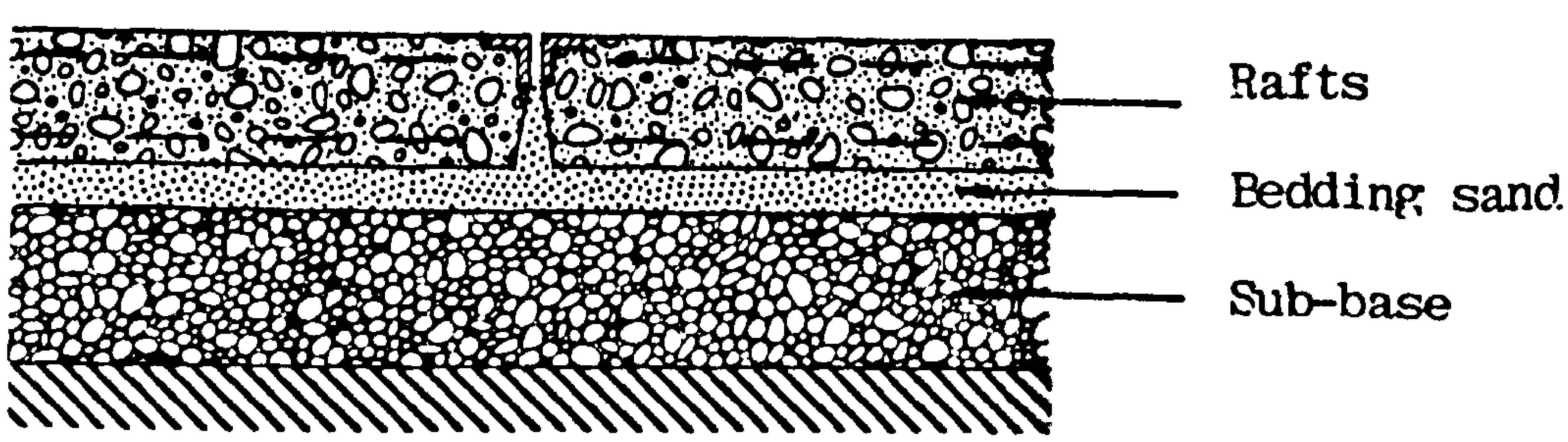
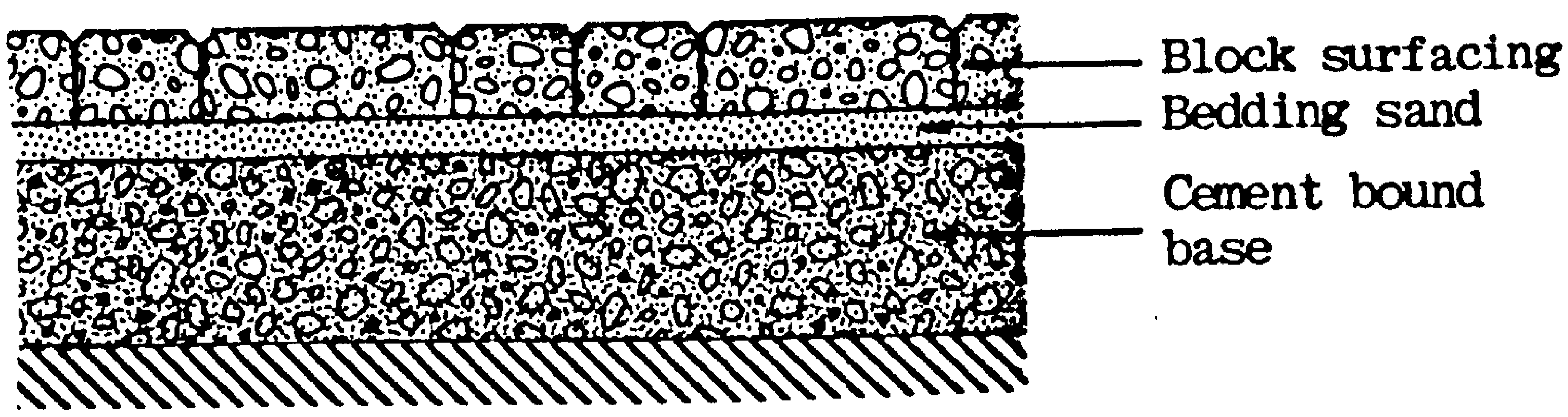
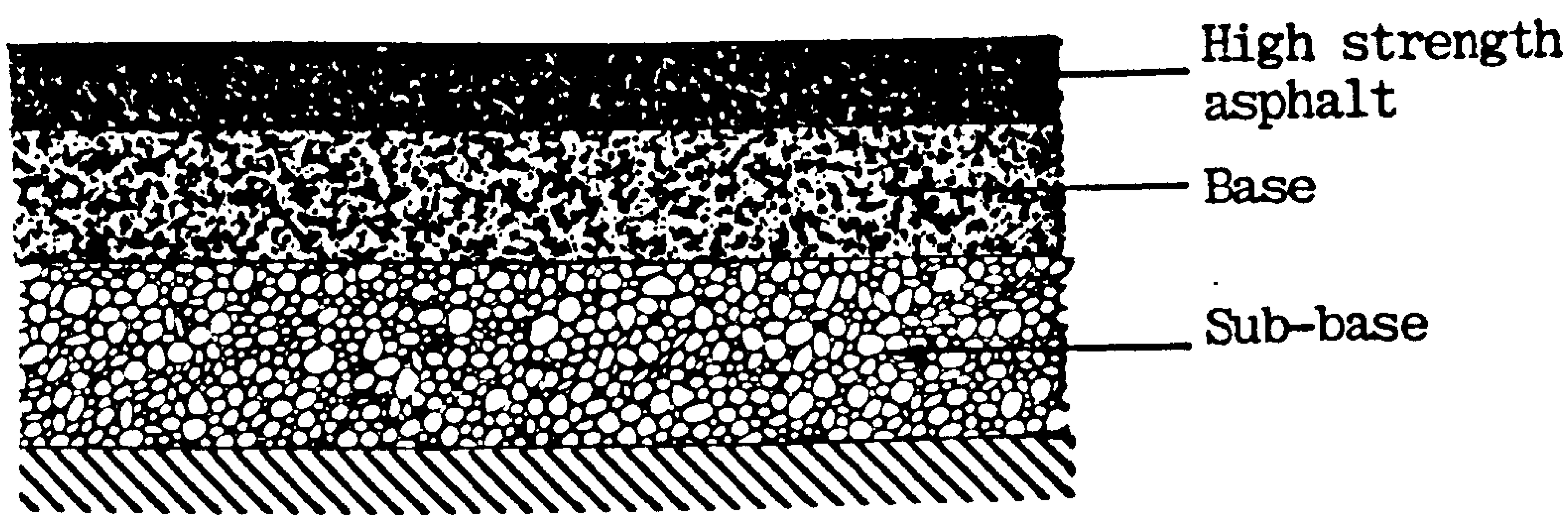


Figure 2.1 (contd) Types of pavement construction

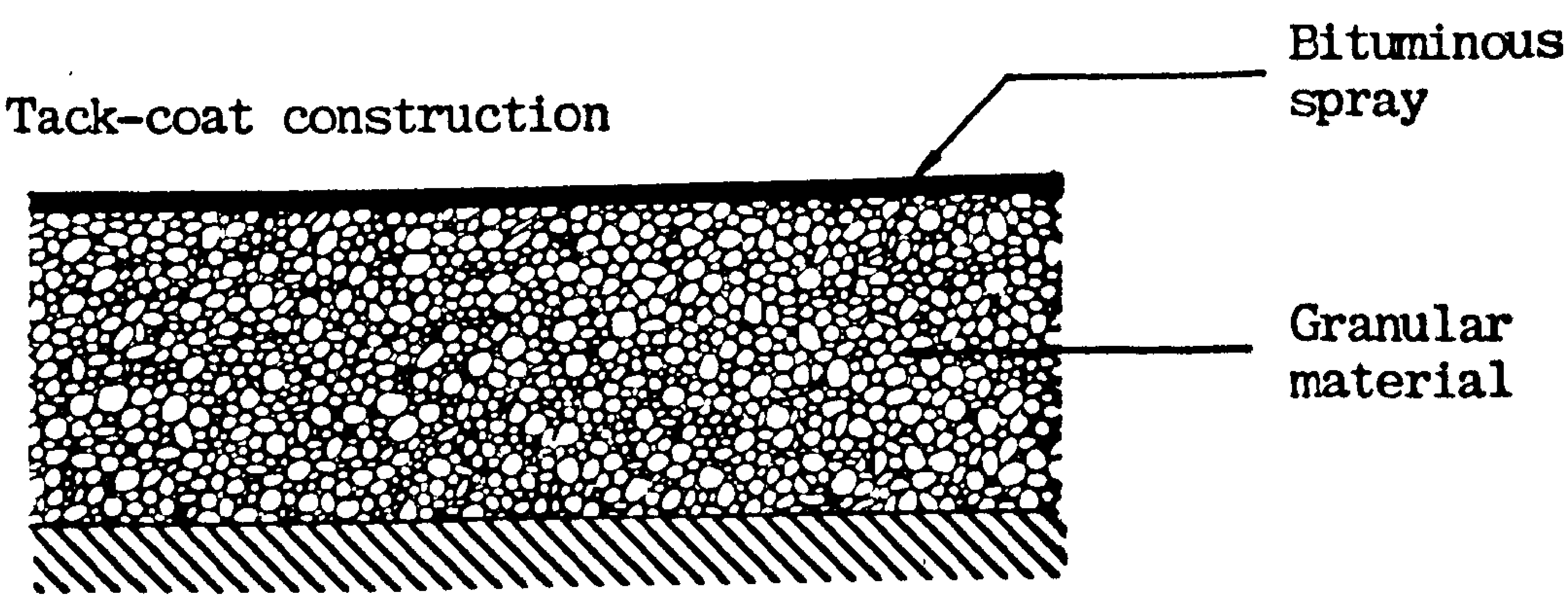
(e) Block construction



(f) Special asphalt surfacings



(g) Tack-coat construction



A secondary element is the SUB-BASE which is provided where the pavement is constructed over a weak subgrade to give extra strength at low cost, it also provides a good working surface for subsequent construction.

Ideally, each component is selected for its suitability to particular conditions and the combinations shown in Figure 2.1 are not intended to be comprehensive. The performance of the various materials used in industrial pavements are now discussed in detail.

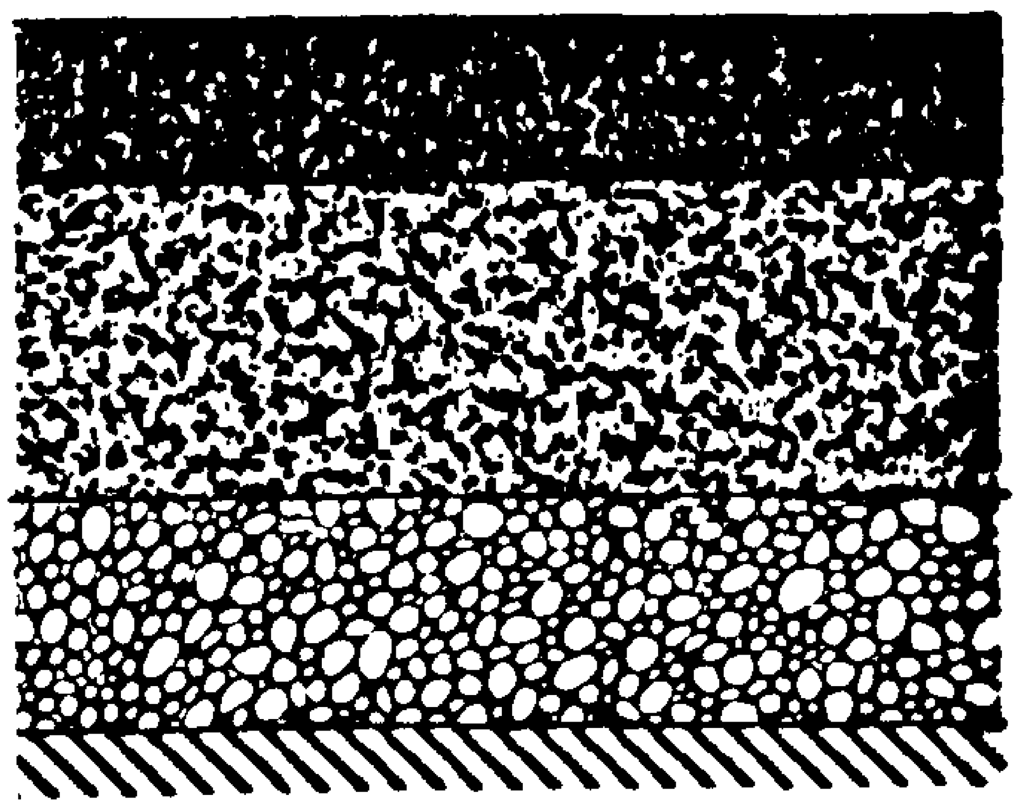
2.3. Full Depth Asphaltic Construction.

Very few ports have adopted a full-depth bituminous bound construction, three that have laid substantial areas are the Ports of Liverpool (Seaforth), Ipswich, both in the U.K., and Aarhus, Denmark.

The Port of Liverpool has four different types of construction in various parts of the handling area. These are shown in diagrammatic form in Figure 2.2, (a) to (d); in all cases the subgrade is a tipped fill overlying sand and stiff brown clay. The surfacings at this port show many of the

Figure 2.2 Pavement constructions at Port of Liverpool

(a)



100 mm combined asphalt wearing course granite aggregate (stone content 55%) and dense bitumen macadam base course

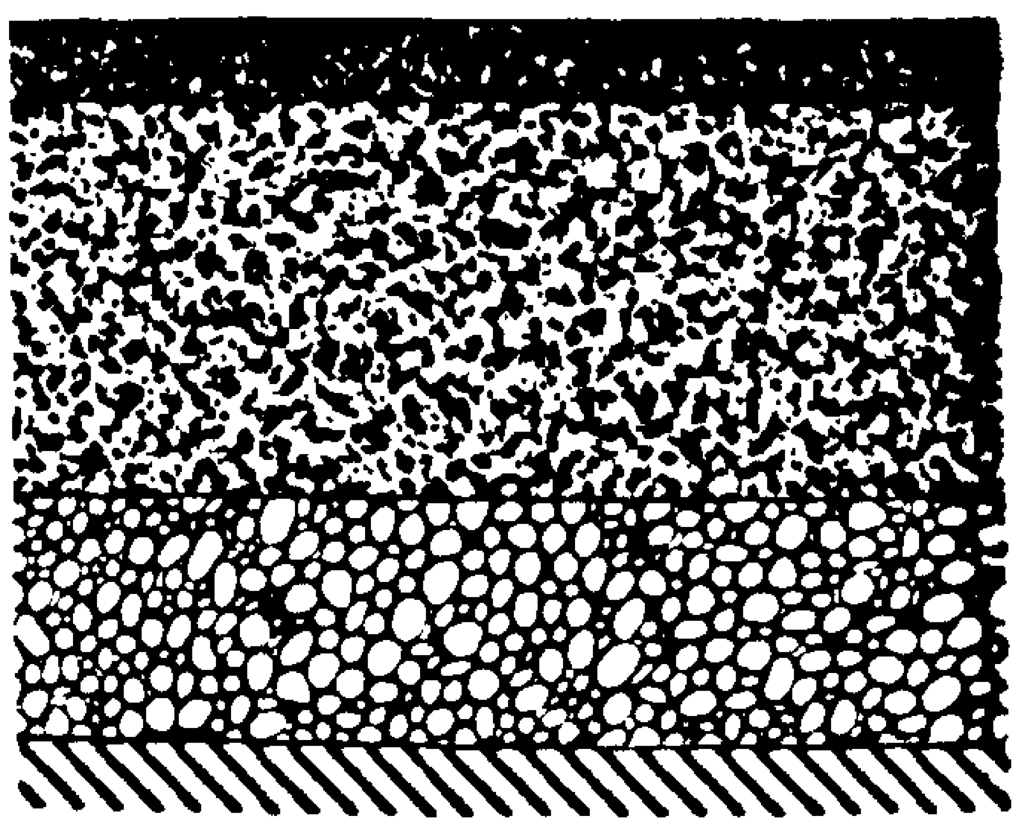
200 mm wet mix macadam base

150 mm type 2 granular sub-base

Tipped fill subgrade overlying sand and stiff brown clay

Container stacking area

(b)



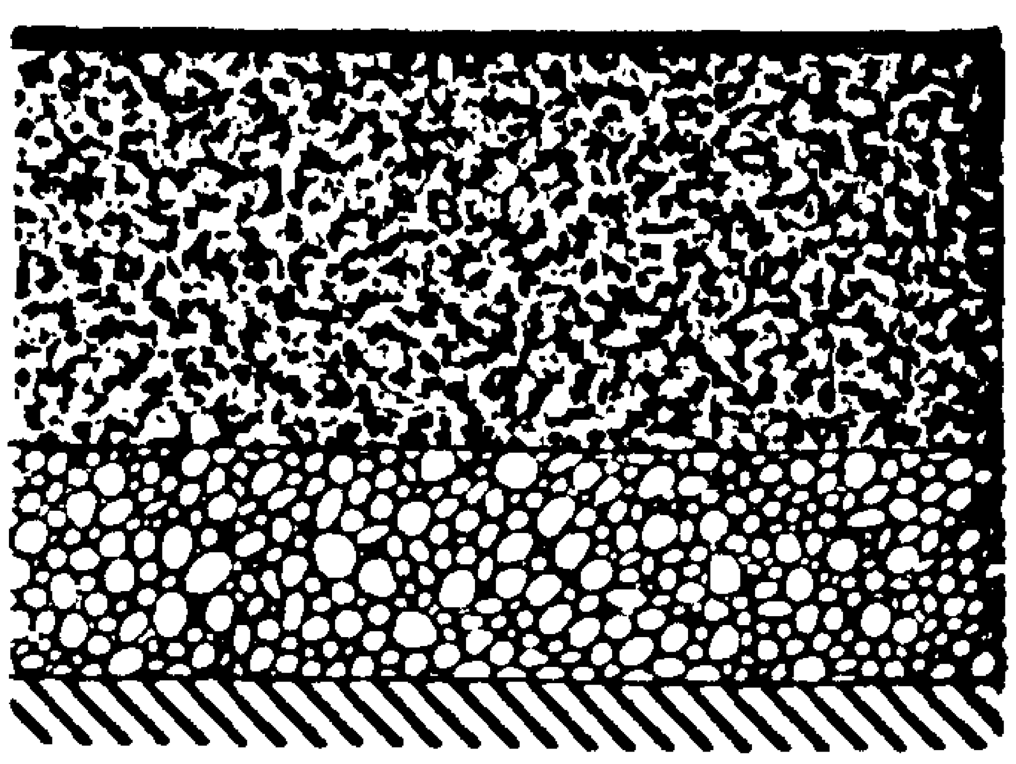
50 mm single course overlay of dense tar surfacing, 50% stone content

255 mm wet mix macadam base

150 mm type 2 granular sub-base

Container stacking area

(c)



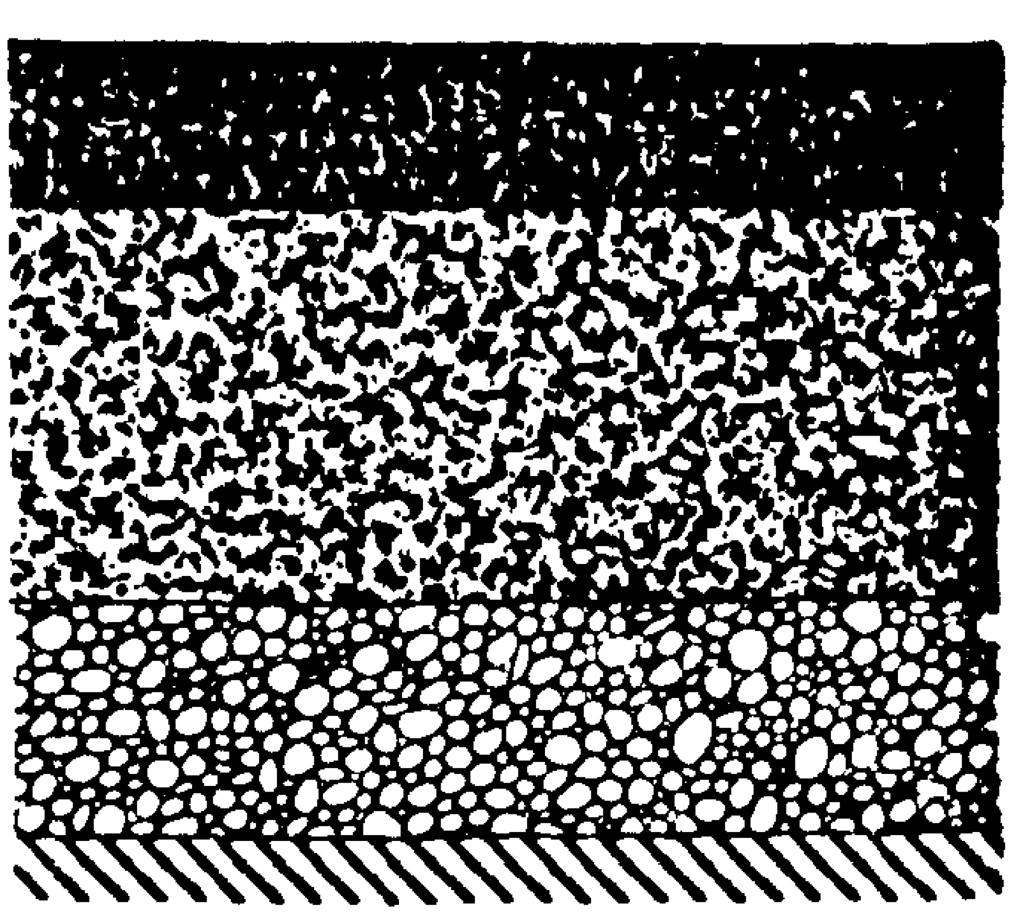
Two coat surface dressing:-
1st cost - tar or cut back bitumen blinded with 6 mm chippings
2nd coat - tar or bitumen blinded with 10 mm chippings

255 mm wet mix macadam base

150 mm type 2 granular sub-base

Trailer and stacking areas

(d)



100 mm combined asphalt wearing course with 19 mm coated chippings and basecourse of dense bitumen macadam

255 mm wet mix macadam base

150 mm type 2 granular sub-base

Roadways

problems associated with heavily loaded pavements :-

settlement

critical surface damage

hydraulic oil spillages

drains and manholes left proud

inadequate and blocked drainage channels

softening of asphalts in summer

very short maintenance free life

Considering the container handling area operated by a fleet of straddle carriers first, Figure 2.2 (a) and (b). The interchange areas, where the carrier operation is concentrated, have numerous potholes and pools of surface water; some places have been re-surfaced four times in its first 3-4 years of service. The 50 mm single course overlay, Figure 2.2 (b), has not stood up to the loading and is broken with many ponds and loose chippings.

Hydraulic oil spillages are blamed for much of the surface damage; a break in an hydraulic line can spill up to 180 litres of oil. Frequent hydraulic failure was a common fault with the older straddle carriers, however this situation has undoubtedly improved over the past few years with the development of better mechanical systems. Now many ports operate this type of vehicle and, with regular maintenance programmes, have few problems with oil spillage. However, Liverpool is reputed to have one of the worst records of oil pollution and this is almost certainly related to its poor maintenance programmes for both vehicles and pavements.

Oily surfaces are a major hazard for straddle carriers, particularly when cornering. To overcome this sand is sprinkled on spilt oil as a temporary measure until it is possible to clean the surface. However, the oil sand mixture hardens and forms bumps which are then very difficult to remove. Several detergents have been tried by the Port Authority, as well as various solvents, but these have proved to be either too weak or, if strong enough, have also attacked the bitumen binder. Crystal detergent and high pressure water is claimed to be the most succesful method.

The trailer parking area, Figure 2.2 (c), has a wet-mix macadam base with a two coat surface dressing of tar or cut-back bitumen. There are no problems with oil spillages as the straddle carriers are prohibited from this area. However, the surfacing has been completely torn away by very severe tyre screwing as trailers turn through 90 degrees into and out of the parking places, thus exposing the macadam base. Surface penetration by trailer dolly wheels, a common problem in Roll-on

Roll-off handling areas, has been overcome here by the provision of concrete strips.

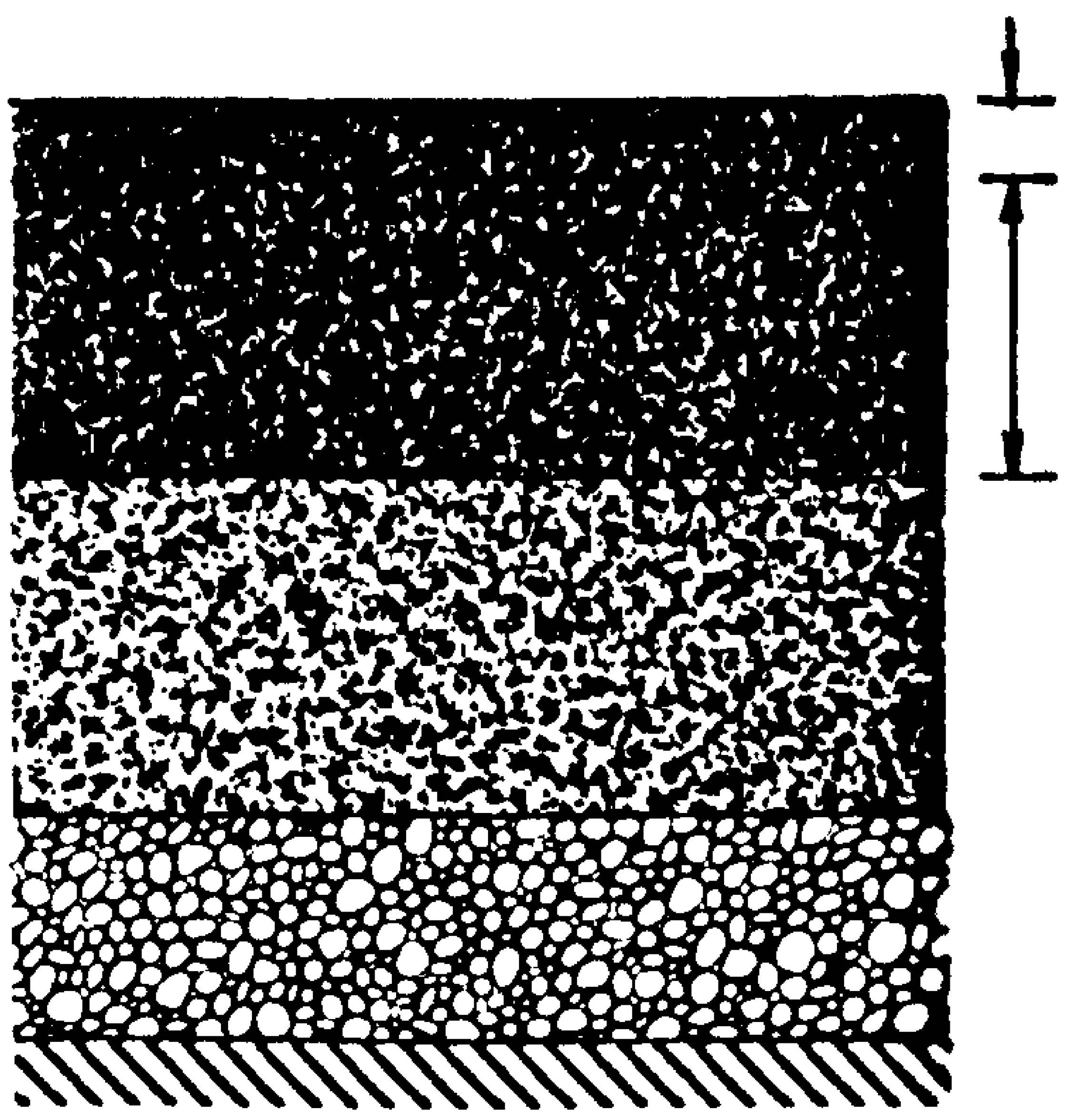
A similar construction to that in the container parks has been used for roadways carrying highway traffic to the interchange areas, Figure 2.2 (d). These roads are stronger than the paving in the straddle carrier park and are in good condition. This clearly demonstrates that hydraulic oil pollution is not the principal cause of pavement failure. It will be seen later, Chapter 7, that under high load conditions a wet mix macadam base course material has insufficient strength. The pavements in the handling areas are, therefore, basically under designed for the high wheel loadings carried.

The Port of Aarhus also has all asphaltic construction. The pavements were laid on about 9 metres of hydraulic sand fill, pumped from the seabed, which had then been left to stand for 12 months to allow settlement to take place. Container handling is carried out by front lift trucks, instead of straddle carriers, and whilst they are not so prone to oil spillages, the wheel loads are significantly greater.

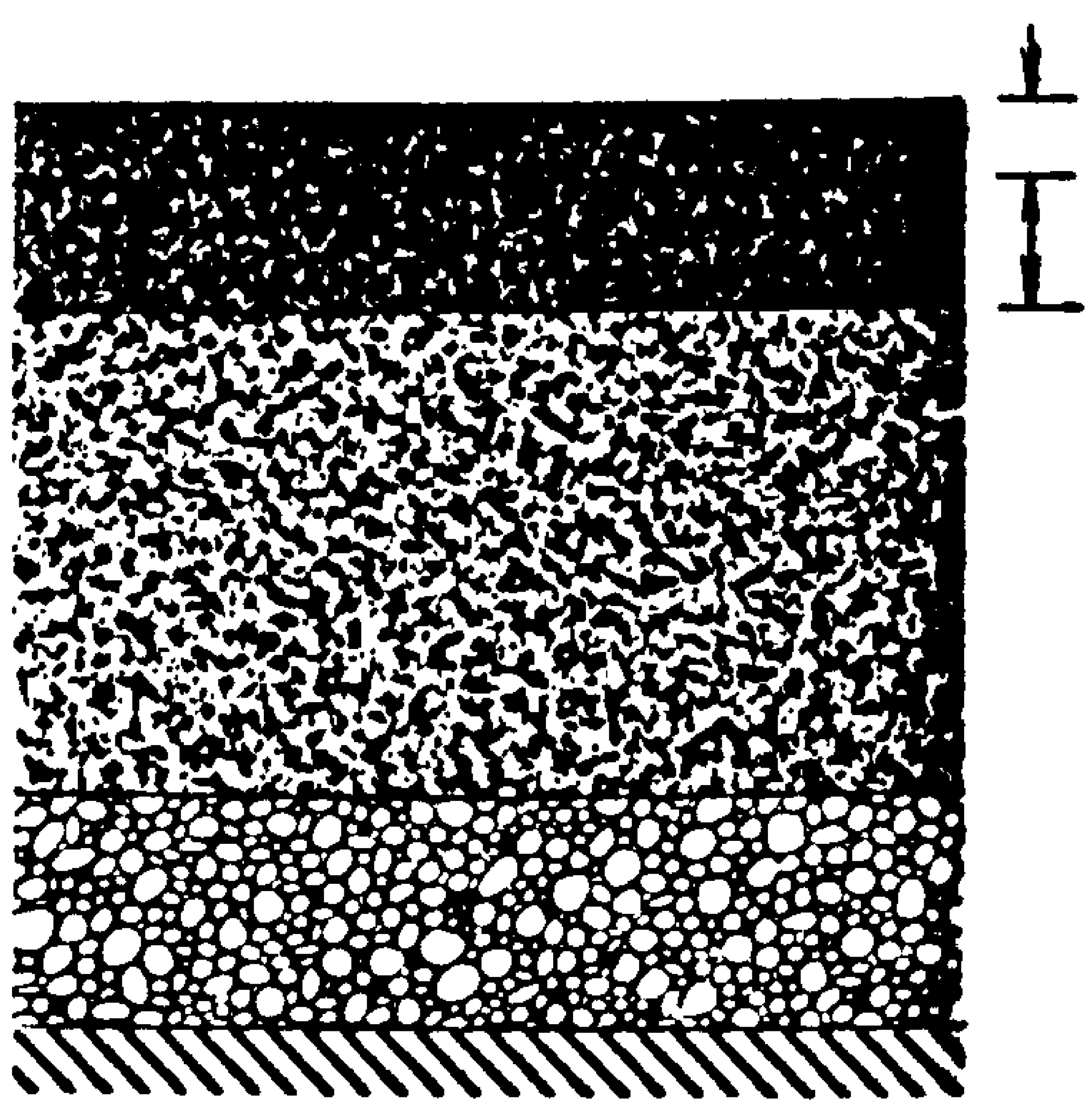
Although various forms of construction were considered, asphalt was chosen, partly due to the well established asphalt industry in Denmark. The first area, Figure 2.3 (a), laid in

Figure 2.3 Pavement constructions at Port of Aarhus

(a) 1969-70 construction



(b) 1973-74 construction



1969/70, was based on road design, modified to accommodate higher wheel loadings. In 1973/74, when a second area was laid, the specification was changed to that shown in Figure 2.3 (b). The latter construction adopted a stiffer base, thus reducing the required thickness of base-course, and, despite a slight reduction in total pavement thickness, pavement strength was increased; it also proved to be quicker and less expensive to construct. Settlement has been significant, up to 200 mm, and required some relaying of drains and minor re-surfacing work.

Both specifications have the same wearing course, an asphaltic mix compacted as densely as possible to ensure good frost resistance. However, the use of 100 pen. bitumen leads to softening in hot weather, causing indentations under container corner castings and trailer dolly wheels. The latter have been overcome by concrete strips whilst the indentations under corner castings are sufficiently shallow to pose no real problem. In conclusion, therefore, these two constructions have proved to be satisfactory.

A plan of the combined container and trailer berth at the Port of Ipswich was given in the previous Chapter, Figure 1.2. The main facilities were constructed in 1973 and, with developing container traffic, was extended in 1977 to provide an area for trailer parking, the extension being paved in concrete blocks. The first area is an all asphaltic construction.

The Port operates 4 four-wheeled Karricon straddle carriers using a "one over two" stacking operation. The pavement construction is shown in Figure 2.4, and it will be noticed that total pavement thickness is substantially less than the structures previously described. There are two reasons :-

(i) high strength asphalts are used throughout.

(ii) the subgrade is a dredged chalk material with self-cementing properties giving a very high strength. C.B.R. values varied between 16 and 40 %, with an average value of 26 %.

The wearing course is a high stone content asphaltic mix and, with slow vehicle speeds and high contact stresses, the following problems have developed :-

(a) surface stripping - under the action of oil contamination, tyre screwing and hot weather the surface becomes very smooth and slippery, particularly when wet.

Figure 2.4 Pavement construction at Port of Ipswich

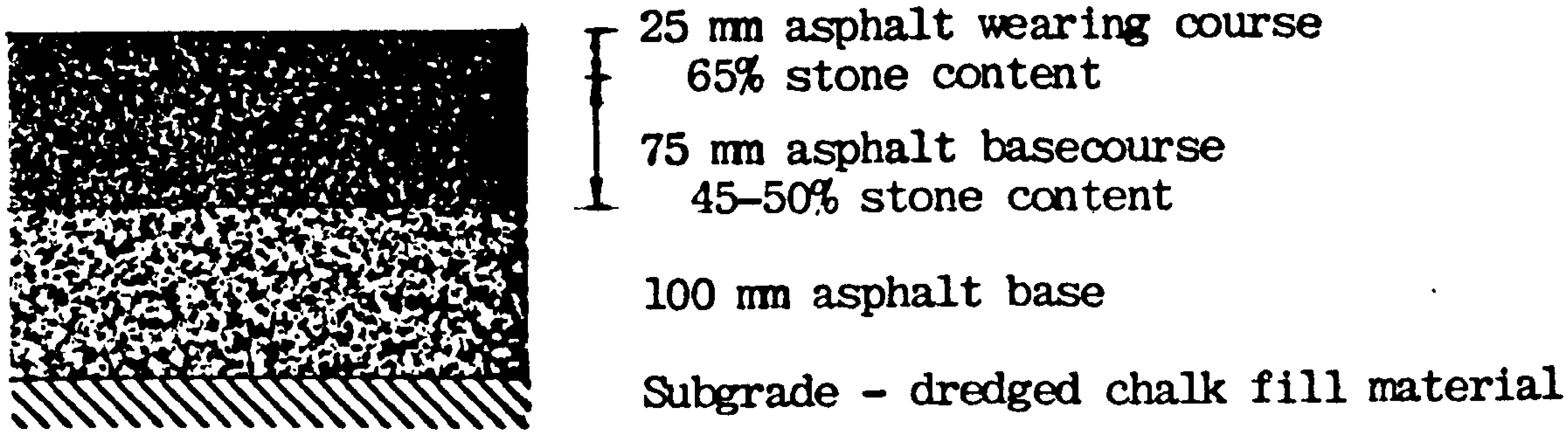
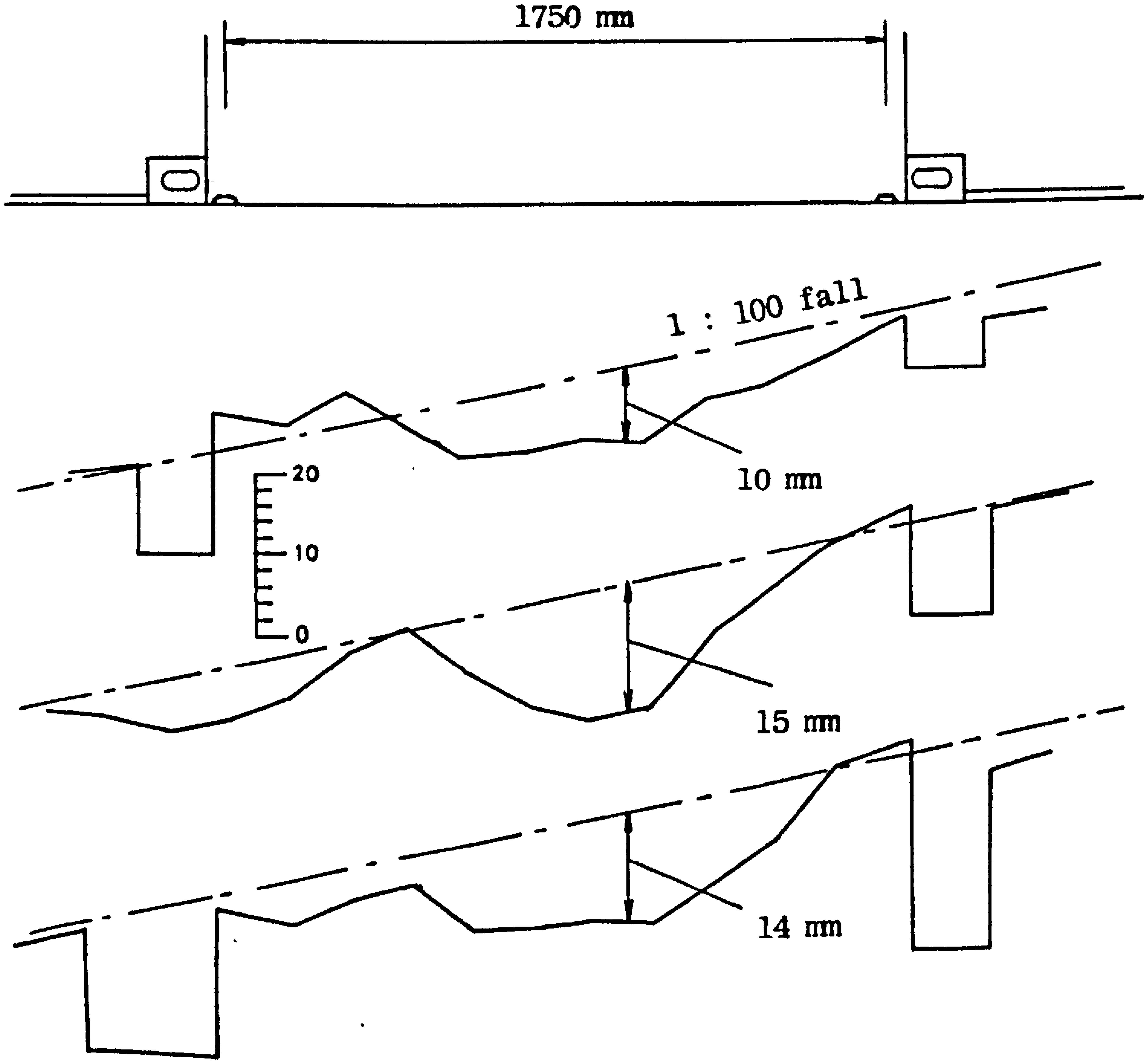


Figure 2.5 Surface profiles across container aisles



(b) rutting - this occurs mainly where straddle carrier operation is restricted to very narrow aisles between stacked containers. Three profiles across the aisles are shown in Figure 2.5 (a) to (c).

(c) indentations - two causes : corner castings of containers and dolly wheels of trailers. Whilst the former are not desirable they cause no real problem since surface penetration does not exceed 20 mm, as shown in Figure 2.5; dolly wheels are the cause of severe surface damage, Figures 2.6 (a) and (b).

Rutting is usually ironed out by shifting the lanes periodically, but at the time of the site visit they had been left for about 12 months and it seemed unlikely that it would be possible to move them in future, the ruts being too large to give an even support to stacked containers. The rut depths here are between 10 and 15 mm, but probably larger nearer the quayside where the lanes are more heavily trafficked. There is, however, little evidence of rutting away from the stacking area, despite heavier trafficking and sharp cornering. It seems, therefore, that wide operating lanes are sufficient to prevent severe of rutting, thus leaving the narrow lanes between container stacks the critical design criterion.

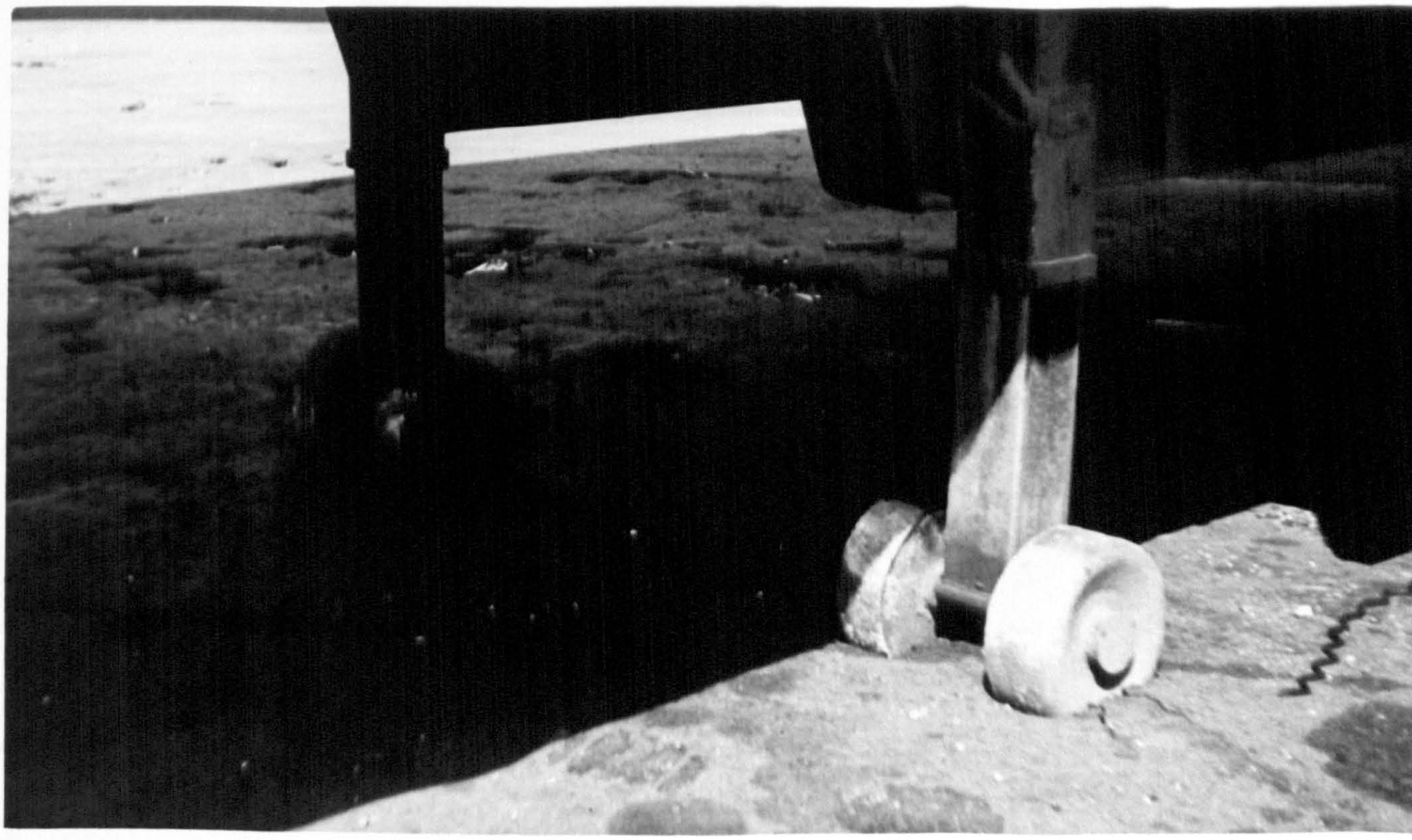
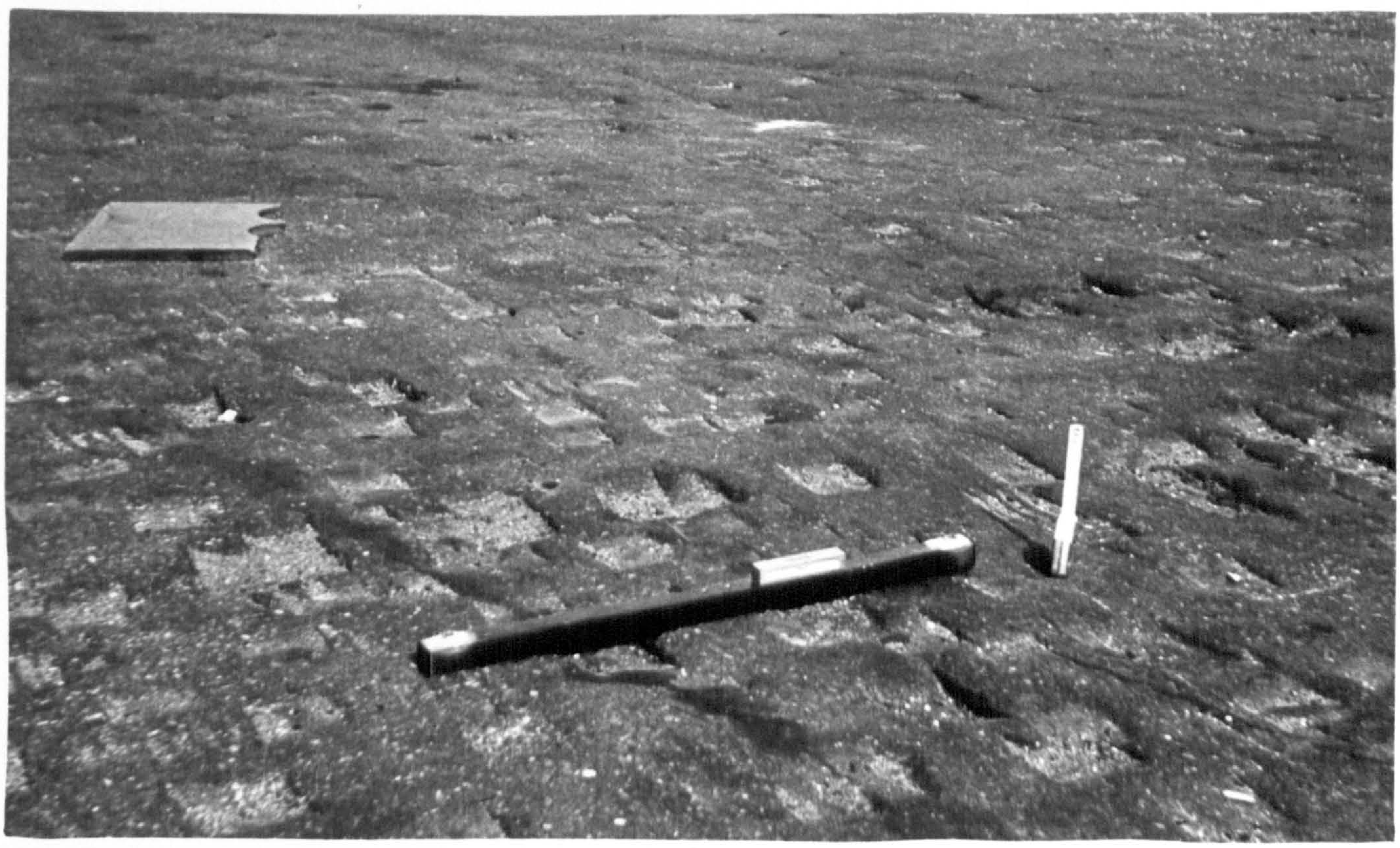


Figure 2.6 Damage to asphaltic surfacing



Indentation of the surfacing by trailer dolly wheels is a critical problem. Indentations of 50 mm were common, i.e. full depth of the surfacing, thus severely breaking-up the surface and leaving it very susceptible to frost action. In an attempt to overcome this, 6 mm steel plates (300 x 300 mm to 300 x 600 mm) have been placed under the dolly wheels, but, since drivers had to leave their cabs to position the plates correctly, there was a reluctance to use them and the measure failed. The most severe damage occurs when trailers are moved before the wheels have been lifted clear of the pavement, completely ruining the surfacing. Trailers having pivot plates instead of small diameter dolly wheels cause little or no damage to the pavement, and if these plates were specified for all trailers using a port, a lot of very expensive maintenance would be eliminated.

2.4. Composite Pavement Construction.

The use of composite construction, an asphaltic surfacing over a cement bound base material, is the most widely used form of construction. This generally overcomes the problem of providing adequate pavement strength, but at the cost of increased susceptibility to subgrade settlement, hence the selection of a suitable surfacing is critical.

Two of the largest container handling ports in the U.K., Felixstowe and the Port of London (Tilbury), use cement bound base materials almost exclusively. Different surfacings have been used with varying degrees of success.

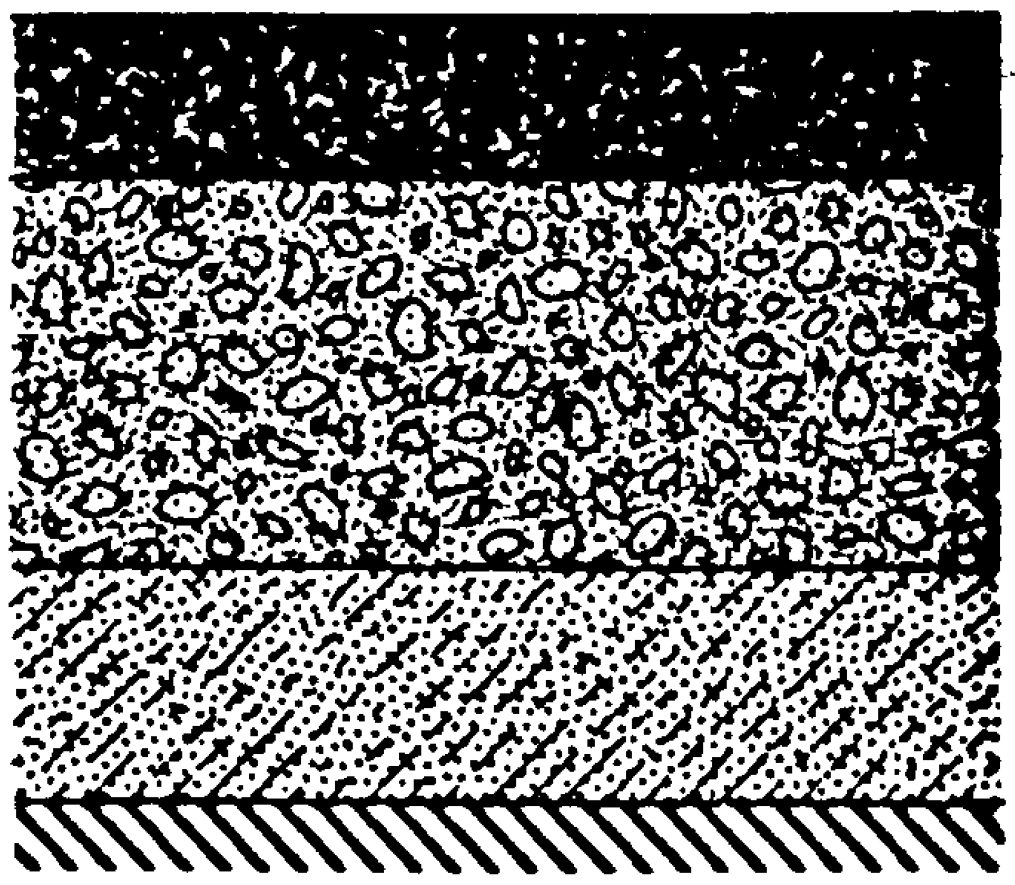
In terms of total tonnage throughput, the Port of Felixstowe is the largest port in the U.K.. The container handling quays were operated by a large fleet of straddle carriers, of various makes, with three rail mounted Portainer cranes for ship loading, the total handling area covers 20 hectares (50 acres). To increase annual through-put to 200,000 containers p.a., the Port was, at the time of this survey, installing a series of Transtainer cranes. These cranes span several rows of stacked containers and are backed up by a fleet of tractor/trailer units ferrying containers to and from the quayside.

The total area of the Port, some 160 hectares, is so large that (there are significant variations in the subgrade) over the site, ranging from beach sand and gravel to silt. The Port Authority has used two principal types of construction, Figure 2.7 (a) and (b). The surfacing is usually hot rolled asphalt, preferably 100 mm thick, but increased costs have reduced this to 75 mm in areas laid recently.

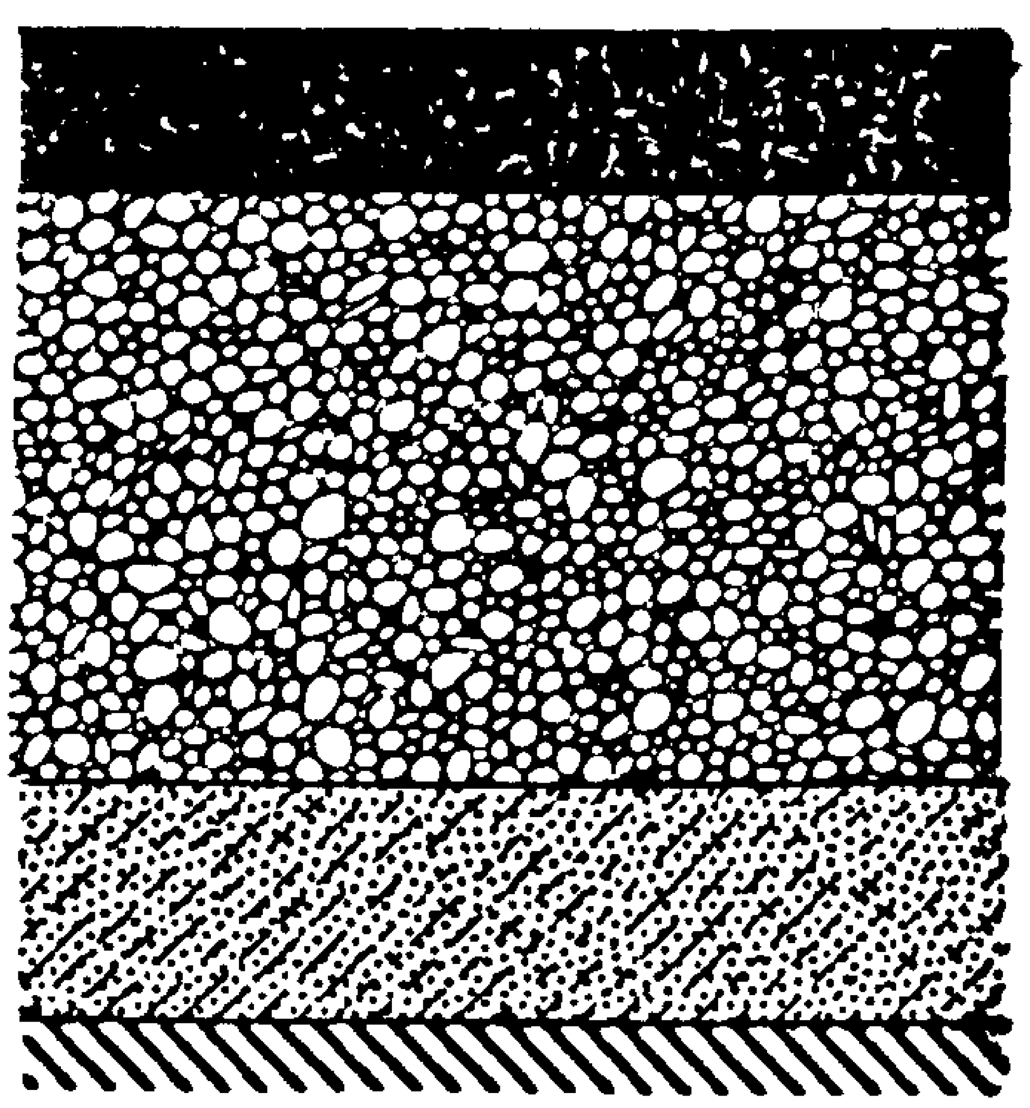
The pavements have been constructed on reclaimed land and

Figure 2.7 Pavement constructions at Port of Felixstowe

(a)



(b)



settlement was anticipated, thus the widespread use of a rigid pavement construction was impracticable. Settlement has been between 225 and 250 mm and has not had an appreciable effect on the pavement base materials. Four different base materials have been tried :-

(a) fly-ash

(b) slag

(c) crushed limestone

(d) dry lean concrete

The latter, lean concrete, is preferred, although it is expensive, and limestone has been used successfully in the lesser trafficked areas. However, the fly-ash and slag bases proved too weak in areas operated by straddle carriers and required complete replacement.

The container handling park, the largest single area in the port, is the only pavement specifically designed for container handling; a problem in common with many ports around the world, where changes in trade call for changes in site usage. The quayside, road trailer loading bay and maintenance areas are all constructed with reinforced concrete to minimise the problem of oil spillage and rutting.

Rutting in the container aisles has also been a severe problem at this port. /The actual rut depth is of little consequence, provided the surface is not cracked, it is the rate of change of rut depth that is critical to srtaddle carriers. This shifts the principal problem to the ends of the container stacks where carriers pull out into wider access lanes, and then run across the ruts.

Where granular sub-bases have broken down, ruts of between 75 and 100 mm have formed. Originally the affected areas were cut out and relaid, but this proved uneconomic in view of lost handling area. The situation was aggravated by a significant cut-back in maintenance spending. To correct the worst affected areas, a fine limestone asphalt was shovelled into the ruts, levelled and rolled. This proves adequate under tyres running up and down the aisles, but it does not withstand any turning action. The principal advantage of this technique is that it can be laid during lunch hours, hence the pavement is not taken out of service.

There was also a lot of damage from trailer dolly wheels in the Ro-Ro handling areas, indentations of 75 mm (full depth of the asphaltic surfacing) were common. This damage is usually left until it is so bad that it is worthwhile excavating a large area and installing 1 metre wide concrete strips for the wheels to stand on. Unfortunately this does not automatically solve the problem since the relative position of the dolly wheels is not the same for all trailers. The cost of providing these concrete strips is expensive, 125,000 per hectare (1977), particularly if there is no guarantee that the area will be used for trailer traffic after the next few years.

In trailer handling areas there is generally little evidence of structural distress in the pavement. Harsh tyre screwing, caterpillar tracks and shovels of heavy industrial plant passing through the port cause severe surface damage, but this is relatively easy to repair.

Tilbury Docks, Port of London Authority, is also a large container handling port. The docks were originally opened in 1886, but major expansion and modernisation did not start until 1963 and is still continuing; a new container berth was opened in September 1978. The history and development of the port and details of handling techniques used are well documented elsewhere, Refs. 2.11 to 2.13.

The soil structure is typical of the poor quality generally encountered in reclaimed port areas, Figure 2.8. During reclamation and construction settlements of over one metre have been recorded, with progressive settlements of 12 mm per year in the worst areas.

Figure 2.8 Soil conditions at Port of London (Tilbury)

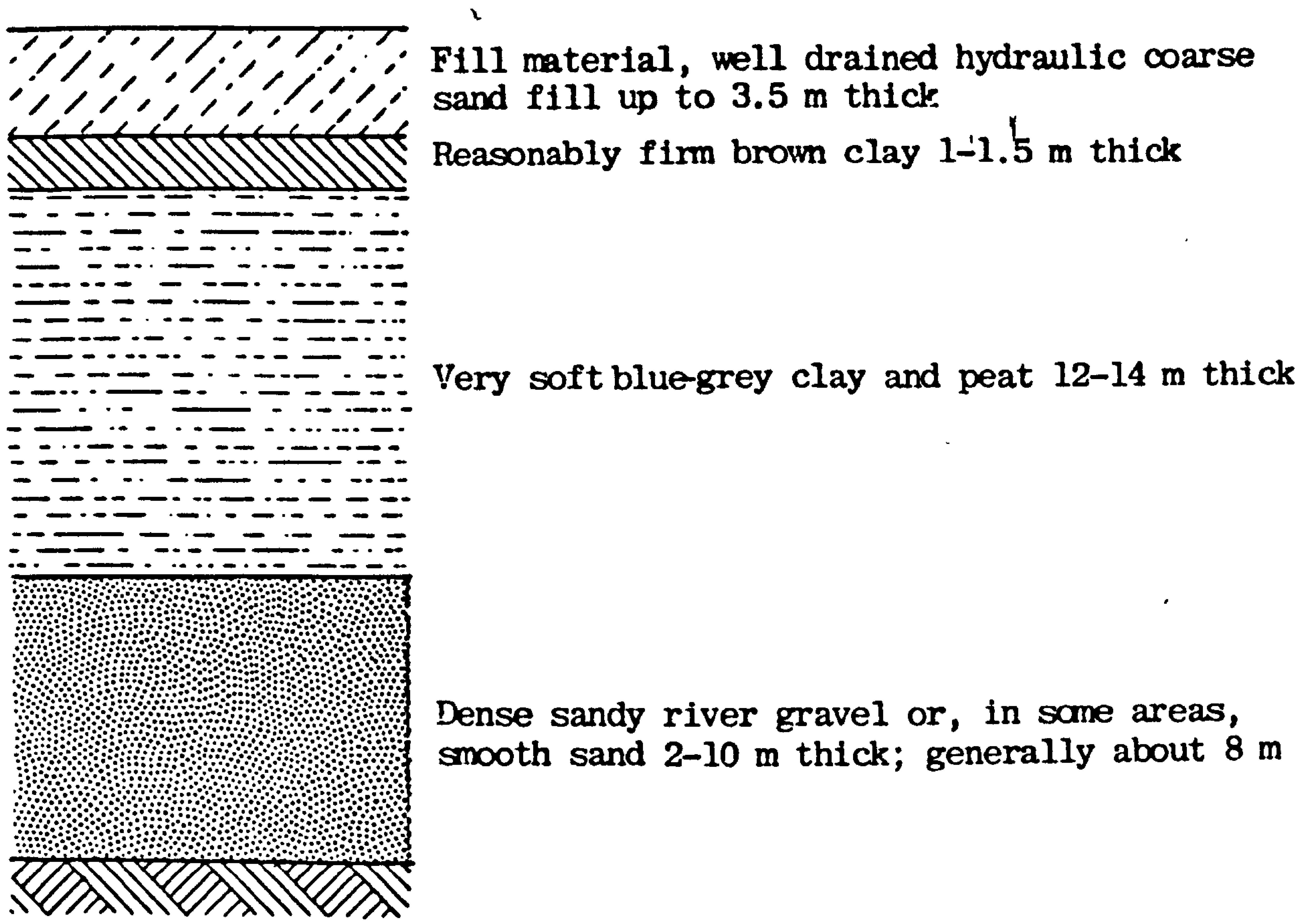
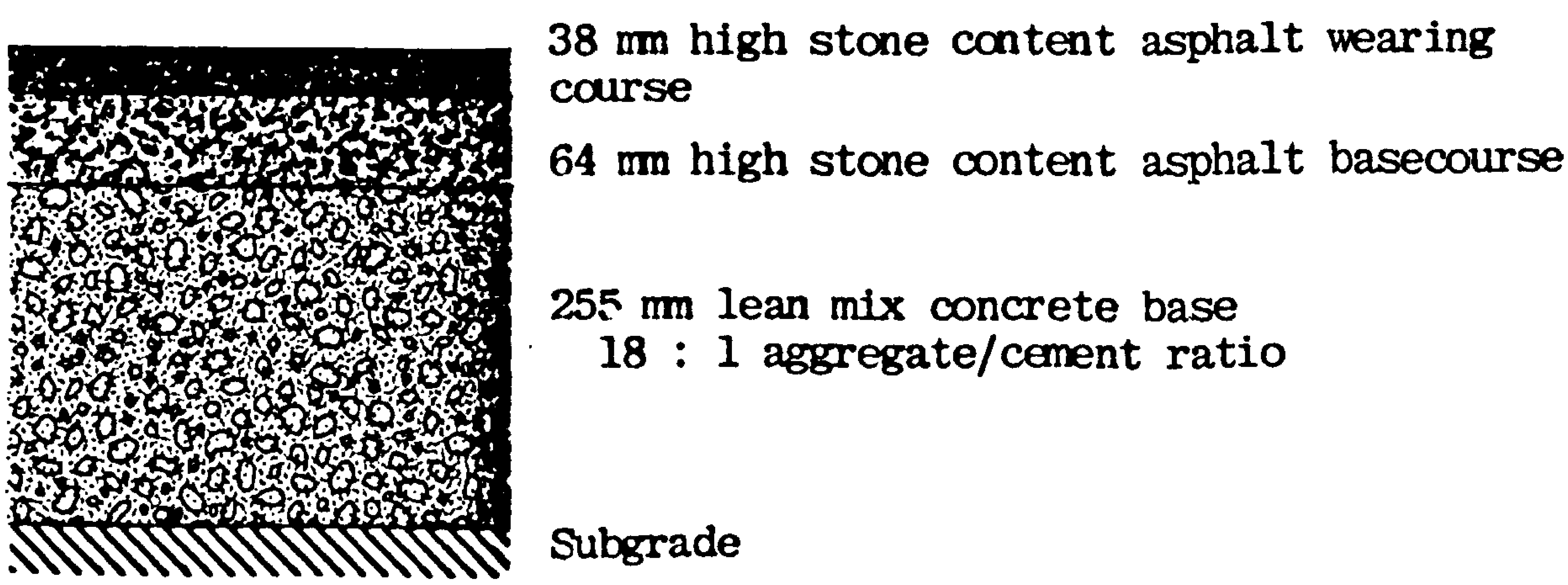


Figure 2.9 Pavement construction at Port of London (Tilbury)



Container handling is carried out by a large fleet of straddle carriers. The boxes are stacked in rows, 1.6 metres apart and 2 or 3 high, depending on the access required. In addition, one of the smaller handling areas is operated by front lift trucks, containers being block stacked three high.

All pavements constructed recently have followed the specification shown in Figure 2.9. In areas where a good subgrade is not found a layer of hardcore is provided to give a good working surface. The dry rolled lean concrete is easy to handle, it is simply tipped, levelled and then rolled. Compressive strengths at 28 days of 20 N/mm2 are commonly attained, although specified strength is only 10 N/mm2. Although cement bound bases require a curing period of 28 days, this rarely delays the construction programme since the surfacing can be laid during this period.

Under conditions of differential settlement a cementitious base does fracture, however since a concreting aggregate has been used there is a very high shear resistance across the cracks. This settlement does not appear to adversely affect the performance of the structure and experience elsewhere, at the Port of Europort in Rotterdam, has shown that a cement bound base can accommodate substantial settlements of over 1 metre.

Composite construction has generally stood up well in both container and trailer handling areas at Tilbury. Although container corner castings do penetrate the asphaltic surfacing, they cause no significant maintenance problems and surface damage caused by trailer dolly wheels is also minimal, unusual for an asphaltic surfacing. This is due to the specification of a high stone content wearing course, unlike the more usual highway asphalt specifications used at the Ports of Felixstowe and Ipswich. This type of wearing surface is one of the two materials that have stood up to the high contact stresses found in ports, the other being concrete blocks.

Rutting is no apparent problem at this port, even in areas operated by front lift trucks. Any areas of paving that do fail are completely excavated as soon as possible and reconstructed. At the time of this visit, only about 300 m2 of the total area of paved surfaces had required replacing, a very small proportion. Nearly all of the pavements in the port were between 1 and 10 years old and in excellent condition. The Port has also laid areas of special asphalt surfacings, Salviacim grouting and Epoxy asphalt, in areas of high contact stresses and severe oil contamination. These materials are discussed later in this Chapter.

2.5. In-situ Concrete.

A concrete pavement is a traditional form of construction for industrial areas. It provides a smooth, hard wearing surface which is not weakened by either oil spillages or high temperatures. It would seem, therefore, to provide the ideal solution to the paving problem, but there are two drawbacks :-

(i) site settlement cannot be accommodated without full depth cracking.

(ii) thermal expansion usually requires the installation of expansion joints.

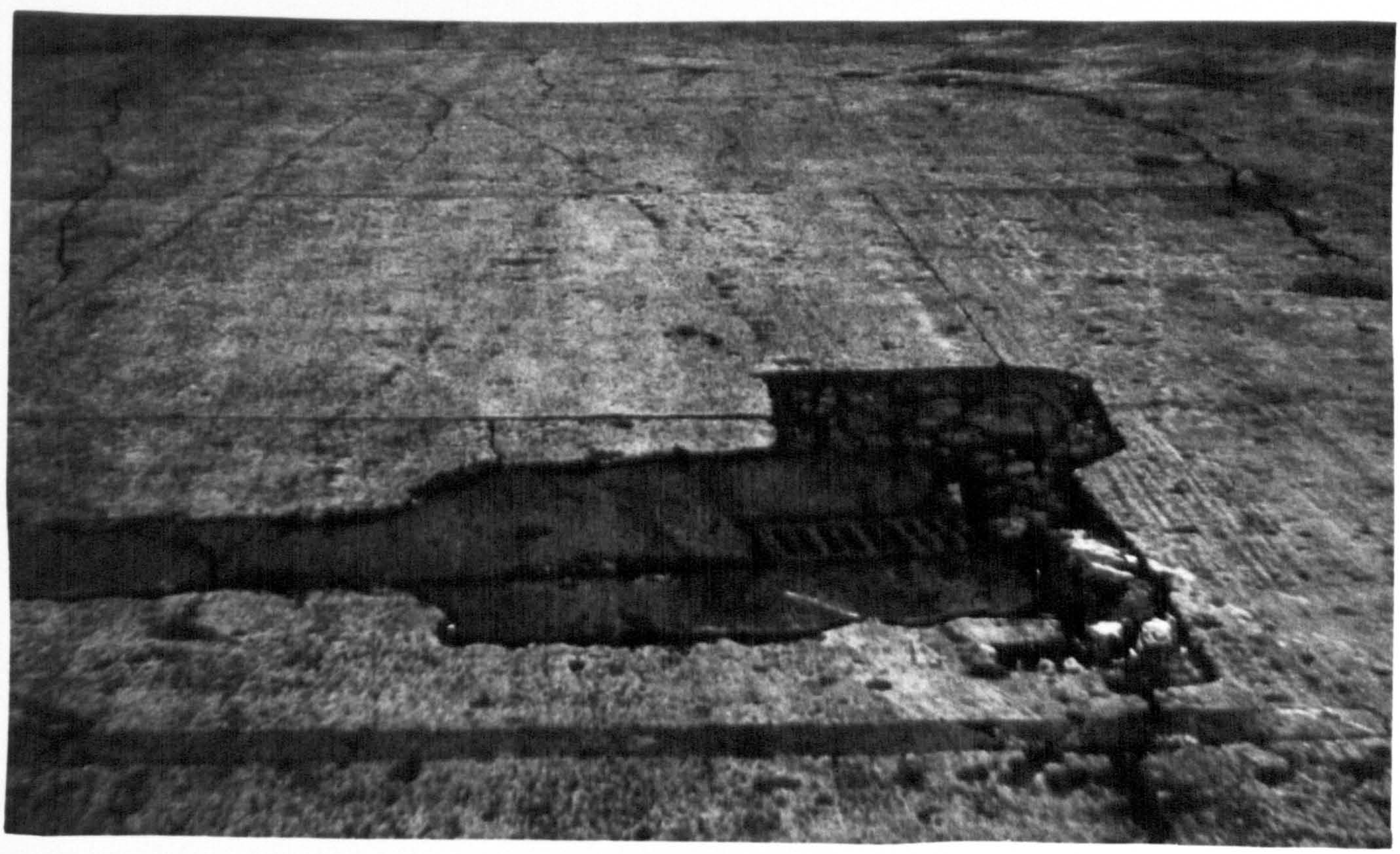
There are two forms of rigid construction, reinforced and un-reinforced. Un-reinforced concrete is completely unsuitable for use in port areas since the combination of contraction, high density vehicle movement and site settlement leads to very severe cracking in the early stages of life. Very few ports have this form of construction, and those that do exist are either badly cracked or not heavily loaded.

Reinforced concrete is more widely used. The steel content has to be designed such that thermal cracking does not occur under the combined stresses of wheel loading and the substantial temperature gradient through the slab. A hot morning sun or cool evening induces both hogging and sagging respectively along both axes of the slab, hence top and bottom reinforcement in both directions is required. Thermal stresses are additive to stresses under wheel loading when the surface of the slab is cooling. To allow overall expansion and contraction, the pavement is usually divided into regular areas, each separated by an expansion joint. These joints are both difficult and expensive to install and also form inherent weaknesses in the structure.

Adequately designed, in-situ concrete can provide a very durable pavement provided there is no subgrade settlement. However, should the supporting subgrade settle differentially, the increased bending moments under loading will cause rapid and severe cracking. This usually manifests itself along the edges of the construction joints and across the corners of the slabs, Figure 2.10.

The problems of construction joints can be overcome by the use of continuously reinforced pavement. In this case sufficient steel reinforcement is provided to distribute thermal stresses evenly throughout the pavement, thus ensuring that cracks are both fine and closely spaced (generally between one and two metres apart). Heavy reinforcement prevents cracks

Figure 2.10 Cracking in rigid pavement construction



opening up and shear load transfer is maintained by mechanical aggregate interlock. This form of construction gives a very good riding quality and decreased maintenance requirements, but only if no subgrade settlement occurs.

Reinforced concrete pavements that have been laid are generally 200 mm to 250 mm thick, with both top and bottom reinforcement. These have only withstood heavy trafficking in older areas of ports where settlement is minimal.

Access to services and rehabilitation of a concrete pavement is very difficult and should heavier loading conditions develop, strengthening of the structure would require a substantial overlay. A high concrete strength, at least 35 N/mm2, should be specified to reduce spalling and impact damage, this, together with a high steel content, makes reinforced concrete a relatively expensive form of paving. Unlike an asphaltic surfacing, where the tyre is supported by the aggregate, it is the cement matrix which is in contact with the tyre. This leads to a very low skid resistance when the pavement becomes oily and wet, although the surface is relatively easy to clean by mechanical plant. Since cement bound materials do not have the pronounced fatigue characteristics of the asphalts, a concrete structure can have a very long life span.

A novel system aimed at overcoming settlement was tried at Hafen, Hamburg. The pavement was made up of 4 m by 4 m by 140 mm thick in-situ slabs, each slab having four 50 mm diameter holes through which sand was injected as differential settlement took place. Unfortunately this system did not prove practicable and has not been repeated elsewhere. The concept of repacking the sub-base is not a new one, pre-cast concrete rafts have been used in ports, these units can be lifted out and relaid as necessary.

In conclusion, rigid pavement construction is rarely used in port areas; it being restricted to established settlement free areas or special applications, such as paper handling, where an asphaltic surfacing is unacceptable.

2.6. Precast Concrete Rafts.

Precast concrete rafts, or slabs, have been widely used throughout European Ports to give a hard surface which can be lifted readily to correct settlement. The slabs have either a champher or a steel surround along the top edges to prevent spalling. They are designed as single units with no load transfer between adjacent slabs, thus facilitating replacement.

Undoubtedly the most widely known concrete slabs are manufactured by Stelcon, based in Holland but with licensed manufacturers in several countries including the U.K.. The units are two metres square by 140 mm thick (maximum), with a protective steel angle along the upper edges. Where the subgrade is of low strength, or loading is heavy, a compacted granular sub-base is necessary. This must be free draining to prevent the development of "pumping" after periods of heavy rain. Rafts without steel surround are relatively uncommon, but some have been used in Hamburg where the standard size is slightly larger at 2.5 metres square.

(The units are bedded on a layer of screeded and compacted sand to give uniform support, 5 mm gaps are left between adjacent units.) Laying usually requires a four or five man team, with a 1500 kg capacity front lift truck to carry and place the rafts. Assumming the granular sub-base has been levelled and compacted, one gang can lay approximately 100 rafts per day, or 400 m2 of pavement.

Three ports in the U.K. have used rafts widely, the Ports of Manchester, Southampton and Greenock. However, only at the Port of Southampton are both the Port Authority and the Terminal Operators satisfied with the rafts' performance. The original specification used here was 114 mm thick units, but these proved unsatisfactory and have been replaced by 150 mm units.

The Port of Manchester has had severe problems with their raft paving which was attributed to poor drainage of sub-base and subgrade. Weakening or settlement of the subgrade very quickly leads to "pumping" or rocking of the units, this must be corrected quickly before the rafts become cracked. To prevent the bedding sand from pumping out, some sites have specified a sealer along all joints. Differential settlement leads to the development of ridges, or steps, between the units, in well maintained areas these are generally between 5 and 10 mm, but in neglected areas can be as much as 50 mm. This is obviously very damaging to pneumatic tyres, increasing the frequency of punctures, and also exposes the metal surround to impact loading. In the worst areas these angles have been completely torn off as they have been caught by trailer bars, forks of front lift trucks or dragged containers.

One inherent fault with rafts is related to their size and shape. The units are slightly larger than the track width of handling vehicles, thus inducing very large hogging bending moments. This is aggravated when vehicles run diagonally across the units, heavily stressing the concrete across the corners, particularly where slight subgrade settlement has occurred. This is liable to crack the rafts across the corners. It is acknowledged by the manufacturers that the very high point loading conditions makes the raft system unsuitable for areas trafficked by terminal trailer systems.

It is generally acknowledged that it is not the raft principle that is at fault. The units are sufficiently reinforced to accommodate induced bending stresses without fracture provided the supporting sub-base does not settle. This method of construction obviously has advantages with a wide range of applications in industrial areas, however further research is needed to establish a suitable design for the lower layers in the pavement structure.

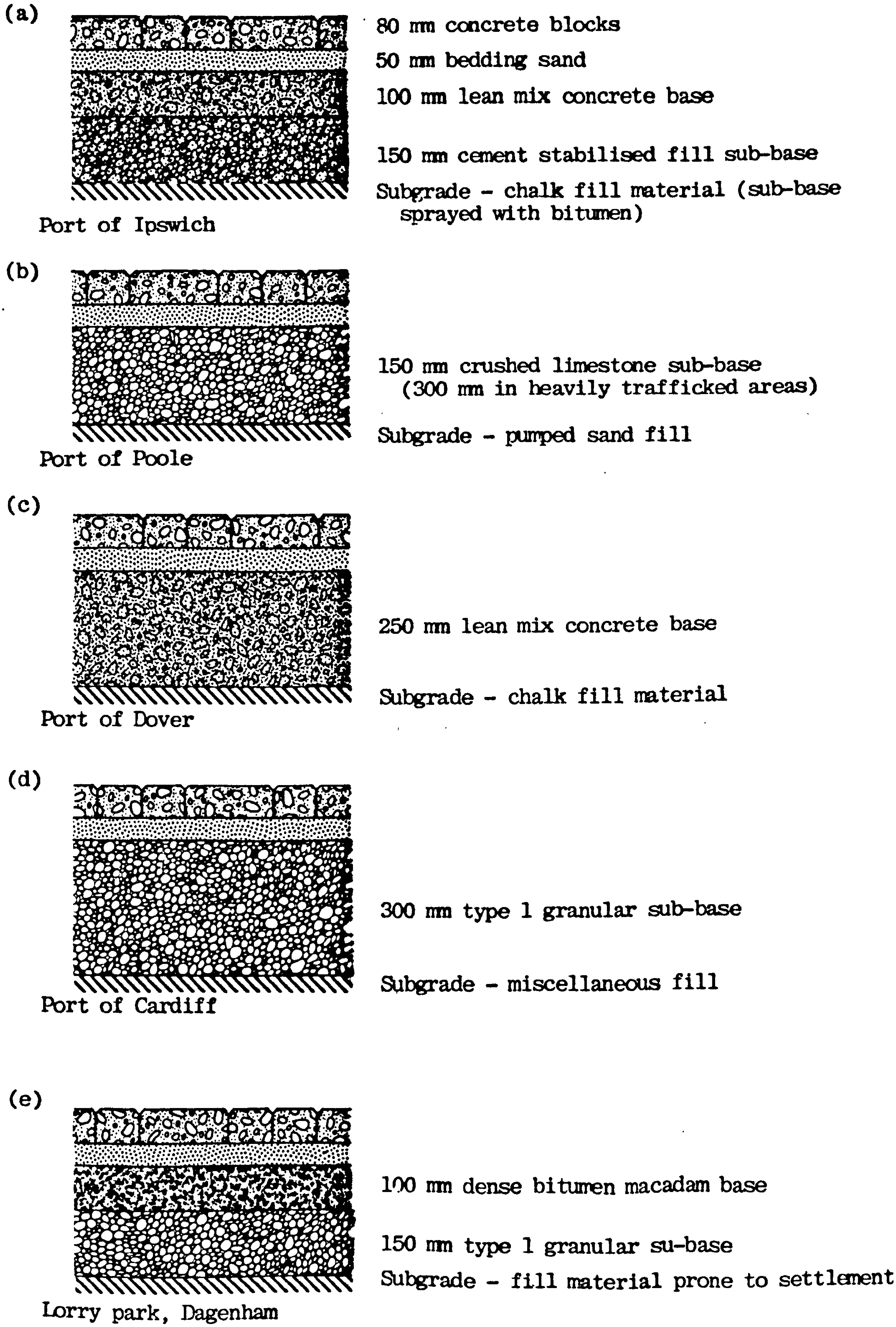
2.7. Concrete Block Paving.

Concrete blocks are a relatively new form of construction for port areas, but are rapidly establishing themselves as the nearest to the ideal requirement. Blocks are generally "hand sized", 100 by 200 mm in plan, but when laid they form an integral pavement surfacing. The finished surface combines the high strength of concrete whilst maintaining the flexibility associated with an asphaltic construction. Also, since the units are small, they are not prone to the large induced bending moments associated with precast rafts. Concrete blocks have been used widely in residential areas, bus lanes, etc., as well as industrial pavements in many countries; particularly Holland and Germany. A more detailed discussion on the use, development and properties of flexible block paving is given later in Chapter 5.

Concrete block paving has been used in several Ro-Ro or cargo storage areas in the U.K.: Cardiff, Dover, Poole and Ipswich. They have also been used for container handling areas operated by straddle carriers at Europort, Rotterdam, and Greenock JK, U.K.. 80 mm thick blocks, both rectangular and shaped, have been used at all industrial sites in the U.K. Since concrete blocks had only recently been introduced into the U.K., there was no experience on which to base design recommendations. Design details of five U.K. constructions are shown in Figure 2.11, (a) to (e). The last of these, at Dagenham, is not a port installation but an inland lorry park. It is included here since it is unique in that it adopts a dense bitumen macadam base for "increased flexibility" under settlement; vehicle loading is similar to that found in a Ro-Ro terminal.

The majority of these sites are trafficked by highway or terminal trailers. The operators were looking for a surfacing which could withstand the high contact stresses of dolly wheels whilst being sufficiently flexible to accommodate settlement. The high resistance to penetration is clearly shown in Figure

Figure 2.11 U.K. pavement constructions with block surfacing



2.12 (a) and (b), compared to the earlier Figures of similar loading on an asphaltic surfacing.

Dover was the first U.K. port to try block paving and a test area of 400 m2 was constructed in the trailer Ro-Ro parking area. The existing 200 mm thick concrete paving was removed, the granular sub-base increased to 280 mm and 80 mm 'Uniblock' laid in the normal manner (see Chapter 5). However, immediately after construction there was a heavy rainstorm and surface water percolated through the joints, saturating the sub-base. The sub-base, a chalk material, was of very poor quality and lost virtually all its strength resulting in a dramatic and immediate pavement failure when trafficked. The Port Authority was convinced that blocks would provide a suitable surfacing given adequate base support and the pavement was relaid with a 250 mm lean concrete base. This area has performed satisfactorily and 40,000 m2 of the same blocks were laid in 1980 on reclaimed land. Although the specification for this second area required a cement bound base, the Port Authority maintain that any granular material giving a minimum CBR of 15 % and not susceptible to water, would be adequate for this type of loading. The problem at Dover was that good quality sub-base aggregates were very difficult to obtain locally. This experience has been bourne out at Cardiff and Poole where, with similar loading conditions, both structures have proved perfectly satisfactory.

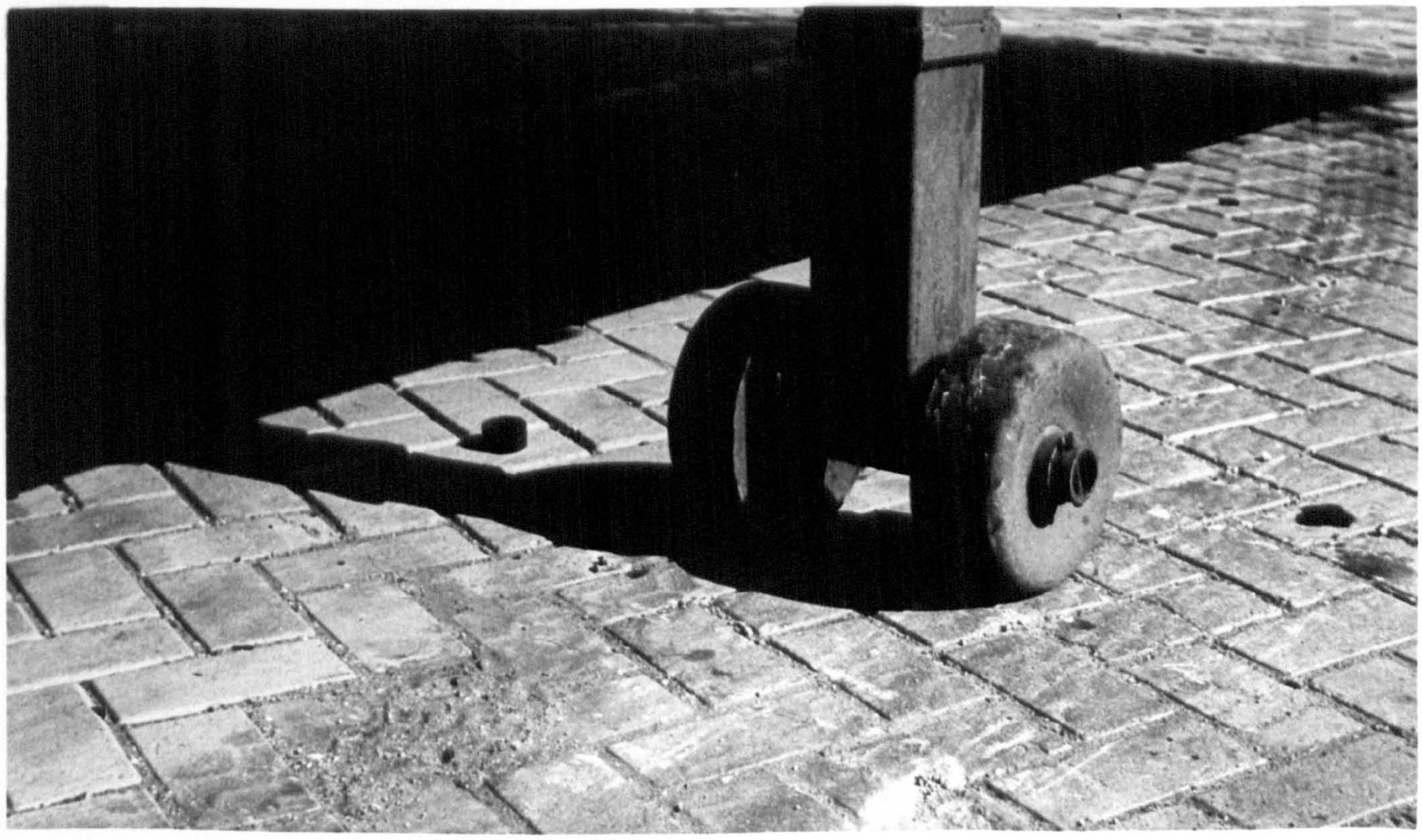
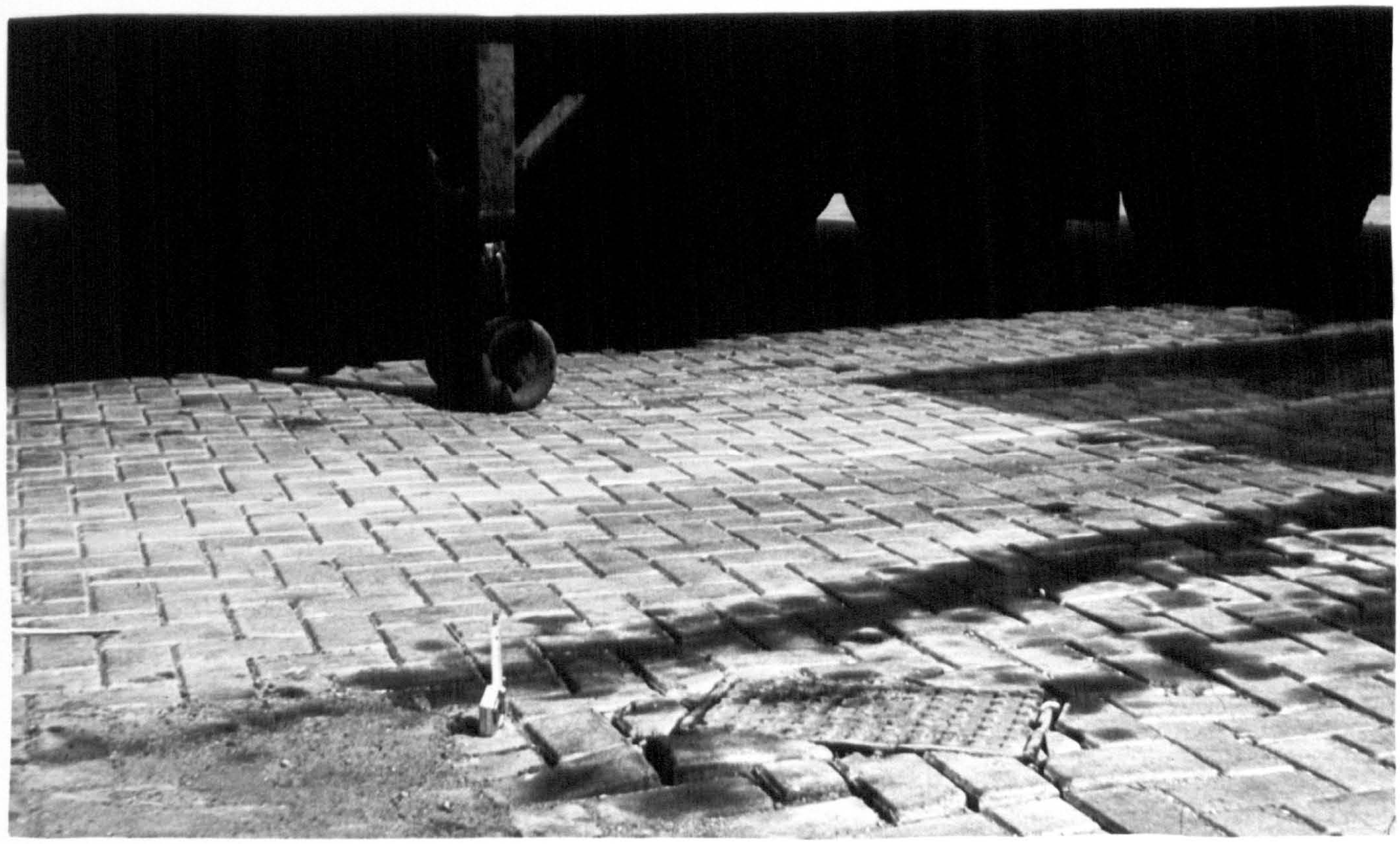


Figure 2.12 Trailer dolly wheels on block surfacing



There is some concern over percolation of surface water through the joints between blocks, particularly in the period immediately after construction. It is generally acknowledged that detritus, rubber, oil etc., reduce the rate of percolation, although it is unlikely there will be a complete seal. To overcome this a waterproof membrane has been included in some constructions, e.g. bitumen spray, polythene sheeting, asphaltic base. However, it has not been shown conclusively that these measures have overcome the problem, or indeed if there is a problem. At most ports the water table is very near the surface, rising and falling with the tide, and water will often remain in drainage sumps.

Europort, Rotterdam, is the largest port in the world, handling over 500,000 containers per year, and the majority of paved areas are made up of blocks. In a port, it is possible to economise on the cost of the paving if specific areas are designated as "no go" areas for straddle carriers or front lift trucks at the design stage. Many ports choose not to do this as it may restrict any future development plans, but it has been done here. The areas not trafficked by straddle carriers, reception areas for highway traffic, are made up of 120 mm thick concrete blocks laid directly on the gravel subgrade. In container handling areas a heavier construction is used, Figure

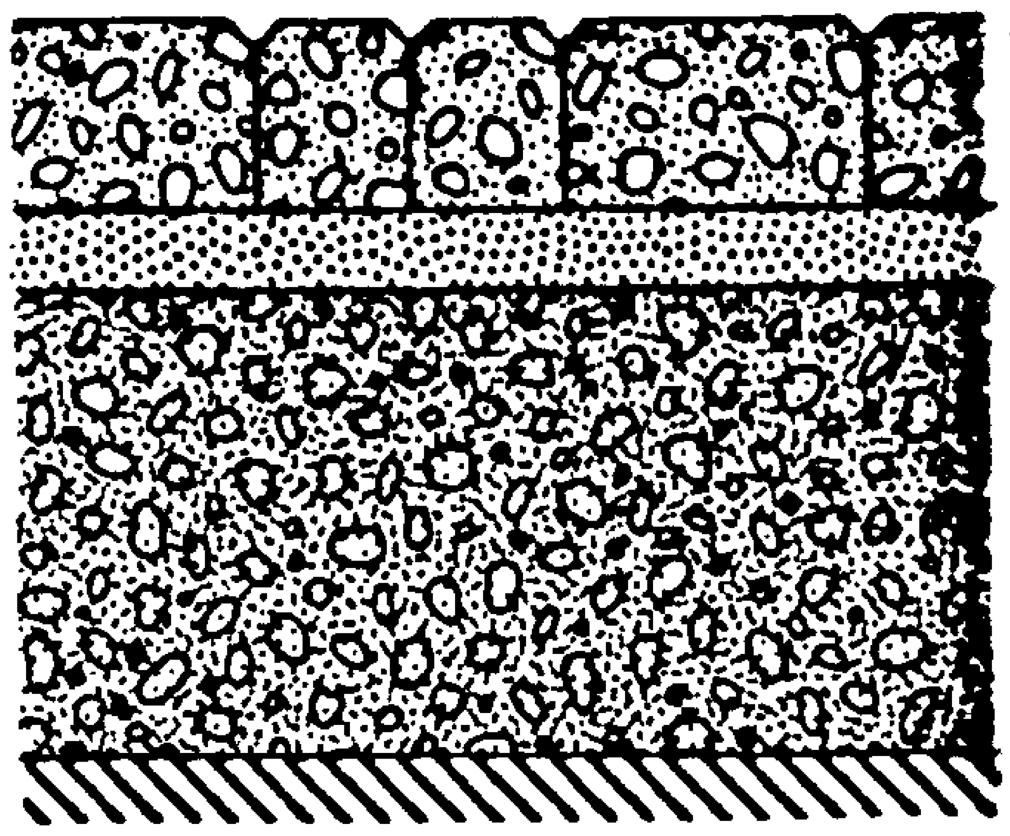
2.13 (a), where the blocks are laid on cement stabilised sand fill. Site settlement has been substantial, over 1 m in places, but the blocks and the cementicious base have performed well and maintenance costs have been minimal.

Two other European block constructions are shown in Figure 2.13, Karlsruhe container terminal and a lorry park at Dusseldorf Trade Fair. Both these pavements have substantial granular layers for frost protection of clay subgrades, hence overall thicknesses are significantly greater than equivalent U.K. constructions.

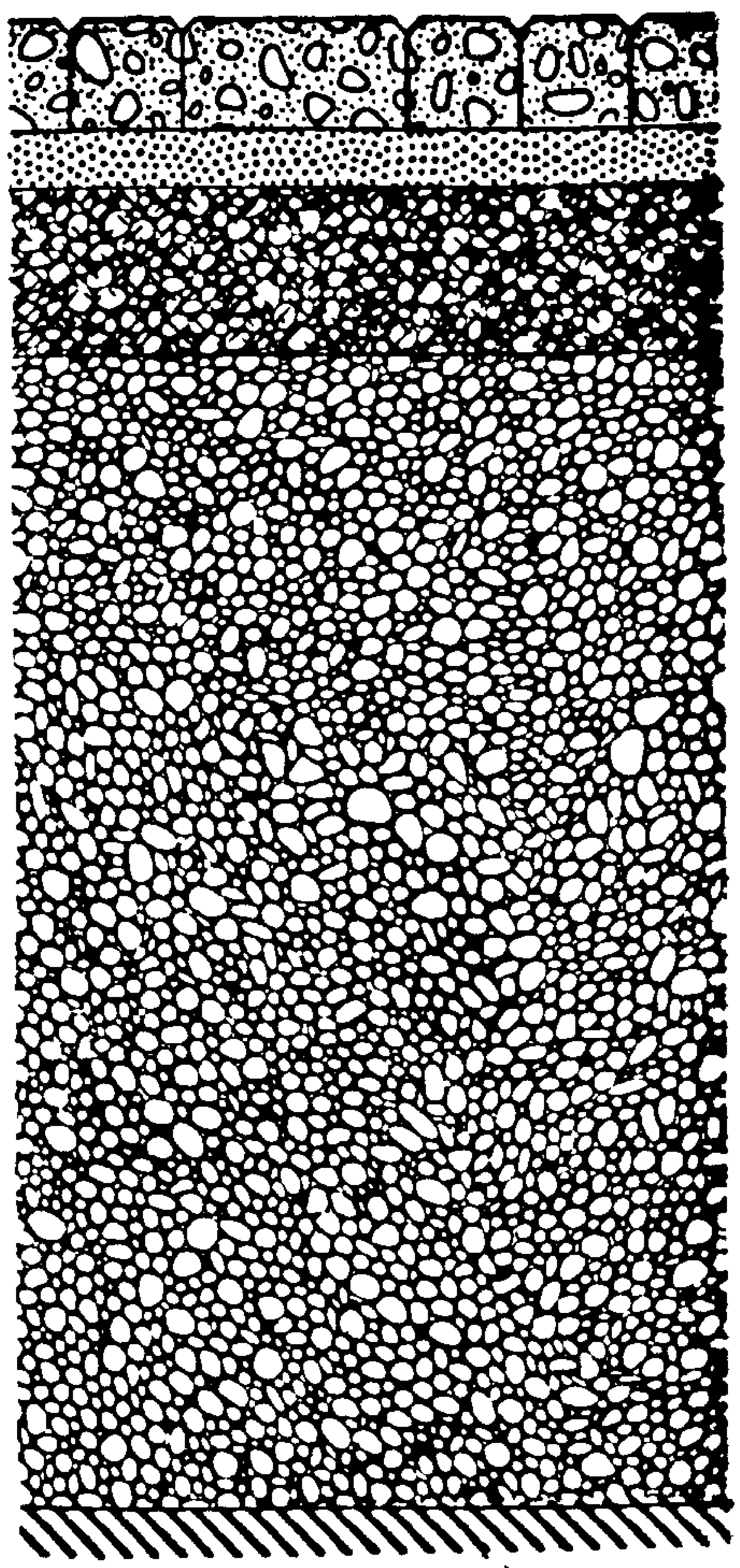
There is no doubt that concrete blocks provide one of the most durable and tolerant forms of construction. The riding quality is not as good as that from an asphaltic surfacing, but has proved more than adequate for straddle carrier operation. With the wider use of concrete blocks, initial problems of ensuring a satisfactory and economic construction technique have been overcome. In the U.K. over 50 % of the blocks laid each year are used in industrial or heavy loading environments. The increased turnover has made the cost of construction of a block pavement comparable to that of other materials, and, with lower maintenance cost requirements, makes the use of blocks an economic solution for industrial areas.

Figure 2.13 European pavement constructions with block surfacing

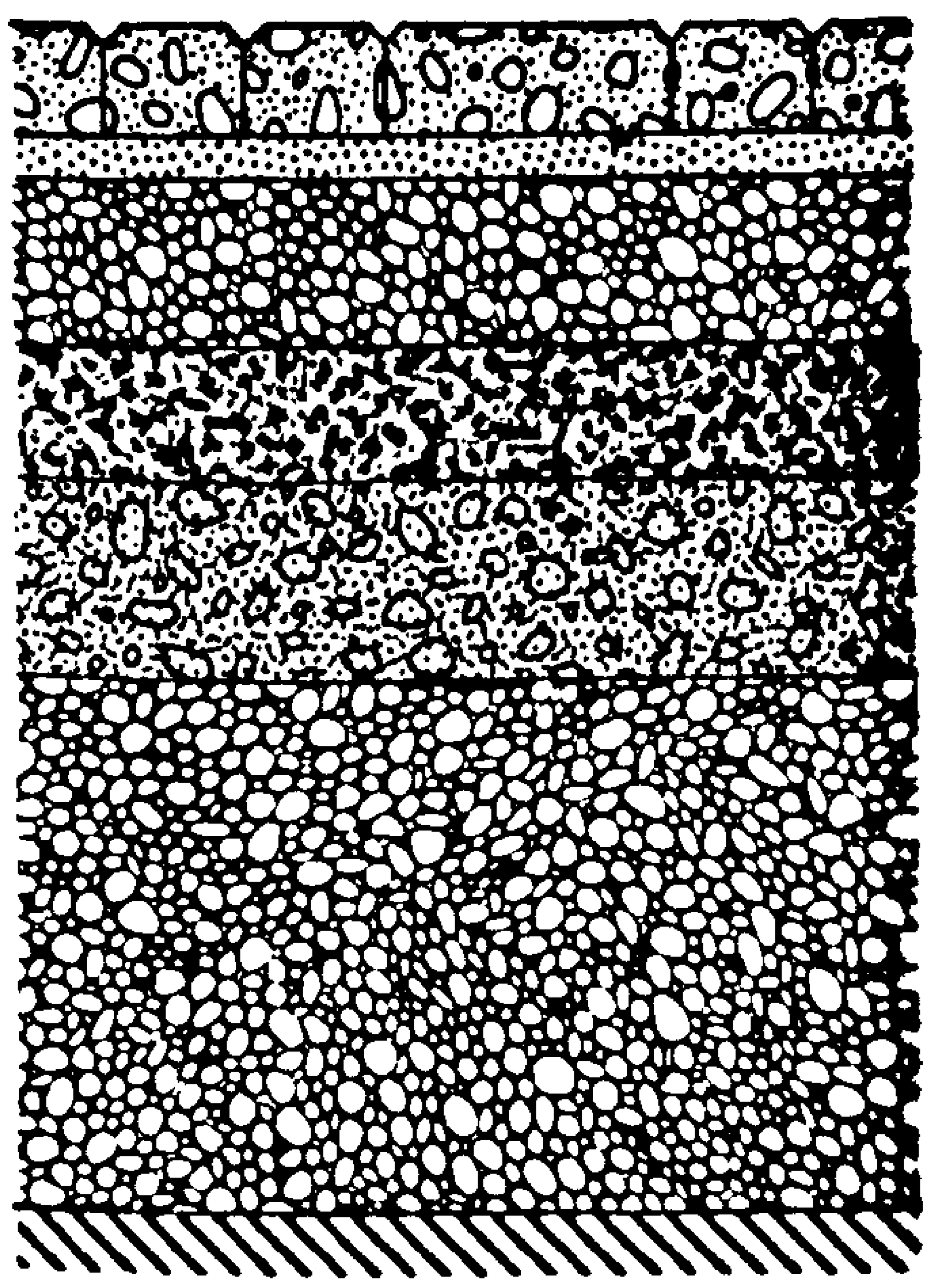
(a)



(b)



(c)



The base, or sub-base, is the critical factor in pavement design. For blocks it is generally found that a granular sub-base is sufficient for areas carrying loads of the same order as highway traffic, but for container handling areas a cement bound base is required.

2.8. Other Forms of Construction.

There are two other forms of construction in common use in ports. These are :-

(a) Special asphalt mixes

(b) Bituminous spray surfacing

2.8.1. Special Asphaltic Mixes.

It is possible to combine the flexibility of asphalt with the rigidity of concrete to provide a semi-rigid wearing course which is resistant to temperature, oil and high contact stresses. This is available in the U.K. under several brand names, 'Salviacim', 'Hardicrete' and 'Worthicim', but all materials use essentially the same technique, ~~see Figure 2.14.~~ JK

Approximately 40 mm of open textured bitumen coated aggregate, no fines, is laid into which is rolled / vibrated a resin cement grout which is then left to harden, since it is a cement grout it takes around 7 days to reach adequate strength. The final stiffness of the surfacing is approximately 12,000 N/mm2, roughly twice that of hot rolled asphalt. However, a high strength, or semi-rigid, surfacing is very susceptible to large tensile strains created at the bottom of the layer as a wheel passes over the surface. Cement based materials have very low tensile strengths and cracking will occur if these are exceeded, thus the strength of the supporting base is important.)

Tilbury, Port of London Authority, laid an area of Salviacim on a general cargo handling berth. The surfacing has stood up well to high contact stresses but it is not sufficiently flexible to follow site settlement without cracking; all the cracks are fine and approximately 1 metre apart. This does not appear to have an adverse effect on the pavement structure, but it is contrary to a traditional asphaltic mix which would have rolled out. Since the surfacing had been laid within the previous 18 months no conclusions on performance could be made, but there were no visible signs of wear or distress. The cost of this form of surfacing was approximately £20/m2 (1980). It must also be laid on a reasonably strong base to prevent tensile cracking under trafficking.

Epoxy asphalts have also been developed. These mixes are resistant to oil attack which makes them highly suitable to areas of high oil contamination, e.g. maintenance areas. Tilbury has a small area of epoxy surfacing in the straddle carrier refueling area and this has proved very successful, whereas the adjacent hot rolled asphalt has been very severely attacked. However, again the cost of construction is high which makes it impracticable to pave a complete handling area with this material. Also, the thermo-setting asphalt mix hardens within 90 minutes of mixing, hence laying has to be fast and efficent.

2.8.2. Bituminous Spray Surfacings.

There are several areas where it is impracticable to construct a full pavement but a clean running surface is required both quickly and cheaply. A bituminous surface spray, or tack coat, is an obvious temporary solution.

This has been done at several ports where cargo is principally light vehicles. Generally a crushed limestone sub-base, 150 to 300 mm thick, is sealed by rolling and washing with limestone dust and then surface dressed using a double treatment of hot cationic bitumen emulsion and 6 mm chippings. This proved adequate for areas not trafficked by trailers, but harsh tyre screwing quickly tore the surface in the main access lanes. Hot tar and chipping surface dressing have also been tried, but these proved unsuccessful.

This method of surfacing can be considered as only a temporary measure to prevent frost and rain turning a granular sub-base into a quagmire. The surface dressing can only withstand light trafficking, e.g. production cars, and harsh turning will tear the surface; it is completely unsuitable for straddle carriers etc. However it could be an ideal solution for a site where substantial settlement is expected to occur in the first 12 or 18 months. Final pavement construction can then begin after the worst settlement has taken place.

2.9. Relative costs.

The final selection of a type of pavement construction is based on both initial construction and future maintenance costs. Unfortunately these are difficult to assess accurately in a general appraisal since they are dependant on the availability of raw materials. Cost comparisons between different countries is even more difficult due to variations in labour costs and different construction techniques.

The following comparative construction costs for different paving materials, in units per square metre, are based on an area in southern Britain and are included as an approximate guide only.

asphalt	530 per 100 mm.
concrete blocks	800
precast rafts	1950
lean concrete	250 per 100 mm.
type 1 granular	75 per 100 mm.
type 2 granular	65 per 100 mm.
cement stabilisation	175 per 100 mm.

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CHAPTER THREE

CHARACTERISATION OF PAVEMENT MATERIALS

Synopsis

This Chapter summarises past research on the characterisation of pavement materials. The four materials commonly used in pavement construction are covered; asphaltic, cementitious, granular and the subgrade. The information presented is not comprehensive but forms the basic data for analytical design methods described in Chapters 4 and 7. Full references are included.

3.1. Introduction.

The selection of suitable materials for pavement construction is usually ensured by the use of empirically established specifications, based on simple grading or proportion classifications. These specifications are still adopted since they require only simple laboratory or on-site tests, hence offering an easy and in-expensive method of maintaining good quality control.

Research into the behaviour of pavement structures has been directed towards the analysis of mathematical models, and relating the results to ultimate strengths of the component materials. All materials used in pavement construction are non-isotropic and exhibit varying degrees of elastic, plastic and viscous characteristics. A mathematical model can be no more accurate than the accuracy of the input variables, hence research into material characterisation has followed closely behind the development of the analysis methods. The only analytical technique available which encompasses this degree of complexity is the finite element method. Unfortunately this requires a vast amount of computer resources, and the information currently available on materials is too variable to justify the costs involved in obtaining a "false accuracy".

Material characterisation has, therefore, been directed towards quantifying the elastic, both linear and non-linear, and the visco-plastic behaviour. The former is used for the analysis of the resilient response of the pavement structure, whilst the latter is related to long term deformation, principally rutting in asphaltic materials. The effects of temperature variation on asphaltic and cement bound materials are also parameters in the basic characterisation.

There are four types of materials commonly used in pavement construction, asphaltic, cementitious, granular and non-granular subgrade. The principal structural element in a highway pavement is usually asphaltic, and the majority of pavement research has been developed by the manufacturers of bituminous products, hence there is extensive detailed information available on the behaviour of bituminous materials. Research on cementitious materials has been much slower to develop, except for structural concrete materials used in slab or rigid pavement construction.

This trend of decreased research down the pavement layers continues for the granular sub-base and the subgrade. Here, parameters substituted into the mathematical models are usually based on simple empirical relationships. Although variations in the lower layers of pavement do not have a significant effect on the structural behaviour, it is clear that material characterisation is far from an "exact science".

This Chapter summarises past research on material characterisation which has been adopted by the principal analysis and design methods. The information presented is not comprehensive, but it forms the basic data input for the development of the pavement analysis program described in Chapter 7. The Chapter is divided into four sections, each covering one of the materials given above.

3.2. The Subgrade.

Of all components of pavement design, the characterisation of the subgrade has received the least attention. This is presumably because different soil types are so variable that they cannot be accommodated in a comprehensive yet simple form. A principal requirement of subgrade characterisation is a simple on-site test. In the U.K. the California Bearing Ratio (C.B.R.) Test is used almost exclusively by engineers for soil strength classification, although the Plate Load Test is used in some countries. It is unlikely that this approach will change significantly in the near future.

Two characteristics have to be established if the subgrade is to be modeled accurately :-

(i) resilient, or elastic, behaviour

(ii) permanent, or plastic, behaviour

These are generally regarded as the relationships between applied stress and recoverable strain, and permanent strain and number of load applications, or design life.)

The critical parameter to both these limiting strains is the elastic modulus. Thus, if an analytical design method is to be used, then the subgrade stiffness or modulus must be quantified accurately; this is usually defined as the resilient modulus, Mr, where :-

$$Mr = \frac{\text{deviator stress}}{\text{resilient strain}}$$

However, the behaviour of a soil is also dependant on many other parameters :-

stress history of subgrade (e.g. over-consolidation)

drainage conditions

pore water pressure

moisture content

magnitude of applied stress

rate of loading

shape of pulse loading

This list is far from exhaustive.

The subgrade strength is usually estimated by a simple standard test and the results correlated to an empirical design method. The C.B.R. penetration test is used in the U.K., whilst the Plate Load Test is prefered in America. Current U.K. design recommendations for highway pavements, Ref. 3.1, gives a table of typical subgrade strengths for a range of soil types; although in some areas a single CBR value is adopted for all highways falling within a statutory boundary. It is mainly because of these simplifications that detailed characterisation of the subgrade is of only academic interest.

Since an elastic modulus is required for analytical design methods, the following expression has been widely adopted :-

$$Mr = 10 \times C.B.R. \quad N/mm2$$

This is a purely empirical relationship. The CBR test measures strains at very large deformations, whereas the loading within the pavement structure consists of very small deformations under a hyperbolic pulse loading cycle, Figure 3.1. However, this expression is adopted by the simpler design methods and is, in the absence of more detailed information, fairly reliable; although for plastic soils a slightly higher modulus than calculated is more appropriate (nearer the upper limit of a

Figure 3.1 Variation of subgrade stress under moving load

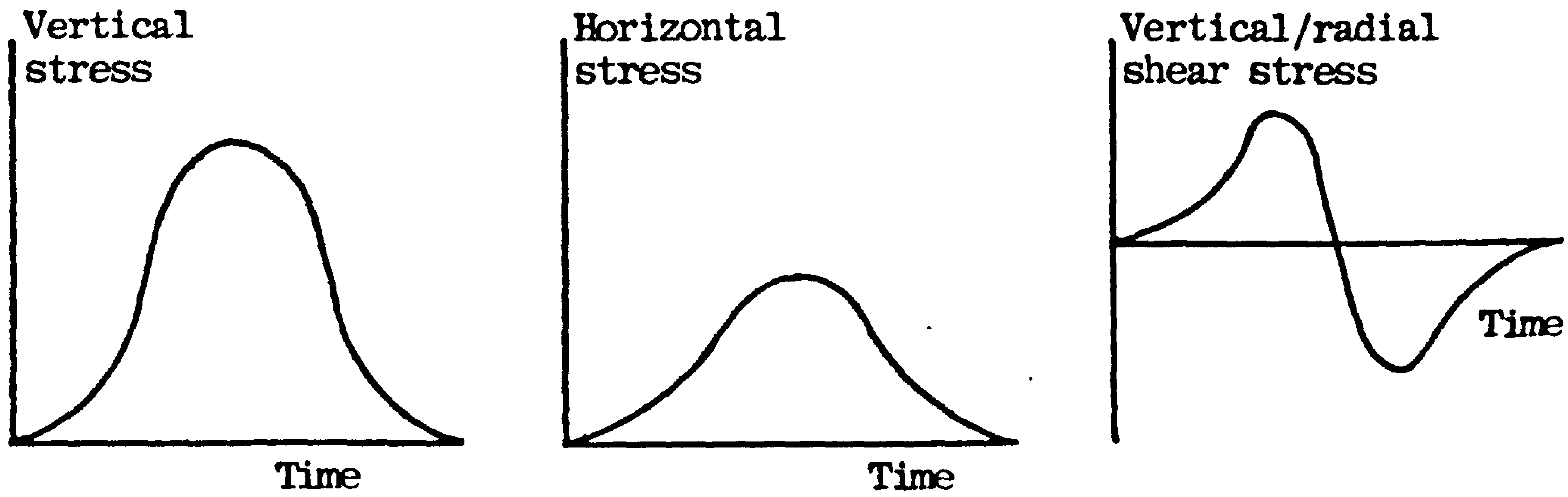


Figure 3.2 Stresses in triaxial test specimen

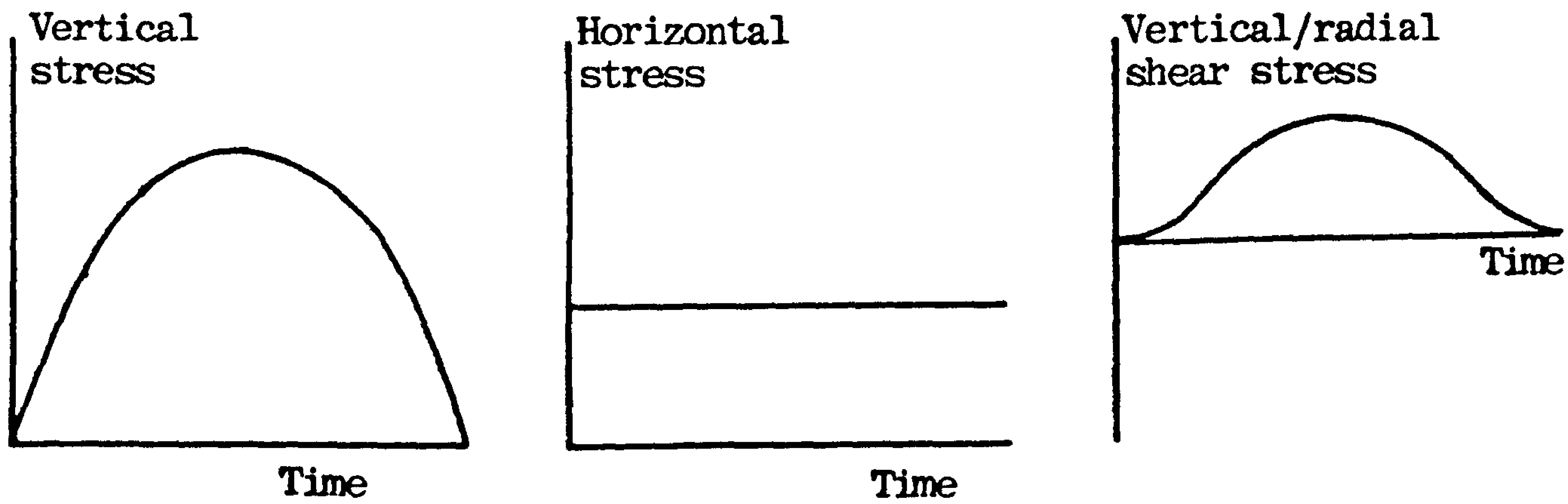
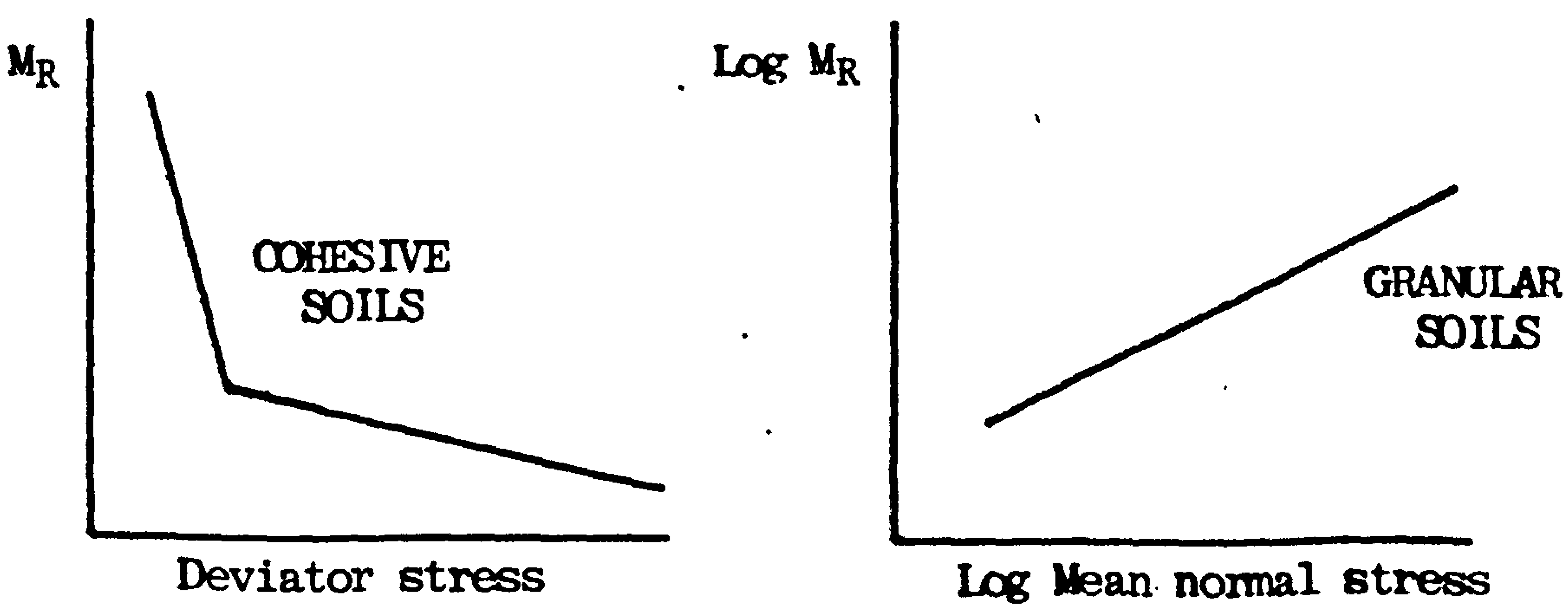


Figure 3.3 Simplified relationship between resilient modulus and applied stress



normal scatter band of two), Ref. 3.2.

One method of estimating the resilient modulus is by use of dynamic deflection or wave propogation measurements. These tests have the advantage that they can be carried out in-situ and with loads representative to real traffic. Another alternative is the laboratory repetitive triaxial test. This is a relatively simple extension to the normal triaxial test, making it possible to establish behaviour under repetitive loading. The applied stress pattern, Figure 3.2, approximates to the true stress pattern and, if the loads used in the test are representative of stresses expected within the pavement, a reasonable estimate can be made.

The resilient modulus is a function of the shear stress and the normal stress; shear stress is the predominate factor for cohesive soils whilst the normal stress applies to granular soils. Simplified models have been suggested (Refs. 3.3 and 3.4.) and these are shown in Figure 3.3. Clearly there is a unique solution for granular materials of the form :-

$$Mr = k \times e ** n$$

the constants can be found from the triaxial test. However, a granular subgrade material is, from the point of view of analysis, purely an extension of a granular sub-base material and will be discussed in more detail in the next section.

Some research work has shown that the non-linear elastic characteristics of granular materials and soils is important only where there is a significant structural granular layer. In highway pavements designed for heavy trafficking a substantial surfacing is specified and the exact subgrade modulus is not critical.

The second characteristic of the subgrade which is important to pavement design is the relationship between permanent deformation and number of load applications. Again there is a distinction between granular and cohesive soils :-

(i) granular material is characterised by a sharp increase in permanent deformation during the first few load applications, followed by little subsequent increase; i.e. after an initial compaction period the material tends to an approximate elastic behaviour.

(ii) for cohesive soils a definite failure point can be established where the deformation shows a sharp increase. The critical factor is the ratio of maximum applied deviator stress to undrained shear strength; the critical, or threshold, stress is approximately 70% of the "one shot" strength for a normally consolidated silty clay. However the stress failures are generally appreciably higher than the stresses found under normal highway pavements.

Despite significant differences in the behaviour of different subgrade soils, analysis of existing pavements showed a unique relationship between vertical subgrade strain and number of load repetitions to pavement failure. Dormon and Metcalf, Ref 3.5, analysed many pavements, particularly from the AASHO Road Test, and proposed the following relationship :-

Number of Repetitions	Vertical Subgrade Strain e
100 000	1 050
1 000 000	650
10 000 000	420
100 000 000	260

This straight line relationship has been widely adopted and was incorporated in the Shell Design Manual, 1963, Ref. 3.6. Edwards and Valkering carried out a more rigorous analysis and proposed the following relationship :-

e = 28000 x N ** (-0.25)

This relationship was substituted in the revised Shell Design Manual, Ref. 3.7.. Brown, Ref 3.8, carried out a similar analysis, with a particular emphasis on British design methods, and proposed the somewhat more conservative relationship :-

e = 21600 x N ** (-0.28)

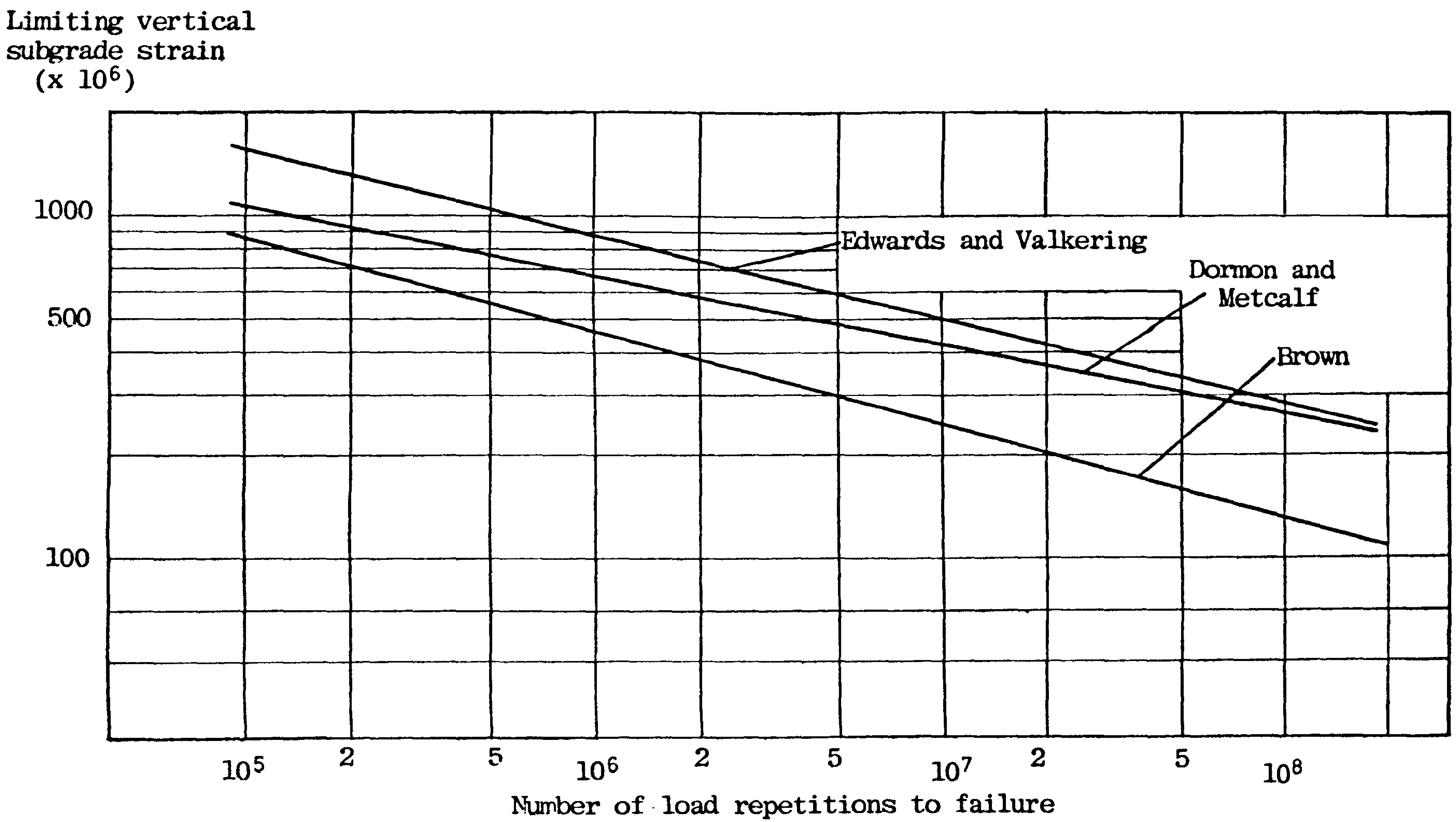
All three design curves, showing the allowable vertical subgrade strain, in microstrain, for a given number of load repetitions to failure, are shown in Figure 3.4 for comparison.

Finally, it is also necessary to have a value of Poisson's ratio for the subgrade. This parameter does not have a large effect on an analytical design method and the following typical values are usually adopted :-

Cohesive soils	0.4 - 0.5
Granular soils	0.2 - 0.35

No conclusive relationship between Poisson's ratio and subgrade type or applied stress has been established and since the overall effect on design is small, many design methods adopt a single value for all materials of 0.35 or 0.4.

Figure 3.4 Vertical subgrade strain design criterion



3.3. Granular Sub-base and Unbound Base Materials.

Unbound materials used in pavement construction have, similarly, received little characterisation research. It is generally agreed that an untreated granular layer will develop moderate strength when compacted to a high density, thus providing additional strength to the pavement at relatively low cost. However, a firm underlying layer, i.e. the subgrade, is necessary to achieve this degree of compaction. The strength of the granular layer is, therefore, a function of the strength of the underlying material and layer thickness.

It has already been shown that the elastic, or resilient, modulus of a granular material is a function of the applied stress, of the form :-

$$Mr = k \times p ** n$$

The decrease in resilient modulus with decreasing deviator stress does place an effective upper limit on the thickness of a granular layer. As the sub-base thickness increases, then its strength increases due to increased compaction, however the modulus of the base actually decreases due to the lower confinement stress. Thus, the two effects tend to cancel giving no real advantage for increased sub-base thickness. If extra strength is required, it is usually achieved by cement stabilisation of the subgrade or sub-base.

Research by Foster, Ref. 3.9, analysing the results from the AASHO Road Test, established the relationship between CBR strength and height above the subgrade, Figure 3.5 (a). All the results showed that the CBR of both the base and the sub-base increased considerably when trafficked and that it was lower nearer the subgrade.

Most simplified design methods rely on graphical presentation and thus require a reduction in the number of variables at any one time. This is usually achieved by expressing the elastic modulus of the granular layer as a direct function of the stiffness of the underlying layer and specifying a constant thickness throughout, typically 200 mm.

Expressing the strength as an elastic modulus, then we have the relationship :-

$$EM2 = k \times EM3$$

The modular ratio, k, across the interface is shown in Figure 3.5 (b) and, although the true value is a function of the subgrade CBR, it is possible to estimate an average ratio.

Figure 3.5 Strength of granular sub-base materials

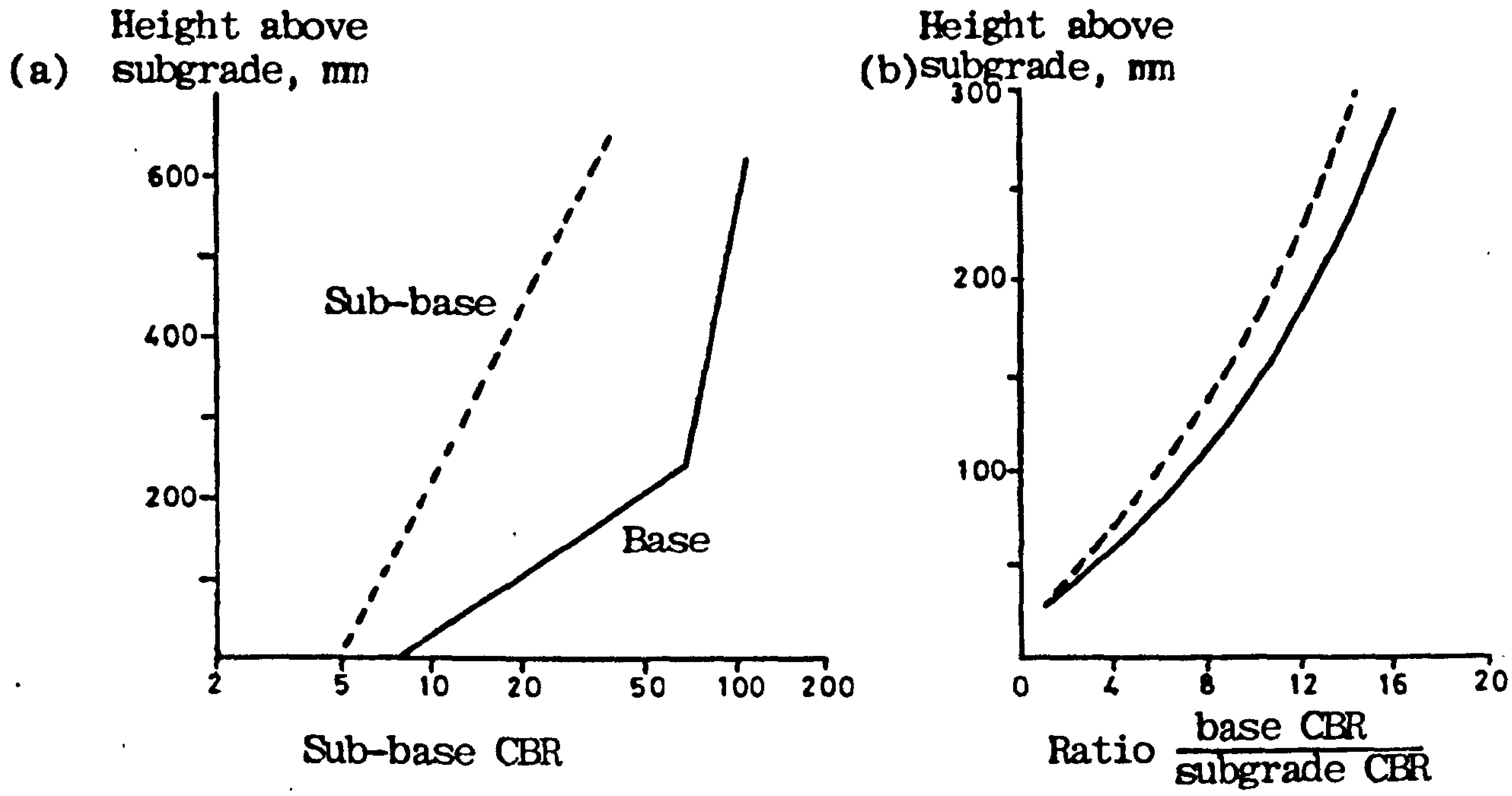
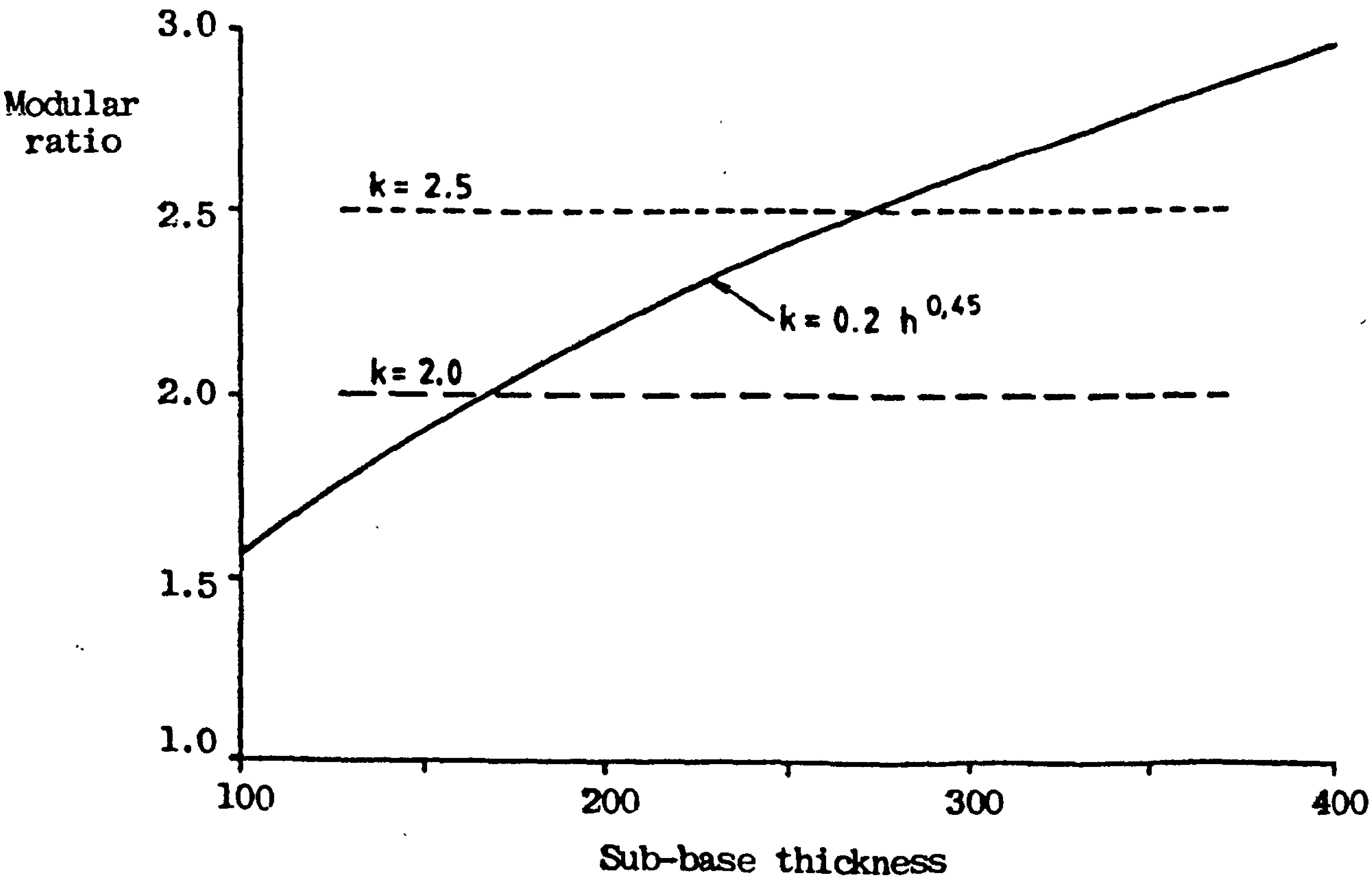


Figure 3.6 Simplified sub-base/subgrade modular ratio



Heukelom and Klomp, Ref. 3.10, established that a value of approximately 2.5 for k was appropriate for typical highway constructions A similar approach is used in the Shell Design Method, where k is found from the expression :-

$$k = 0.2 \times H ** 0.45 \qquad 2.0 < k < 4.0$$

where H is the thickness of the sub-base. This relationship is plotted in Figure 3.6. It has been shown that, for a granular layer built up in layers with a modular ratio of 1.5 to 2.5 between each, the maximum overall ratio is approximately 4.0 for realistic thicknesses, Ref. 3.8.

Also shown in Figure 3.6 are the modular ratios developed by Heukelom and Klomp (k = 2.5) and Brown (k = 2.0), Ref. 3.11. The modular ratio is largely independent of soil strength, i.e. soil type and seasonal variations, and grading of the granular material, e.g. type 1 or type 2 grading.

The simplified design procedure proposed by Brown, Ref. 3.11, assumes that the the sub-base to subgrade modular ratio is 2.0 and adopts a sub-base thickness of 200 mm for all structures. Comparing this with Figure 3.6 shows that this assumption is in reasonable agreement with the Shell Design

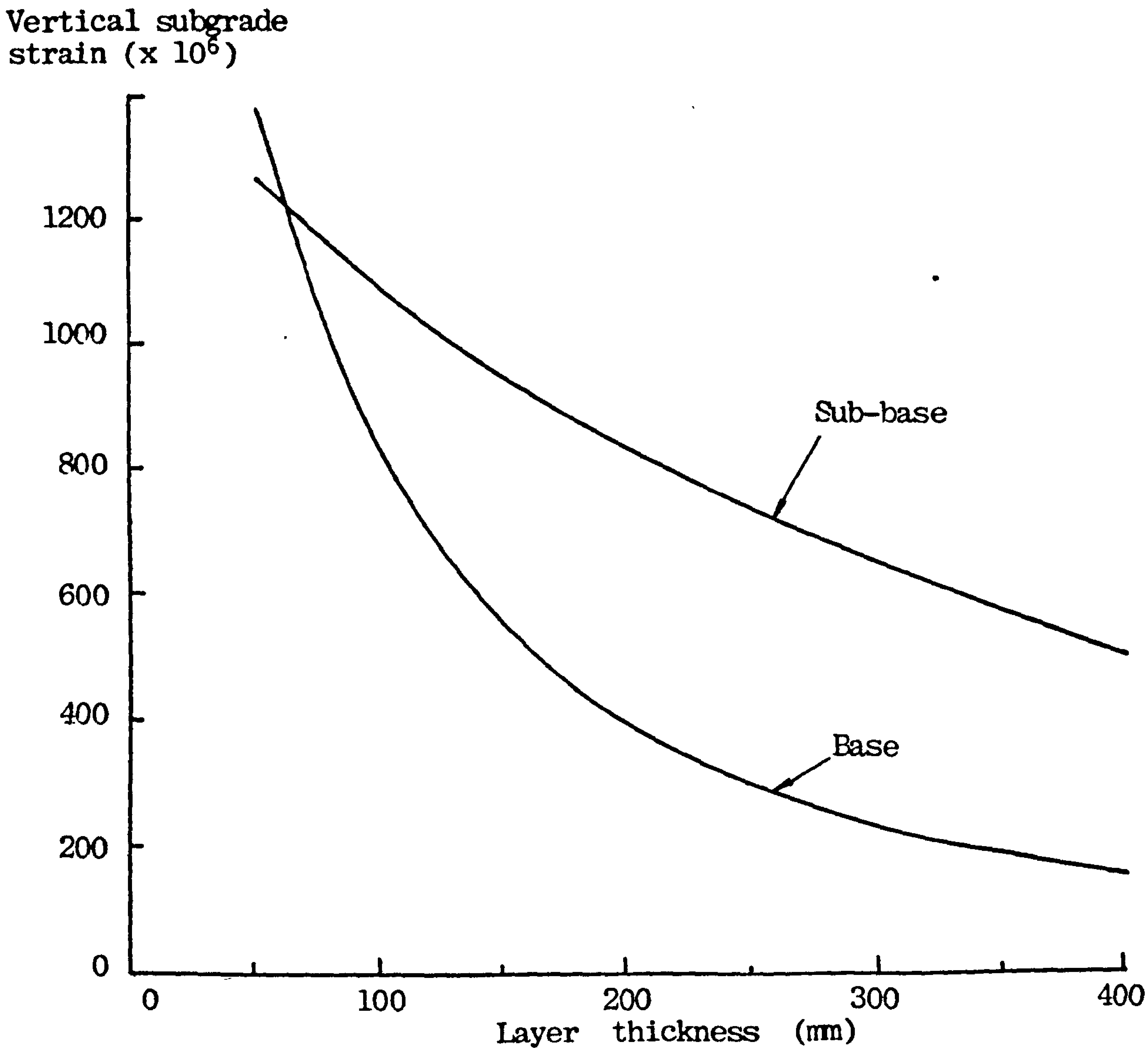
Method.

The use of thick granular layers in a pavement do not contribute appreciably to its strength. The effect of sub-base thickness and base thickness on vertical subgrade strain, the principal design criteria, is shown in Figure 3.7. Current U.K. sub-base specifications have been based on the results of pavement tests at Alconbury Hill, but long term surveillance shows no clear justification of the use of thick sub-bases. Foster is of the opinion that a 100 mm granular layer put down to provide a working surface for construction plant could hold water under the completed structure and hence reduce subgrade strength, thus questioning the provision of a granular layer in the pavement.

3.4. Cement Stabilised Materials.

The use of cement to enhance the strength of granular material or the subgrade was first used during the post-war rebuilding programme. However, despite its fairly common use it has only recently received the detailed research it deserves, Refs. 3.12 and 3.13. The material is basically defined as an aggregate of some kind, with low cement and low water contents added to give modest strength at low workability. The material

Figure 3.7 Variation of vertical subgrade strain against base and sub-base thickness



Wheel load 20 kN

Surfacing	Asphaltic h = 75 mm E = 6000 N/mm^2
Base	Asphaltic (h = 200) E = 4000
Sub-base	Granular (h = 200) E = 0.2 $h^{0.45}E_4$
Subgrade	CBR 3%; E_4 = 30 N/mm^2

can be transported, levelled and rolled very easily, although it requires at least 7 days to develop reasonable strength.

The current U.K. highway specification, Ref. 3.14, divides cement stabilisation into three broad groups depending on aggregate type and compressive strength of the stabilised material. The code also places an upper limit on the quantity of traffic the structure can carry. The three groups are :-

(i) Soil Cement - general fine grained soils, although a minimum liquid limit and plastic limit are specified. Cube strength at 7 days is approximately 3.5 MN/mm2. Usually applied to the subgrade and mixed in place. Cement contents vary, 10 % is typical. Not recommended for highways carrying more than 1.5 million standard axles (msa).

(ii) Cement Bound Granular Material (CBGM) - naturally occurring gravel/sand, usually batched as dug. Cube strength is of the same order as soil cement, but material variation is lower. Cement content approximately 7.5 %, maximum highway trafficking recommended is 5 msa.

(iii) Dry Lean Concrete - a concrete specification aggregate with a minimum 28 day cube strength of 11 MN/m2. Cement contents typically 5.55 %, (18 : 1 mix), and used for all classes of highway construction.

To facilitate mixing and rolling, minimum cement content should be approximately 4 % and maximum compaction is obtained if the mix is slightly wet of optimum moisture content, typically 7.5 %. This causes a slight increase in shrinkage cracking, but is no real problem. It should be noted that current U.K. codes require the cube strength to be measured at "field density" for soil cement and CBGM, hence the laboratory strength should be achieved in the field. However, lean mix concrete is tested when compacted "to refusal" and probably achieves only 75 % of this value in the pavement.

It is clear that this triangular division is crude and can be very misleading. The true properties of soil cement and CBGM are so dependant on the type of aggregate that there is an obvious necessity to base a design on true properties and not those of the above code. This is complicated by the different testing philosophies applied to the materials. Soil cement and CBGM are derived from soil mechanics test procedures; compressive testing of cylinders at 7 days, compacted to field density and moisture content. Lean concrete testing follows concrete specifications; compressive testing of cubes at 28 days after water curing with full compaction. This makes direct comparison between different materials very difficult, particularly since compressive strength has little correlation to the properties related to pavement design.

Cement stabilised material is, by definition, a rigid material and, as with asphalt, it is subject to tensile stresses at the bottom of the layer under wheel loading. The tensile stresses caused by heavy traffic would, therefore, cause extensive cracking in an underdesigned cementitious material.

A cemented material is also subject to stresses from shrinkage during curing and thermal stresses from the temperature gradient between the top and bottom of the layer. Shrinkage during curing is a normal characteristic and leads directly to transverse cracking across a highway pavement. However, both the cement content and the water contents are low thus shrinkage is not as pronounced as in structural concrete. Expansion joints are not provided and wide cracks must be avoided since they are reflected in a bituminous surfacing. To overcome this, specifications in the U.K. have tended to move towards thicker layers of moderately strong material to keep cracking as closely spaced and as fine as possible, typically between 2.5 and 6.0 metres.)

Stresses due to temperature warping are dependant on the temperature difference between the top and bottom of the layer and the coefficient of expansion. The latter is directly dependant on the aggregate type. These stresses are tensile at the bottom, i.e. additive to the stresses from trafficking, when the slab is cooling in the late afternoon. Further, the type of surfacing is important. A black top surfacing of less than 40 mm causes an increase in temperature gradient compared to fully rigid construction, i.e. structural concrete. However a 100 mm surfacing results in a significant reduction, this is usually the case under highway design recommendations. The stresses even in the stiffest of cement bound layers from temperature warping will rarely exceed 700 kN/m2 (0.7 N/mm2), which is of the same order of magnitude as the stresses under trafficking.

Cement bound bases, therefore, will crack soon after construction due to shrinkage. (For lean concrete, with a well graded aggregate, the cracks are fine and well interlocked, and do not affect structural integrity.) Test pavements have shown that the high potential life of a cemented base is realised only when the surfacing is thick enough to minimise thermal stresses. However, once cracking has become significant cementitious bases will deteriorate more rapidly than asphaltic bases. It should also be noted that although the subgrade and sub-base strength do not have a large influence on the stress distribution of an uncracked base, their strength is critical in determining the rate of deterioration after cracking.

The behaviour of a fine grained soil cement or a poorly graded CBGM is significantly weaker than a lean concrete or well graded aggregate mix. These materials tend to crack easily to give a closely interlocking granular structure with a strength, elastic modulus, similar to that of a true granular material, approximately 1/50 th of the uncracked material. The use of soil cement, therefore, is usually limited to improving the structural characteristics of the subgrade.

As the base ages the load carrying capacity without cracking increases as the hydration process progresses and the strength probably becomes greater than the maximum stress to be carried by the structure. This helps to reduce rapid deterioration as higher strength materials are better at withstanding abrasion at construction joints and cracks.

The elastic modulus is the parameter of direct interest to analytical design methods, there are several methods of determining this :-

(i) uni-axial compression

(ii) uni-axial tension

(iii) flexural bending

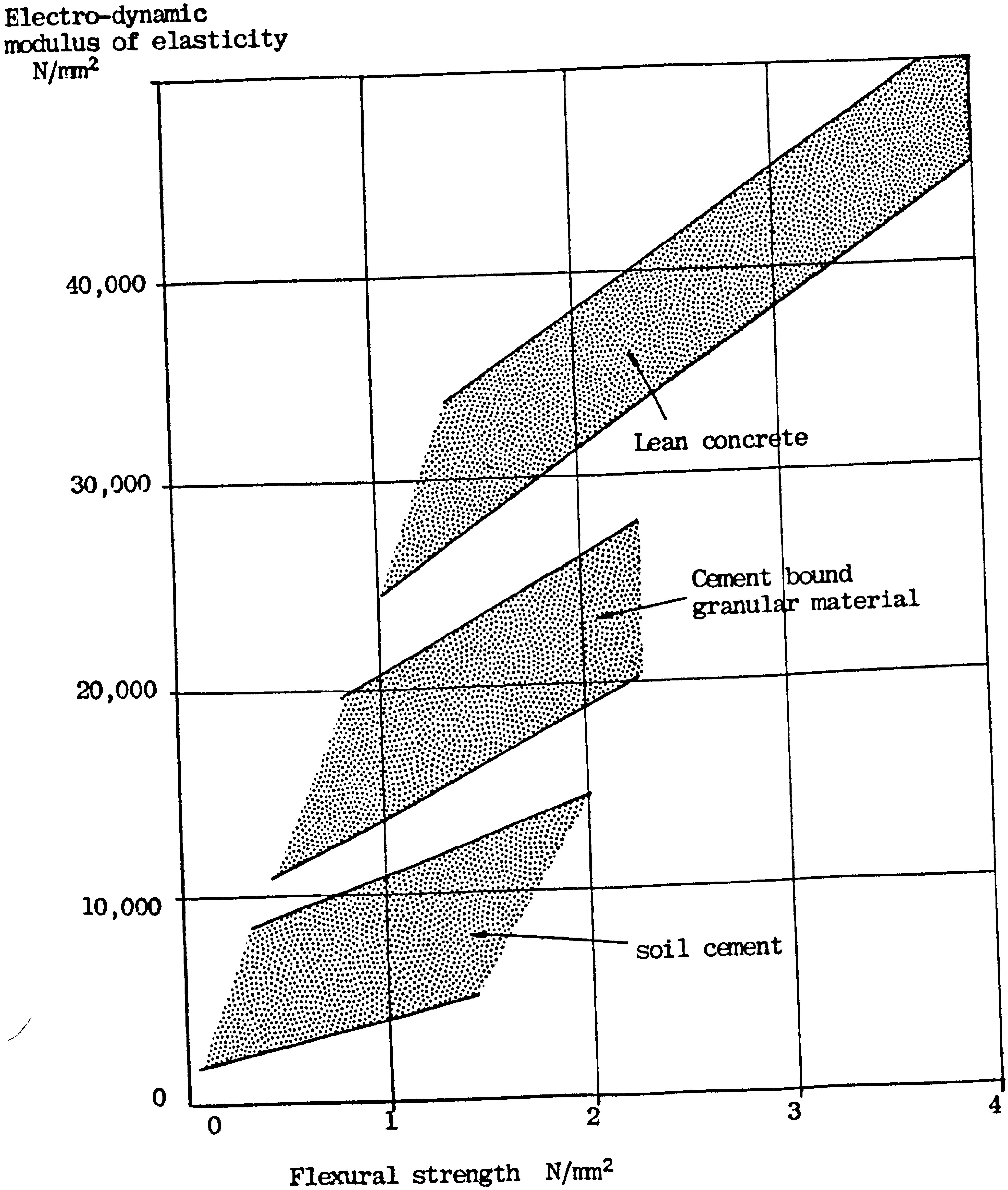
(iv) dynamic testing methods

Current codes specify cement bound materials by compressive strength alone, but this does not give a unique modulus. There is a well defined relationship between compressive strength and flexural strength, but the modulus of elasticity is dependant on the aggregate type,) Figure 3.8. Generally the nearer the aggregate is to a standard concreting aggregate, the higher the elastic modulus.

With dry lean concrete in compression there is a linear stress/strain relationship up to approximately half ultimate strength. The elastic modulus is usually calculated from the strain at a stress of one third the ultimate strength. However, in tension the material behaves elastically almost to the point of rupture and the modulus is calculated at a stress of half tensile strength. The tensile strength is typically one tenth the compressive strength and the elastic modulus is the same by both methods.)

The flexural strength is usually determined by a two point load test on a beam specimen. This is particularly appropriate since the bending action in the test resembles that occurring in the pavement. The flexural strength is typically 0.6 times the

Figure 3.8 Elastic moduli of cement bound materials against flexural strength



uni-axial tensile strength.

Resonant wave techniques can also be used to determine the elastic modulus, this is particularly useful for in-situ and non-destructive use. It is generally found that dynamic values are approximately 7/6 the values obtained by other methods, presumably due to the absence of creep characteristics.

Typical values of elastic moduli are given in Table 3.1, these are taken from the results of extensive testing by Kolias and Williams, Ref 3.15. Very little information is available on the fatigue characteristics of cement bound materials. Structural concrete shows a logarithmic decrease in life, i.e. number of stress repetitions, with an increase in applied flexural stress. Limited research by Symons, Ref. 3.16, has indicated a similar relationship for lean concrete mixes, although the results were not conclusive.

It is generally agreed that the yield stress, defined as the start of micro-cracking and hence the largest stress that can be sustained for a large number of repetitions, is between 60 and 90 % of ultimate flexural strength. Some work has suggested that the dynamic fatigue strength, i.e. under pulse loading conditions, is greater than the static fatigue strength obtained in the laboratory, and that the static flexural strength is a

Table 3.1 Typical elastic moduli for cement bound materials.

	Compressive Strength N/mm2	Flexural Strength N/mm2	Elastic Modulus N/mm2
Lean concrete	6	1.0	27,000
	12	2.0	35,000
	18	3.0	42,000
	24	4.0	48,000
Cement bound	3	0.5	15,000
granular material	6	1.0	17,000
	9	1.5	20,000
	12	2.0	22,000
Soil Cement	2.5	0.5	3- 7,000
	5.0	1.0	5-11,000

good approximation of the fatigue yield stress. However, Kolias and Williams gave the fatigue strength after 1,000,000 load cycles for uni-axial tensile loading as follows :-

Lean Concrete	60 %
Cement Bound Granular Material	55 %

'It has already been stated that fine grained soil cement materials crack readily under loading and cannot be considered as being capable of sustaining a tensile load, unless specific laboratory tests indicate otherwise.'

Research to establish a suitable Poisson's Ratio gives two values :-

Fine grained soil cement	0.33
CBGM and dry lean concrete	0.20

3.5. Asphaltic Materials.

The use of bitumen stabilised materials in pavement construction is well established and, as a result, a very large amount of detailed information is available on the characterisation of bituminous materials. The Proceedings of four conferences on The Structural Design of Asphaltic

Pavements, Refs. 3.17, 3.18, 3.19 and 3.20, contain, or refer to, much of this research. A summary of current knowledge on material characterisation has been presented by Brown, Ref. 3.21. Characterisation of asphaltic materials is divided into two groups, elastic behaviour for the analysis of the pavement model and plastic behaviour for the evaluation of permanent deformation. Detailed information on these can be found elsewhere, Refs. 3.22 and 3.23 respectively. In view of the wide use of asphaltic materials in industrial as well as highway pavements, a summary of material characteristics relevant to asphaltic pavement design is included here. However, in order to limit the size of the discussion, only the characteristics needed for two specific design methods are included.

The first design method, the Shell Design Method, Ref. 3.8, is a rigorous analytical approach based on the solution of an elastic multi-layer mathematical model. It includes the effects of seasonal air temperatures and temperature gradient through the pavement structure. The results are presented in graphical form, the large number of parameters requiring an extensive number of graphs. The second method, proposed by Brown, Refs. 3.9 and 3.12, uses the same analysis program, but the number of variables is reduced to give a much more simplified presentation.

An asphaltic mix exhibits both visco-elastic and visco-plastic characteristics. The stiffness, or elastic modulus, of the mix is dependant on many factors :-

(i) proportion of bitumen, aggregate and air voids of the mix.

(ii) stiffness of the bitumen binder

(iii) temperature of the mix.

(iv) duration of the applied load.

(v) magnitude of the applied stress.

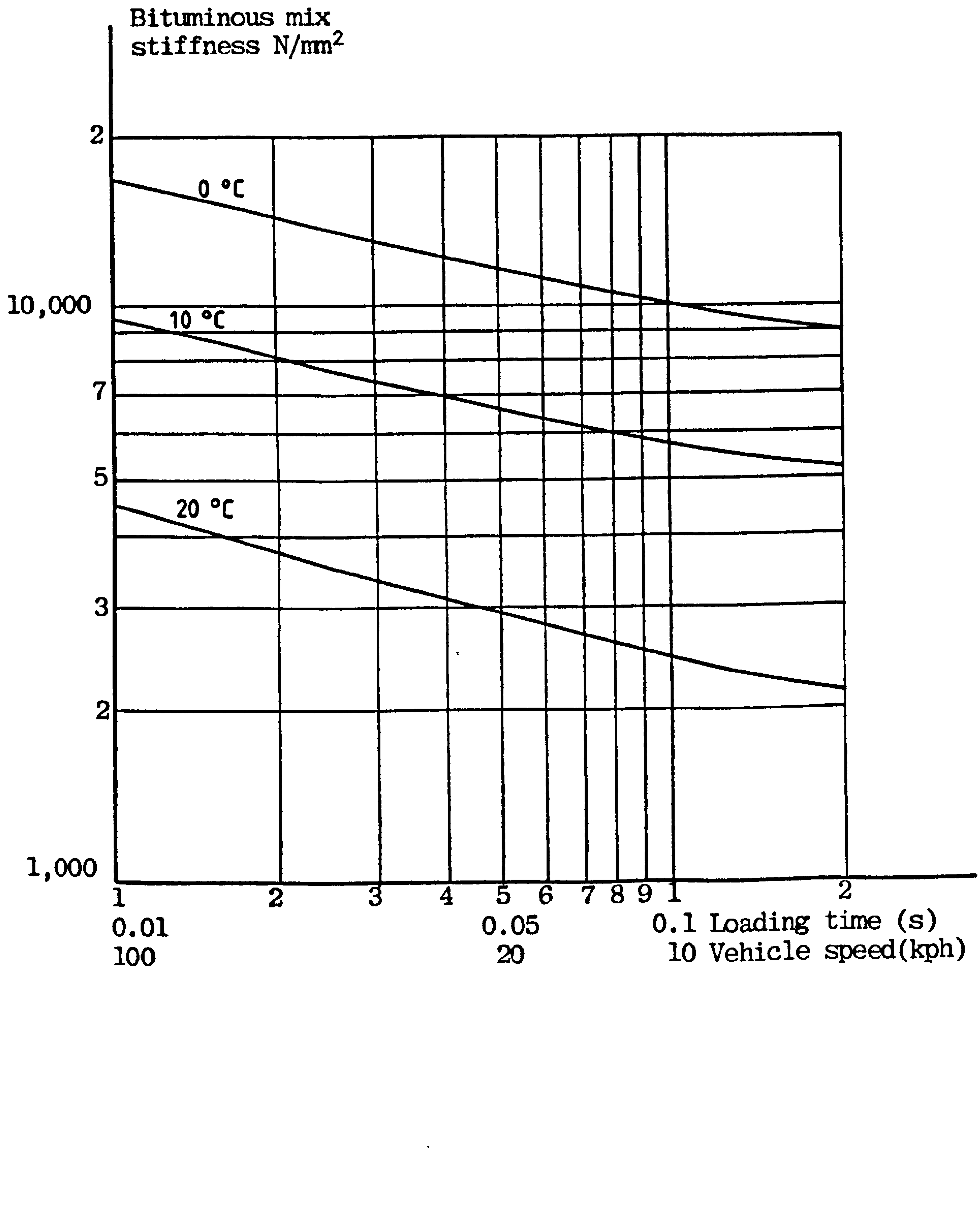
(vi) nature and grading of the aggregate.

(vii) the method of mixing and compaction on site.

The stiffness of an asphalt mix can vary between 10 and 50,000 N/mm2, although a typical lower limit is approximately 100 N/mm2. In the estimation of final mix stiffness only the parameters under (i) to (iv) are considered. The non-linear stress-strain relationship is generally regarded as negligible compared to the other variables. The effects of aggregate type and construction method cannot be established without tests on representative samples cut from trial pavements.

The stiffness of a bituminous mix is dependant on temperature and duration of the load, the effect of these parameters on a typical mix is shown in Figure 3.9. It is clear

Figure 3.9 Stiffness of bituminous mixes against temperature and loading time

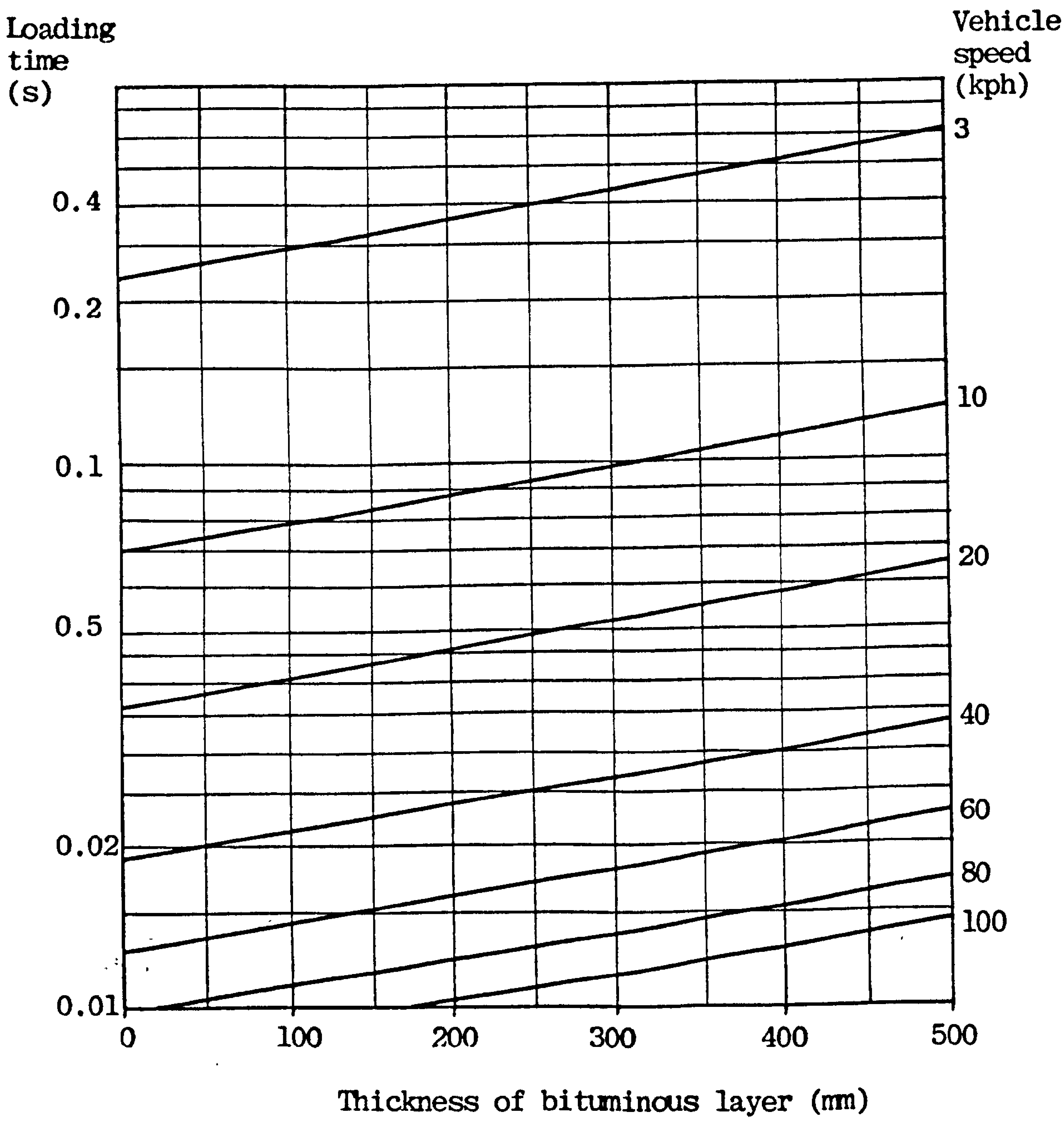


that the higher temperatures will have a disproportionate effect on the strength of the pavement, and it is necessary to weight these accordingly to give an effective mean annual temperaure within the pavement. This calculation is more critical in the prediction of rutting throughout a pavement's life and a detailed study of this can be found in the Shell Design Manual. For general pavement construction in the U.K., Brown suggests a mean annual air temperature 15 degs C.

The duration of the loading pulse also affects the effective stiffness of the bituminous mix. The duration of the loading cycle is a function of vehicle speed and asphaltic layer thickness. Brown, Figure 3.10, shows the relationship between these variables; as an approximation, loading time, in secs, is equal to the reciprocal of vehicle speed in km/hr.

A third variable is the Penetration Index, PI, of the bitumen binder. This is a measure of temperature susceptibility of the bitumen and is established from the results of penetration and softening point tests. The PI is usually found from a nomograph after Van der Poel, Ref. 3.24. It should be noted that "recovered bitumen" properties are be used in design calculations to allow for hardening during construction. As an approximation, the ~~PI~~ penetration JK of a bitumen after laying is assumed to be 65 % of that before laying.

Figure 3.10 Loading time against vehicle speed and layer thickness



Having established binder stiffness, mix stiffness can be determined from the mix proportions. There are several methods available, either by equation or nomograph calculation, and both design manuals contain the necessary information. Since it is unlikely that asphalt mix design will have been finalised at the preliminary design stage, Brown has included typical stiffness values for asphaltic materials commonly used in the U.K..

Asphaltic surfacing forms a high strength layer at the top of the pavement and, like cement stabilised materials, is prone to tensile stresses at the base of the layer as a wheel passes over the surface. However, unlike cement bound materials, tensile behaviour is neither elastic nor comparatively free from fatigue characteristics. The fatigue properties of bituminous materials have been summarised by Pell, Ref. 3.25, and the relevance of this failure criterion to U.K. conditions discussed by Brown and Peattie, Ref. 3.26. The number of load applications to initiate fatigue cracking is related to the radial tensile strain, e, as follows :-

N = k x e ** (-n)

where k and n are constants.

Estimates of fatigue life are based on laboratory experiments and do not include effects of "rest periods" between load applications. Thus, the crack propagation time, i.e. the time taken for the crack to propagate through to the surface, has not been allowed for and the true fatigue life is underestimated. Little information is available on this, but factors of 5 and 20 are appropriate for the effects of rest periods and crack propagation respectively, hence calculated fatigue lives are multiplied by a factor of 100. To aid preliminary design calculations, approximate fatigue curves for hot rolled asphalt and dense bitumen macadam (both 50 and 100 pen. bitumen) have been produced by Brown, where the correction factor of 100 has been included.

There are several methods of calculating the allowable strains in current use, both in equation and nomograph form. The equations adopted for the analysis programs here are given later in Chapter 7, Figure 7.2 (a).

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CHAPTER FOUR

PAVEMENT DESIGN AND ANALYSIS TECHNIQUES

Synopsis

This Chapter summarises the development of empirical and semi-empirical pavement design methods. The different design concepts for highway and airport pavements are compared and related to the design of heavy duty pavements. The application of the multi-layer elastic analysis model to current design requirements in the U.K., and its use as a tool to extrapolate existing design methods to include other types of pavement discussed.

4.1. Introduction.

Pavement design methods can be divided into three categories: theoretical, semi-empirical and empirical. Very few purely theoretical design methods have been proposed due to the appreciable difficulty of attaining satisfactory correlation between theoretical and true behaviour of the component materials. On the other hand, a large number of empirical design methods have been, and still are, used across the world. This is not only applicable to highway pavement construction, airport runways and taxi-ways follow similar design concepts despite significant differences in the type of loads carried by these structures.

Each empirical design method has been developed to suit a particular set of conditions, hence very different design thicknesses are obtained from different methods even when input factors are identical. Empirical design methods are severely restricted since they cannot be extended beyond the bounds of the constraints used in the initial development. However, to construct trial pavements every time new conditions are encountered is both time consuming and costly, some form of extrapolation is essential.

In recent years, a lot of research has been directed towards the application of theoretical analysis to existing empirical design methods, thus making extrapolation possible. There are many ways in which this has been approached, and not all can be covered here. The most successful has undoubtedly been the use of a multi-layer elastic model for pavement analysis. This Chapter outlines the development of this theory, and its application to current design requirements in Britain. The design of airport pavements is also discussed and the extension of the basic concepts to include other types of pavement is outlined.

4.2. Development of Current Design Codes.

The major constraint on the development of a satisfactory theoretical basis for pavement design has been a lack of detailed information on material characterisation. Although computational resources to accommodate the large number of calculations necessary for a rigorous analysis have been developed in the last 20 years or so, material characterisation has only recently reached a sufficiently advanced level; computational accuracy of current analysis techniques is, in some cases, still based on approximate empirical data.

There are two basic methods of approach to material characterisation; either the construction of a specific test pavement or the monitoring of an experimental and heavily trafficked highway. Both methods are expensive, requiring extensive laboratory back-up, the former allows controlled loading, thus results can be obtained more quickly, whilst the latter forms a perfect model allowing assessment of long term seasonal effects.

Of the numerous test pavements that have been constructed around the world, two of the best known were carried out in America in the 1950's :-

(i) W.A.S.H.O. Road Test (Western Association of State Highway Officials)

(ii) A.A.S.H.O. Road Test (American Association of State Highway Officials)

The principal aim of these tests was to examine the effects of various axle loads and wheel configurations on the performance of both flexible and rigid pavements. It was necessary to develop some form of monitoring to assess different pavement conditions and provide a basis for direct comparison of pavement performance. This was expressed in terms of "present servicability index", PSI, a concept based on the riding quality of the pavement. Although an empirical relationship was developed between PSI and the characteristics of rutting and cracking, it cannot be defined as a true structural parameter. These American tests have, however, yielded a lot of very detailed and useful information, much of which was included in an interim design guide published after the AASHO test. Brief summaries of these tests and their application to design methods can be found elsewhere, Refs. 4.1 and 4.2.

It is difficult to compare British and American design data directly since material specifications are different; for example, asphaltic concrete wearing course widely used in America has different characteristics to rolled asphalt mixes used in the U.K.. Britain also enjoys a much milder climate, frost penetration is relatively small making it economical to adopt thick granular sub-bases to prevent subgrade frost heave, in America loss of subgrade strength during the spring thaw is a principal design criteria.

One of the major conclusions of the AASHO road test was the development of equivalence damage factors for different axle loads. The tests showed that damage sustained by a pavement was dependant on the vehicle's axle load, but independant of pavement structure. The following relationship between pavement damage, D, and axle load, W, was established :-

$$D :: W ** 4$$

This relationship has been verified by results from other test pavements and studied in more detail by Heukelom and Klomp, Ref. 4.3, who proposed the following :-

$$D :: W ** 3.75 \times p ** 1.25$$

where p is tyre, or surface contact, pressure.

The use of either of these equations makes it possible to reduce a spectrum of axle loads carried by a pavement to an equvalent number of "standard" or "design unit" axle loads. This concept of traffic characterisation has been widely adopted throughout the world and the "standard axle", sa, now forms an almost universal design unit for highway pavement design. The standard axle is usually defined as follows :-

Axle load	80 kN
Tyre Pressure	0.500 MN/m2

Some of the more recent design methods have assumed this to be a twin tyre load, giving two 112 mm diameter contact areas 250 mm apart, although some methods use a single, circular contact area

160 mm in diameter.

The validity of assuming a circular contact area equal to the tyre pressure has been investigated. Feitag and Green, Ref. 4.4, found that under normal inflation conditions, the contact stress at the centre of the elliptical contact area is of a similar magnitude to the tyre pressure. However, in under inflated conditions, and to a certain extent when the wheel is overloaded, the tyre wall carries a disproportionate amount of load, resulting in high contact stresses around the perimeter of the contact area. Load distribution is also dependant on tyre stiffness, the higher the ply rating the less even the load distribution. Radial ply tyres are generally stiffer than cross ply tyres, giving a smaller contact area and a lower pressure differential under the tyre.

The effect of a simplified surface loading on stress distribution throughout the pavement has also been studied, Lister and Jones, Ref. 4.5. They showed that the errors in assuming a uniform circular loading with a contact pressure equal to the tyre pressure were less than 2 %. They also studied the effects of dynamic loading on the pavement, compared to the normally assumed static load. This proved to be much more difficult to assess since it was dependant on surface uneveness of the pavement. It was concluded that true dynamic loads may be up to 15 % greater than the static loads under extreme loading conditions for highway trafficking. This effect is usually neglected in current design methods.

Pavement research in the U.K. has been concentrated on long term study of heavily trafficked major roads. Several sites have been used over the years, but test areas on the A1 trunk road at Cannington and Alconbury are probably the best known. These tests cover both rigid and flexible pavement construction and have a fundamental guiding influence on the British pavement design code, Road Note 29, Ref. 4.6. However, the conclusions from these tests take several years to develop and periodic updating of the design code has been necessary. A summary of U.K. research, including more detailed references to particular tests can be found elsewhere, Refs. 4.2 and 4.7.

4.3. Current Empirical Highway Design Codes.

There are many empirical design codes in current use and not all of them can be covered here. American design codes have been reviewed extensively by Yoder and Witchzak, Ref 4.1, and a state of the art was published by Monismith and Finn in 1975, Ref. 4.8. The different design methods used in Europe were discussed by Peattie, 1977, Ref. 4.9, and details of several specific methods can be found in the proceedings of four conferences at Ann Arbor, 1962 to 1977, Refs. 4.10 to 4.13. The only empirical design method discussed here is the current U.K. design recommendations, Road Note 29, Ref. 4.6, details of other methods can be found in the above references.

The first major step towards a comprehensive design strategy in the U.K. was made in 1951 when a review of all methods then in use was published. This gave rise to two tentative design standards in 1955, Road Notes 19 and 20 for rigid and flexible pavement constructions respectively. The first edition of the current design recommendations, Road Note 29, was published in 1965 and covered both flexible and rigid construction. The recommendations given in these documents have been developed from the results of numerous pavement experiments throughout the U.K.. Road Note 29 was completely rewritten in 1970 and this second edition has been the basis for design over the past few years, although further experimental information has resulted in some further modifications, Refs. 4.14 and 4.15. These design recommendations were primarily developed for the design of motorways and other heavily trafficked roads such as trunk roads, although provisions were included to cover design of minor residential roads and culs-de-sac. Material specification normally complies with the Specification for Road and Bridge Works, 1976, Refs. 4.16 and 4.17. A more detailed analysis of the development and use of Road Note 29 can be found elsewhere, Leigh and Croney, Ref. 4.18.

Highway traffic is assessed in terms of the total number of standard axles the pavement is expected to carry throughout its design life. This is estimated from traffic surveys and forecasts, the total number of vehicles is then converted to "standard axles" by the application of a factor derived from the type of highway under consideration and the axle weight distribution for that road.

Recent experience of premature failure on heavily trafficked roads in the U.K. has brought about a dramatic increase in the average number of standard axles per commercial vehicle, Ref. 4.15. To account for future increases in traffic flow, growth rates of between 3 and 6 % are applied. In residential areas, where detailed traffic data is not relevant, typical design values are included in the recommendations. Design charts are provided to give required thicknesses of sub-base, road base and surfacing based on the cumulative number of standard axles and subgrade strength or CBR. The charts cover the majority of materials commonly used in U.K. pavement construction.

4.4. The Use of Elastic Theory in Pavement Design.

To bring a rational approach to pavement design it is necessary to develop a means of calculating stresses and strains within a pavement structure. For this, elastic mathematical models have been developed and the majority of research is based on this approach. Although pavement behaviour is not truely elastic, plastic behaviour is much more difficult to model, requiring much greater computation time, and, at the time of development of the analysis techniques, the relevant parameters were not known to a sufficient accuracy. The finite element theory provides a ready method of solution for plastic behaviour in a multi-layered system, however these have been developed only where sufficient computer resources have been available.

The first elastic mathematical model, proposed by Boussinesq, considered a point load over a semi-infinite homogeneous elastic material, integration giving the solution for a circular loading plate. Influence charts were published by Newmark, but these considered a rigid loading plate and were related more to foundation design and rigid pavements. Foster and Ahlvin, Ref. 4.19, produced a series of charts for the principal stresses and strains along the axis of loading. These were later extended to give the solution of stresses and strains at any point within a half-space for any value of Poissons ratio, Ref. 4.20.

However, a pavement is built up of distinct layers and the simplification to a single homogeneous half-space can only be regarded as a crude assumption. It is true that at large depths, greater than three times the loading radius, the effect of the surfacing diminishes and the single layer Boussinesq solution is a reasonable approximation, but at the base / sub-base interface the stiffness of the upper layers is significant.

Burmister, Ref. 4.21, published the solution to a two-layer elastic model, considering the pavement structure as the top layer. He also published the equations for the solution of a three layer model, thus making it possible to seperate bound and unbound layers within the pavement, Ref. 4.22. Fox, 1948 Ref. 4.23, produced the solution of Burmister's two layer equations in graphical and tabular form for a number of different parameters.

The first solutions for the three layer system were published by Acum and Fox, 1951 Ref. 4.24. These were subsequently expanded by Jones, 1962 Ref. 4.25, to cover a wider range of parameters. Interpolation between the various parameters was greatly simplified when these results were presented in graphical form, Peattie 1962 Ref. 4.26.

It is generally accepted that the three layer model is a reasonable approximation to the true pavement structure. Larger, and more rigorous models have been developed over the past few years which are capable of handling any number of layers and different loading conditions. These analyses have only been possible with development of powerful computing facilities, and it is not plausible to present the results from these programs in simple graphical or tabular format. However, despite the increased theoretical accuracy attained by increasing the number of layers in the model, current analysis is limited by the approximate material characterisation techniques usually adopted.

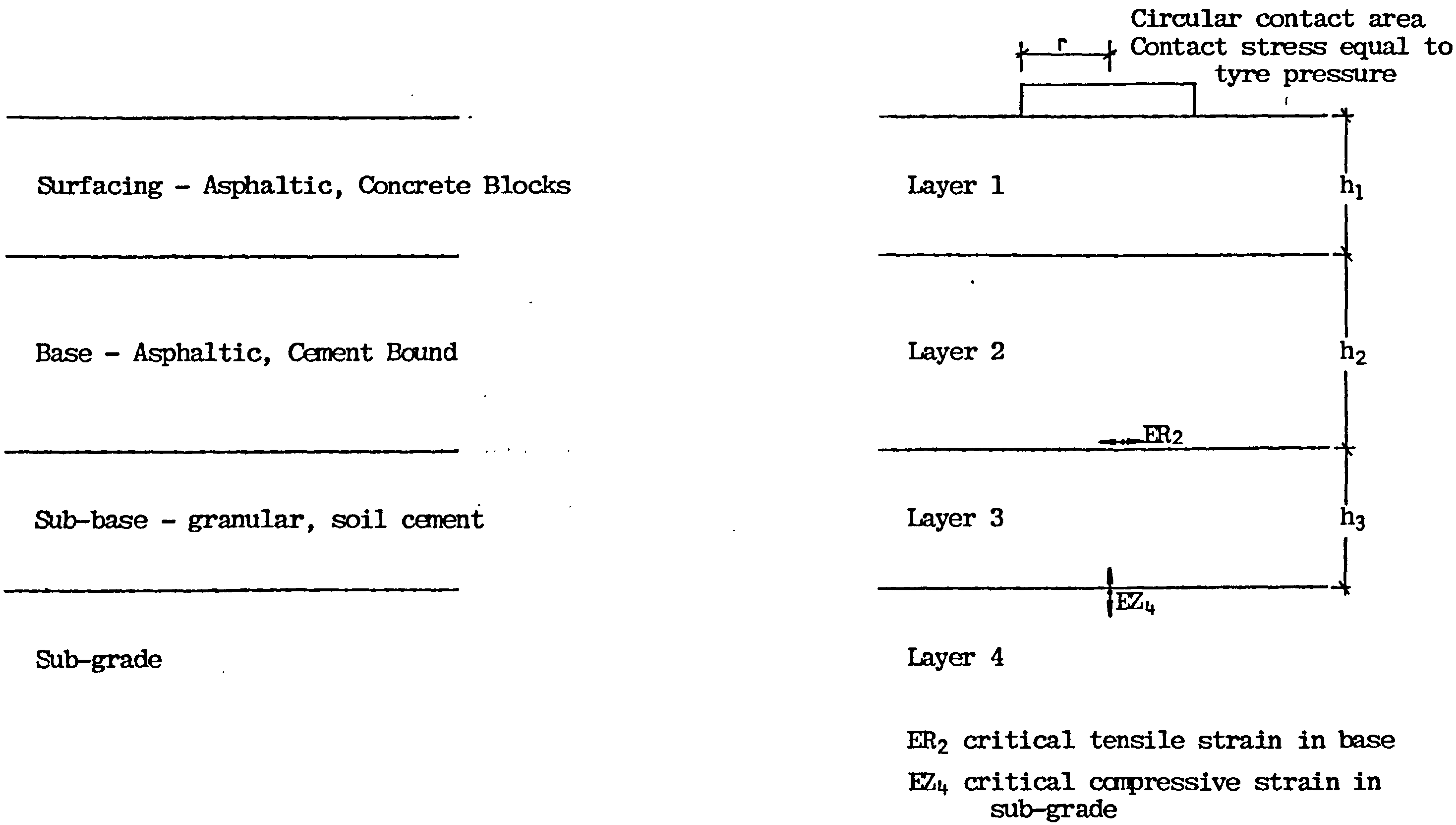
The solution of a three layer elastic model is not straight forward and large amounts of computer time are required. To overcome this it has been recognised that it is possible to apply the simple Boussinesq equations together with suitable correction factors. The correction factors can be obtained either from experimental tests or by comparison to the more rigorous analyses, both techniques have been tried and encouraging results obtained. This approach is particularly attractive since access to the rigorous multi-layer analysis programs is severely restricted.

Ullidtz and Peattie, 1979 Ref. 4.27, published a complete analysis method based on simple elastic solutions and a concept of equivalent thicknesses. Here stresses and strains are calculated at any point within the structure by reducing all layers above that point to a single layer of equal stiffness, but with an elastic modulus equal to that of the underlying material. The results obtained with this approach are within a few percent of those obtained from more rigorous solutions. Since this analysis method has been adopted for this research it will be described in more detail later in the text.

4.5. Highway Design Methods Based on the Multi-Layer Elastic Model.

Several complete design methods have been based on the multi-layer elastic model shown in Figure 4.1. Two of the largest are those developed by the Shell Petroleum Co. Ltd. and the American Federal Highway Administration. The latter, developed by the Department of Transportation of the F.H.A., uses the VESYS II M program; this was described in detail at the fourth Ann Arbor Conference, 1977 Ref. 4.28. The Shell Design Method is based on BISTRO and , more recently, BISAR programs. A series of design charts were first published in 1963 and this was extended and revised in 1978, Refs 4.29. This method has

Figure 4.1 Multi-layer pavement model



been developed to cover the conditions typically found within Europe and is, therefore, briefly described here.

As in the case of empirical CBR design methods, trafficking is assessed in terms of the standard axle using equivalence factors developed in the AASHO Road Test. Since the resilient properties of pavement materials are dependant on the length of the loading pulse, vehicle speed has to be specified; typically 50 kph or a loading time of 0.02 sec. The surface loading is assummed to be a twin tyred axle, represented by two 112 mm diameter cicular contact areas at a centre to centre spacing of 250 mm.

All pavement materials are assumed to be homogeneous, isotropic with infinite horizontal dimensions. Since the stiffness of bituminous materials are severely affected by temperature change, a sub-system of temperature weighting has been developed to give a realistic mean annual asphalt temperature. The analysis program, BISTRO, gives all stresses, strains and displacements for any point within the structure. For a satisfactory design two criteria have to be met :-

1) vertical compressive strain at the sub-base/subgrade interface.

2) horizontal tensile strain in the bound layer; this is generally at the bottom of the layer.

The magnitudes of these two critical strains is dependant on material properties and were discussed in the previous Chapter.

This is obviously an extremely rigorous approach and although an extensive set of design charts have been published, a simplified presentation has been put forward for use in the U.K. by Brown, Refs. 4.30, and 4.31. The following simplifications being made :-

(i) granular sub-base specified at 200 mm

(ii) stiffness of sub-base is twice subgrade stiffness.

(iii) all bituminous bound layers considered as a single layer.

(iv) twin tyred loading condition, with a contact pressure of 0.5 N/mm2.

A series of charts were presented giving the two critical strains for various combinations of subgrade strength and stiffness and thickness of the asphaltic layer. In order to make complete design possible, fatigue characteristics of the bituminous mixes typically used in U.K. highway construction were also presented; i.e. hot rolled asphalt and both 100 and 200 penetration dense bitumen macadam.

The structural integrity of a pavement is of prime importance but it is not the only criterion that should be considered. The point of failure of a particular pavement is somewhat arbitrary, dependent on the actual use of the pavement as well as material characteristics. Two obvious signs of distress are cracking and rutting and, since they are relatively easy to assess in real terms, they are often applied to faliure determination. Cracking only manifests itself when the cracks have propagated through to the surface,thus visual assessment of cracking is often too late for preventive maintenance of the structure.

There are two causes of rutting, compaction of material layers and plastic deformation, particularly in asphaltic layers. Since rutting is such a critical part of the pavement behaviour, rut depth prediction is a logical extension to a comprehensive design procedure. However, plastic behaviour is very dependant on external factors and an accurate assessment is very complicated, but a prediction method has been included in the current Shell Design Manual.

The advantage of these comprehensive design methods is that they can be readily extended to cover other types of pavement, which is a major drawback to empirical design methods. Airport pavements are an obvious example and it will be shown shortly that the Shell Method has been extended to this area. These analytical methods have also been applied to heavy duty industrial pavements, particularly where straddle carriers operate. However, conditions are so variable between different sites that simple classification, such as that used in highway design, is not possible. Further, because of commercial restrictions on the use of these large programs, very little specific information is available. Although there are very few similarities between the requirements of highway and airport pavements, heavily loaded industrial pavements, such as those in port areas, fall somewhere between the two categories, hence a brief summary of airport design methods is now presented.

4.6. Airport Pavement Design.

Airport pavement design has, like highway design, been largely empirical. There are obvious and fundamental differences in the requirements of these two categories of pavement and different design concepts are applied, these are summarised in Figure 4.2.

The design method used for airport pavements in the U.K. was issued by the Department of the Environment in 1971, Ref. 4.32.

Figure 4.2 Difference in pavement design concepts.

	Highways	Airports
Design unit	Equivalent wheel load; 80 kN standard axle	Usually a critical Aircraft
Number of repetitions	Up to 10,000,000 or 100,000,000	Rarely above 100,000; typically 5,000 to 25,000
Range of loads	Up to legal axle weight limit	Increasing with aircraft size
Vehicle speed	Generally up to 80 km/h	Up to aircraft take-off and landing speeds
Pavement width	Standard widths specified; approx. 3.75 metres	Dependant on aircraft type
Channelling	Vehicles run in lanes	Wide distribution about centre-line
Variation in vehicle trackwidth	Very little	Gear configuration dependant on aircraft
Riding quality	Fair to good	Very good
Economic effect due to loss of service	Undesirable but not critical	Critical if second runway not available
Subgrade variation	Various soils usually encountered along construction length	Confined site, usually uniform soil conditions

This method is wholly empirical and divided into two parts :-

(i) Characterisation of loading.

Individual aircraft types are classified on a numerical scale, Load Classification Number (LCN), according to the severity of loading; this is dependant on undercarriage gear loads, gear configuration and tyre pressure. Charts are presented to make this classification possible, although the DoE also publish a booklet giving the LCN for aircraft in common use. Great precision is not justified here as the overall loading on the pavement is determined from the highest LCN group that is expected to use the pavement. The highest LCN is defined as the Load Classifcation Group (LCG), a numerical scale from I to VII.

(ii) Pavement Design.

Three types of construction are considered: rigid, un-reinforced concrete; composite, reinforced concrete with a flexible surfacing; and flexible. All types of construction have a sub-base of 100 mm of lean mix concrete. A design chart links directly between LCG and subgrade strength, classified as either "good" or "bad", to give minimum thickness of concrete based on a minimum flexural strength of 3.5 N/mm2.

Generally, composite construction is used for high speed areas and rigid construction for taxiways, runway ends etc.. Flexible construction is usually considered to be uneconomic for this type of pavement.

Several other design methods are in common use and a brief appraisal of American specifications can be found elsewhere, Ref. 4.1. These are also empirical design methods where loading is classified directly from the highest aircraft weight or gear load. An exception to this is the Asphalt Institute design method which considers the pavement as a multi-layer elastic system and design is based on limiting critical strains within the structure. However, the method is only applicable to a full depth asphalt construction, i.e. where all the layers above the subgrade are bituminous, and assumes the use of asphaltic concrete. It has, therefore, little application to other types of pavement.

The Shell Design Method has been extended to airport pavements and is presented as a design method in its own right, Ref. 4.33. The only difference between this approach and that used for highway pavements is the loading classification. The material characterisation is similar and, since there are no differences in material behaviour, the same critical strain criteria are used. Different climates are readily accommodated by a change in mean annual air temperature.

An aircraft under-carriage consists of several gears, each made up of a number of wheels, the number of wheels, gear positions, tyre pressure and load on each gear vary between aircraft. Further, there is a lateral distribution of the loading across the pavement, unlike the significant laning effect on highways. The loading analysis finds the point across the pavement where the combined loading from the different aircraft is a maximum. Strains within the pavement, both vertical subgrade strain and tensile horizontal fatigue strain, are calculated for each aircraft at a series of points across the pavement. For simplicity, remote gears are neglected whilst those within the area of influence are represented as a circular load with a contact pressure equal to the tyre pressure; individual tyres on the gear under consideration are treated as such. These strains are then converted to total number of load repetitions to failure assuming that only this type of aircraft was to use the pavement.

To find the cumulative effect of different aircraft, Miner's Rule is applied. The "damage factor" is calculated for each point across the pavement and for each aircraft; this factor is defined as the ratio of actual number of repetitions of a particular aircraft to the number causing pavement failure. The combined loading is found by summing damage factors for each point; the sum for the critical point across the pavement should be unity, or just below, for an economic design. Lateral wander of aircraft can be included simply by applying a histogram distribution about the centre-line of each wheel before summing.

This is obviously a relatively long iterative design procedure, where the wide variation in design factors make general graphical representation impracticable. It does, however, clearly demonstrate the relative ease with which analytical design procedures can be extended to different classes of pavement.

4.7. Other Developments in Analytical Pavement Design.

It is clear that a large amount of analytical research has been directed towards pavement design over the past few years, and is still continuing. This final section discusses two topics which are important aspects to overall design, but which have yet to be fully incorporated in current design methods. These are prediction of rutting in asphaltic constructions, and the influence of horizontal surface loading.

4.7.1. Pavement Rutting.

The measurement of rut depth provides a quick and easy method of determining a pavements integrity. In the U.K. the rut depth under a 2 metre straight edge is used to specify critical and failure conditions of the pavement, corresponding to rut depths of 15 and 20 mm respectively. The analytical prediction of rut depth at the design stage is, therefore, a more logical design criterion. However, advanced research on this is still at an early stage of development, and a sufficiently accurate correlation between theoretical and in-situ behaviour has yet to be obtained. At present, pavements are designed to limit strain criteria, then rutting is checked to be acceptable.

Measurements on test pavements show that after an initial settling period, a well designed asphaltic pavement will progressively deform by between 1 and 2 mm per annum. Of the total surface deformation 50 % is attributable to the subgrade and sub-base and 50% to the roadbase and surfacing. This is only an approximation and the proportions within these divisions vary with material quality. Only about 25 % of total deformation is due to compaction of the air voids, which usually takes place within the first year of trafficking, the rest is due to plastic deformation.

For unbound materials stress level and time of loading are the critical factors. From preliminary tests based on repetitive tri-axial test, Barksdale, 1972 Ref. 4.34, concluded that the cumulative plastic strain follows an approximate logarithmic relationship with the number of load repetitions. However :-

(a) with a low deviator stress, the rate of strain accumulation decreases with the number of repetitions.

(b) above a critical deviator stress, the plastic strain tends to increase with the number of repetitions.

(c) after a comparatively large number of repetitions there is a sudden increase in plastic strain.

Barksdale also showed that, at a constant number of load repetitions and constant confining pressure, axial strain of a laboratory specimen followed an hyperbolic relationship with deviator stress. Hence, by dividing the pavement into a number of sub-layers and calculating the axial strain in each sub-layer, the rut depth can be obtained by summation. This research is still in its early stages and, unfortunately, the materials tested conform to American specifications and are not directly applicable to the grading requirements in the U.K..

For asphaltic materials, deformation is assumed to be purely viscous and is thus dependant on only cumulative loading time. However, by assuming that all vehicles travel at the same speed and are evenly distributed over 24 hours, this reduces to a summation of the number of load repetitions. Asphalt viscosity is dependant on pavement temperature and a weighting system, in addition to that developed for structural design, must be applied to the annual temperature variation. This has been extensively research by van der Poel and is included in the Shell Design Manual, Ref. 4.29.

The temperature of the pavement follows both a daily and an annual cycle. To accommodate the temperature gradient through asphaltic layers, van der Poel found the following division of bituminous materials suitable :-

H (1.1) 0 to 40 mm below the surface.

H (1.2) 40 to 80 mm below the surface.

H (1,3) remaining thickness, 80 mm to base of asphalt.

Plastic strains are estimated from mix stiffness, but since this is stress dependant, an elastic analysis is required to find the average stress in each sublayer. Also, the number of axle passes and average stress within the sublayers have to be weighted against bitumen viscosity. Having done this, the plastic strain in each sublayer is calculated and summation gives total deformation in the asphalt.

4.7.2. Horizontal Surface Loading.

Horizontal loading conditions, for example under braking and sharp cornering, have not been considered in the design methods. However these loads are substantial and, although they have little influence on structural design criteria, they may provide an explanation for observed pavement damage.

Koole, 1979 Ref. 4.33, observed that, in some cases, pavement cracking had occurred in areas of severe aircraft turning on airport taxiways. He analysed the effect of this horizontal loading in combination with the more usual vertical loading, assuming that the horizontal force on the pavement surface was approximately 70 % of static vertical load. The strains were calculated with element orientation such that shear stresses on the planes of the element were zero. It was found that at normal temperatures there was a slight increase in strains in the surfacing, the effect at points lower in the pavement structure was negligible. However, when a low stiffness surfacing was considered, such as a warm sunny afternoon, these maximum strains increased appreciably and, for some points around the tyre contact area, exceeded the failure strain obtained from laboratory samples of similar stiffness.

Although this characterisation is far from conclusive and more research is needed, it does indicate that horizontal surface loading may be one explanation of surface cracking in asphalt.

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CHAPTER FIVE

CONCRETE BLOCKS

Synopsis

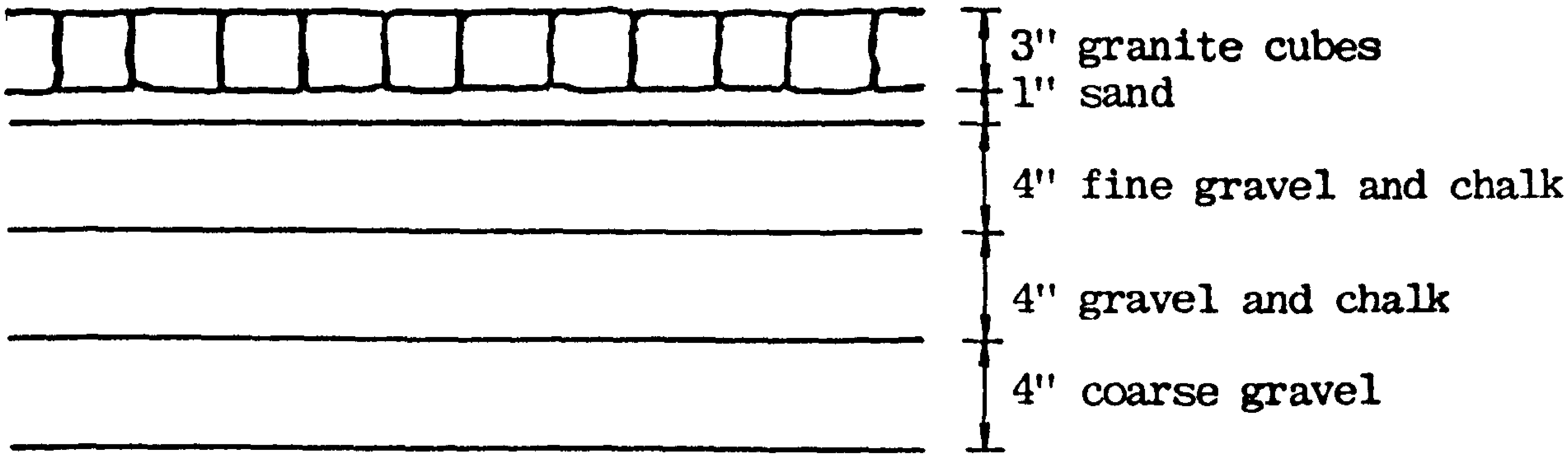
This Chapter reviews the development of block paving since its introduction into the U.K. in 1973. Current construction techniques and design methods are discussed, and research into characterisation of block surfacing summarised. Design recommendations for highway and heavy duty pavements are outlined and compared.

5.1. Introduction.

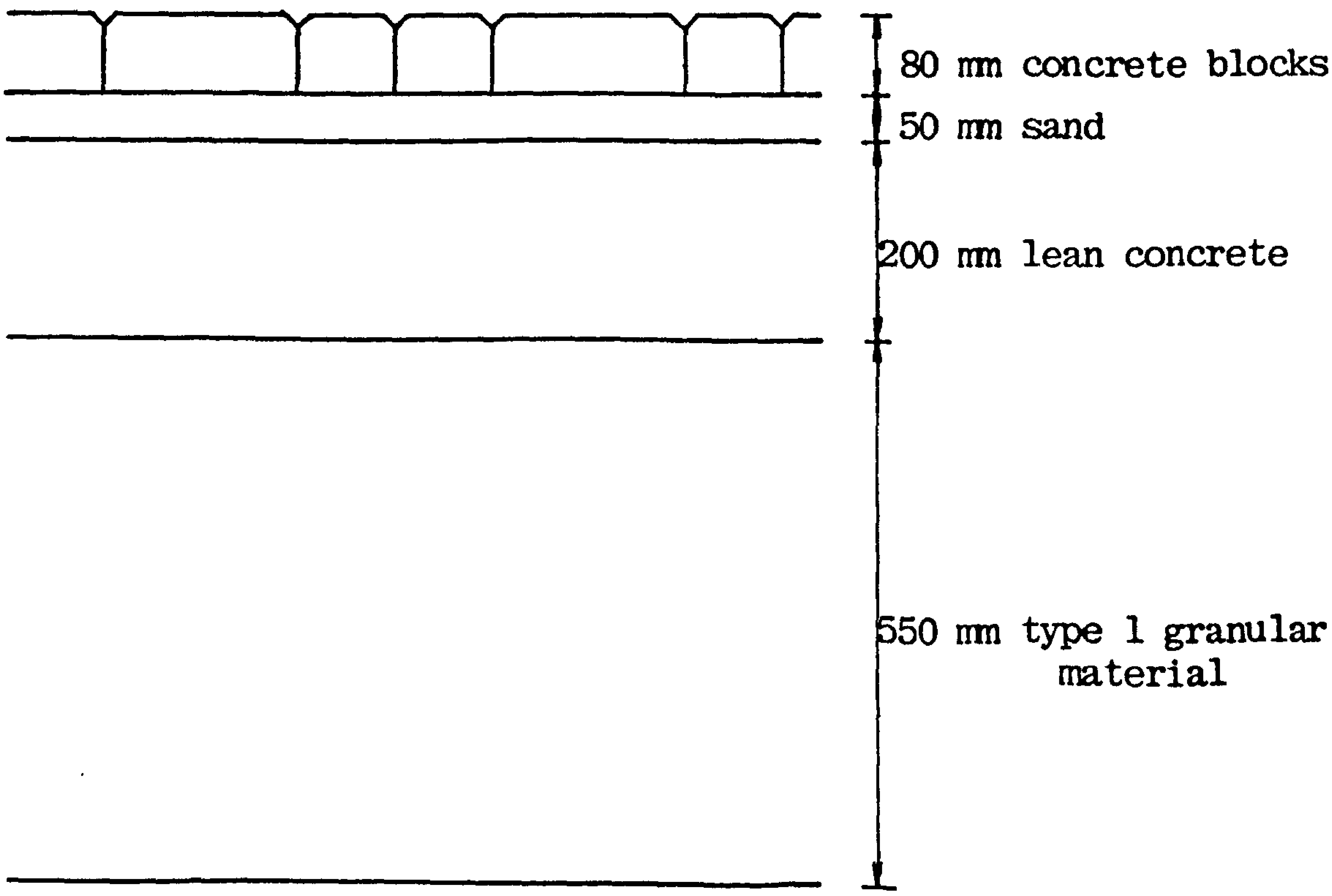
The use of "blocks" as a form of pavement surfacing is not a new one and many examples of 19th Century construction are still in use in the U.K., Ref. 5.1. The principal form of construction used to carry the steel wheeled traffic was made up of granite setts, approximately 9" deep, 8" to 15" long and 6" to 8" wide, laid on ashes or sand overlying the natural soil. This type of surfacing is still laid in the U.K., although its use is limited to pedestrian areas where an 'Olde Worlde' atmosphere is to be maintained. Unfortunately, the setts did not wear well, the edges fractured, and the stones became dome shaped and slippery. Despite the use of different types of stone, or the use of pitch soaked wooden blocks to reduce noise levels, overall performance was unsatisfactory.

These problems were overcome by William Taylor when, in 1843, he constructed a pavement for Euston Square railway terminus. The structure he adopted was a foundation of gravel, overlaid with gravel/chalk mixtures (naturally cementitious) to form the base material. The surfacing was made up of 3" granite cubes, bedded on 1" of sand, see Figure 5.1 (a). The whole surface was cambered and the joints filled with fine gravel. The structure performed satisfactorily, the stones did not wear as severly as setts and maintained their original level.

Figure 5.1 Block pavement constructions at Euston Square



(a) Taylor's construction, 1843



(b) Concrete block construction, 1979

In May 1979 a new bus terminal was opened in Euston Square with a similar form of pavement construction, Figure 5.1 (b). The foundation, or sub-base, was 550 mm of type 1 granular material, overlaid with 200 mm of lean concrete. The surfacing was made up of 80 mm thick rectangular concrete blocks, bedded on 50 mm of sand. The surface was laid to falls for drainage and the joints filled with dry sand. This structure is also performing satisfactorily, the surface is not deformed by heavy vibrating axle loads and, by use of different coloured blocks, the area is as attractive in appearance as the original Taylor structure.

Concrete paving blocks were introduced into the U.K. from western Europe in 1973 when it was proposed to use them for residential roads in Milton Keynes New Town development, 50 km north of London. This proposal coincided with research by the Cement and Concrete Association, C&CA, who were seeking to increase the use of concrete for minor roads. A research project was started and two surveys on the use of block pavements in western Europe were commissioned. The results were published in 1974 and 1975 by Knapton and Lilley, Refs. 5.2 and 5.3, and these showed that concrete block pavements constituted a considerable sector of the pavement surfacing market in West Germany, Belgium, Denmark and the Netherlands.

This Chapter traces the development of block paving in the U.K. from 1973 up to the present day, outlining the research and testing used to develop the existing design recommendations, Ref. 5.4.

5.2. The Blocks.

Since the first blocks were produced in the U.K. the number of manufacturers has increased from 4 to 10, with a total annual output of just over one million square metres in 1979. The blocks may be rectangular in shape, nominally 100 mm by 200 mm, but many manufacturers also offer proprietory shaped blocks. All units are "hand sized" and generally available in two thicknesses, 60 and 80 mm. The majority of blocks have a 5 mm champher around the wearing surface to reduce effects of spalling from high local stress concentrations.

The most important characteristic of the individual blocks is their compressive strength since this gives a general, but by no means unique, guide to durability. The method of strength testing and specification have received considerable attention and several procedures have been proposed. The first strength test specification, C&CA 1976 Ref. 5.5, required that blocks be compression tested to failure, air dry, with insulation board cushioning the surfaces. The maximum load sustained is divided by the area of the wearing surface; this is smaller than plan area if there is a champhered edge. From a series of ten compressive tests, the charateristic strength is calculated, this should not fall below 50 N/mm2.

This was superceded in 1978, C&CA Refs. 5.6 and 5.7, when the procedure was changed to a saturated testing condition, and a series of correction factors were introduced to accommodate effects of block thickness and surface champher. Also in 1978 a further specification was published by Interpave, the Interlocking Paving Association, Ref. 5.8, here the concept of characteristic strength was rejected in favour of mean, or average, minimum strengths.

Although these are the three principal specifications, others are used by purchasers of large numbers of blocks, e.g. Ref. 5.9. There is, however, little value in a detailed comparison of these methods since there is insufficient recorded information correlating crushing strength to the fundamental and critical characteristic of durability. The general requirement of a characteristic strength in excess of 50 N/mm2 does ensure good quality control in manufacture and ensure that the surfacing will not deteriorate unduly, it is, therefore, generally adopted.

5.3. Block Laying.

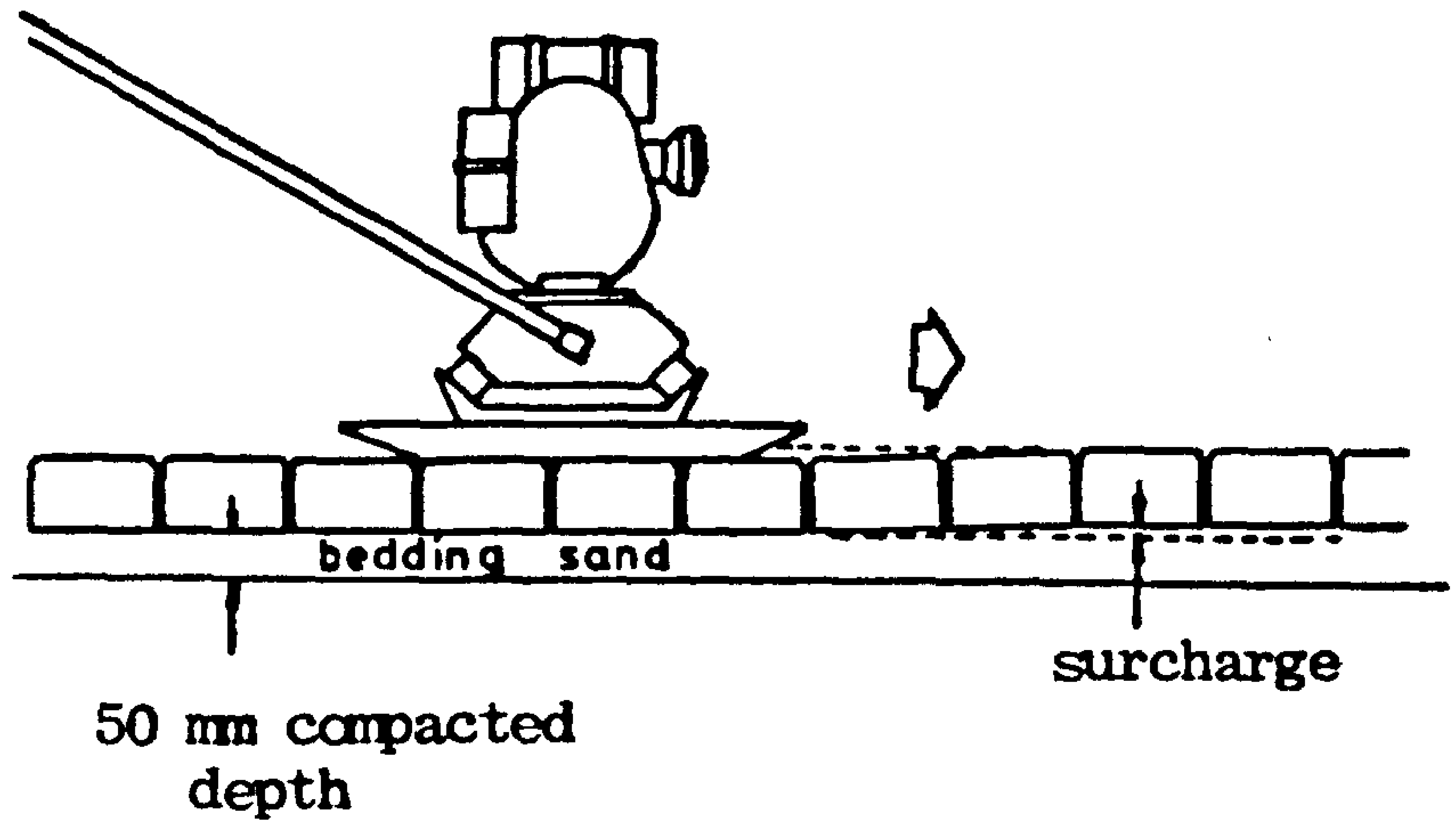
The blocks are laid by hand on a layer of screeded, uncompacted sand. This bedding sand should be sharp and well graded, zone 2 concreting sand is usually specified, and laid with a surcharge of 5 to 15 mm so that the final compacted thickness is approximately 50 mm, Ref. 5.10, Figure 5.2 (a).

After the blocks have been laid, and the edges filled, the surface is vibrated with a medium sized vibrating plate. This beds the blocks evenly, forcing the sand up into the joints to a height of approximately 25 mm. This is generally supplemented by a fine dry sand brushed and vibrated into the joints from above. The pavement is then ready for immediate trafficking.

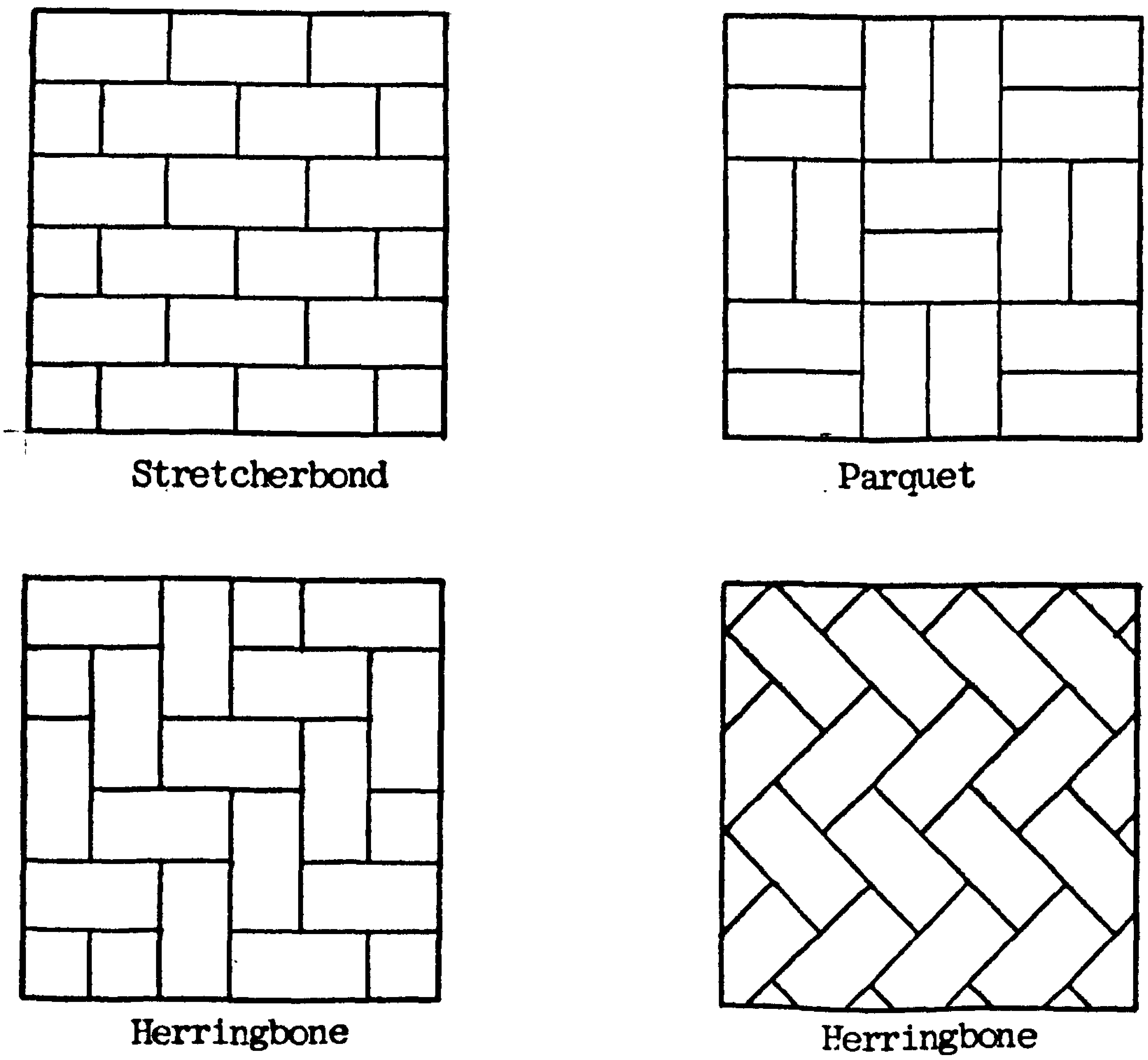
Concrete blocks are available in three thicknesses; 60, 80 and 100 mm. Research by the C&CA indicated that there was a significant increase in load spreading ability between 60 mm and 80 mm blocks, but a minimal increase when 100 mm thick blocks were tested, 1976 Ref. 5.11. This has been verified by further research and is reflected in the use of blocks; 60 mm blocks are considered suitable only for non-trafficked areas, whilst 80 mm

Figure 5.2 Construction of block surfacing

(a) Vibration



(b) Laying patterns



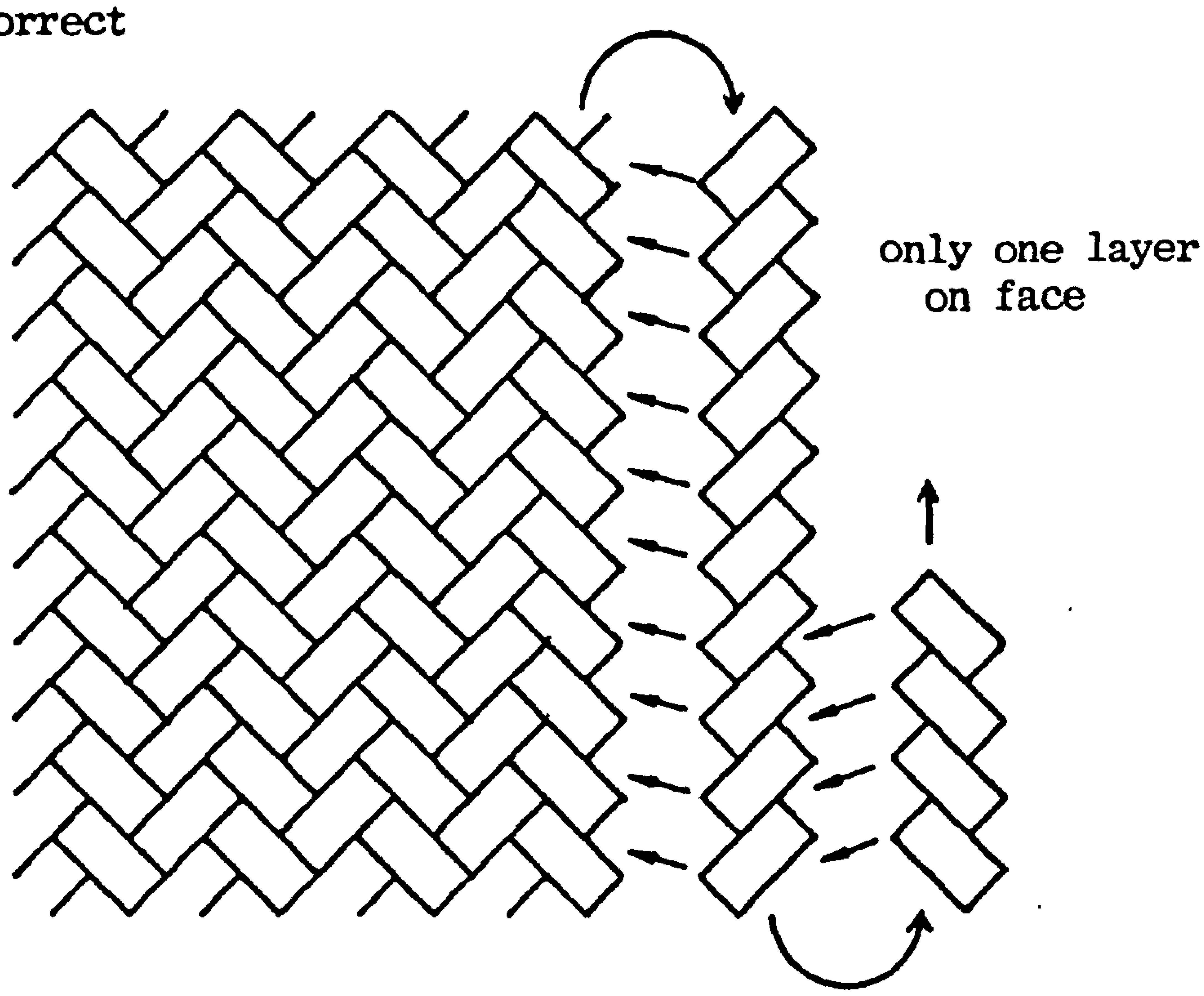
thick blocks are used for all other types of pavement. Some manufacturers recommend 100 mm thick blocks for heavy duty pavements, but very few structures have used them, it being more economical to increase the strength of the base.

The blocks can be laid in either a stretcherbond or a herringbone pattern, Figure 5.2 (b). It is generally accepted that the herringbone pattern should be used for rectangular blocks and for heavily trafficked pavements since it not only gives increased strength, but it also gives better resistance to horizontal braking and acceleration forces. If this pattern is used, with the line of blocks set either perpendicular to or at 45 degrees to the line of traffic, then there is no significant difference in the performance of rectangular and shaped blocks.

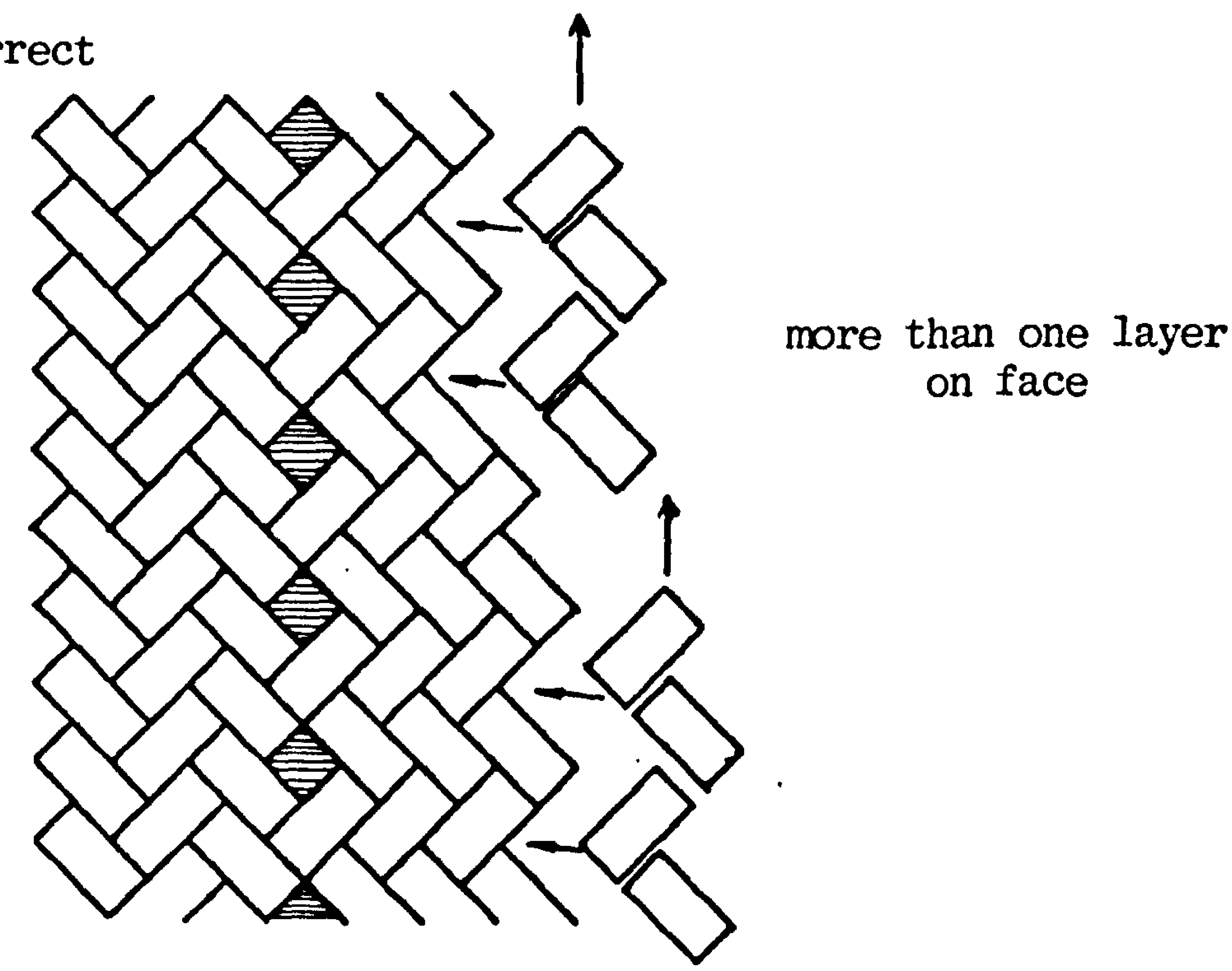
Block laying is obviously labour intensive, hence care is needed in setting out the correct pattern. It is possible to lay blocks two at a time, with several layers operating on the same face. At Ipswich, where a large area of rectangular blocks (20,000 m2) was laid in 1978, this was clearly demonstrated, Figure 5.3. The aim was to lay the area in two halves, opening each to traffic as soon as possible. Laying progressed with the main face moving from north to south, but it soon became apparent that the pattern was the wrong way round; the blocks were having to be constantly threaded into the pattern when more

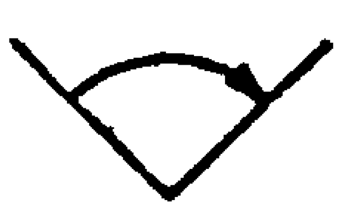
Figure 5.3 Laying method for herringbone pattern

(a) Incorrect



(b) Correct





JK

than one layer was operating, Figure 5.3 (a). To correct this a series of half blocks was inserted to reverse the pattern, Figure 5.3 (b), significantly improving the laying rate.

3 JK

Also, the contractor insisted on working to string lines to prevent the pattern from wandering slightly. This made it impossible to match up the second half of the area to the free edge from the first section. With hindsight it is easy to conclude that accuracy of this sort is not plausible on an area of this size, and eventually the pattern had to be broken and a straight construction joint made between the two areas. This joint has shown no sign of differental movement under trafficking and similar joints are being used increasingly to divide up large areas to ease laying.

5.4. Interlock.

With the joints between the blocks filled with compacted sand, no single block may move rotationally or translationally in isolation from its neighbours. This property of block surfacing is termed "interlock" and gives the necessary structural rigidity. The blocks are laid with joint widths of between zero and 6 mm. However, the vibration evens these out giving a typical joint width of 3 mm. Adequate edge restraint is important since interlock will be lost if the blocks are allowed to migrate horizontally, increasing the joints beyond 6 to 7 mm.

5.5. Water Penetration.

It is obvious that a newly laid block surfacing is not a waterproof covering for the supporting material. The joints will be filled with detritus in the early stages of trafficking which will not only decrease permeability, but will also increase interlock. Recent tests have shown that under heavy rainfall an appreciable quantity of water will penetrate to the lower layers, Ref. 5.12. Therefore, the sub-base and subgrade must not be susceptible to high moisture contents.

This problem was clearly demonstrated at the Port of Dover in 1976 when a trial area of 400 m2 was laid. The blocks were laid on 280 mm of compacted type 2 chalk granular material. There was a heavy rainstorm just prior to opening of the pavement and surface water percolated through the joints. The structure failed completely within minutes of it being trafficked. Subsequent tests on the sub-base confirmed that the moisture content was 8-9 % and the CBR only 3 %. The pavement was lifted and reconstructed with a cement bound sub-base, which has since performed satisfactorily. However, it was believed by the engineers that, subject to the provision of an adequate sub-base with a minimum CBR of 15 %, blocks were a suitable surfacing material for heavy freight vehicles on reclaimed land.

5.6. Uses and Advantages.

In the U.K., the aesthetically pleasing appearance of blocks led to their widespread use for pavements where a decorative finish is desired. They have been used for culs-de-sac, minor residential roads and shopping precincts. The use of different colours has been used to great effect in the layout of car parks and bus lay-bys, where lane differentiation was required.

Blocks have also been widely used for bus stations and lorry parks where high resistance to spilt oil and harsh tyre screwing is essential. The high flexibility of a segmental surfacing, allowing it to accommodate substantial settlement without structural damage, combined with its high strength was quickly recognised by users of heavy duty pavements. The advantages of concrete blocks over other forms of surfacing for heavy duty Port pavements was clearly outlined earlier in Chapter 2. There is little doubt that the ability to lift and relay a block surfacing to correct settlement over soft spots or to gain access to services is a valuable one.

There is an obvious caution in applying any new form of construction, but many people completely disbelieve any claim that a pavement made up of small, individual blocks of concrete will withstand trafficking. Unfortunately U.K. manufacturers have carried out minimal research in the development of design recommendations for the various types of pavement, and it has been left to outside research organisations to provide this information.

5.7. Design of Residential Roads.

Initially, design research was orientated towards an examination of the dissipation of a vertically applied load by concrete blocks. A test by Knapton, 1976 Ref. 5.11, on a small area of blocks of various thicknesses and shapes, indicated that block surfacing constitutes an integral pavement course having elastic properties analagous to those of conventional flexible paving materials. This initial conclusion led to recommendations for the design of residential roads, now almost universally adopted, based on the sub-base design curves of the current pavement design recommendations, Road Note 29, Refs. 5.13 and 5.14.

Compared to similar pavements designed to western European practice, pavements in the U.K. are frequently thicker and more accurately specified. This, combined with greater tolerance of blocks to surface deformation, is seen by many as being both wasteful of material and indicative of regressive thinking. However, there is no doubt that this caution has been valuable in the development of concrete block paving and that it has also ensured surprisingly few occurences of pavement "failure" due to deformation and disturbed drainage levels. Researchers have, therefore, turned their attention towards the greater problem of industrial pavement design, which is calculated to be the greatest application for block paving.

5.8. Concrete Block Research.

Very few large scale tests have been carried out on concrete block pavements. The many decades of design and constructional experience in Western Europe has yielded empirical design recommendations which are virtually impossible to apply to other countries, because of different subgrade conditions. This section briefly describes two field tests that have been carried out at Newcastle, both tests are described in full detail elsewhere, Refs. 5.15, 5.16 and 5.17.

5.8.1. The behaviour of a concrete block pavement.

In this experiment a small trial area of concrete block paving was constructed and tested to failure to determine its behaviour under very severe loading conditions with a weak subgrade. The true behaviour of the pavement was compared to a theoretical model using a three layer elastic analysis.

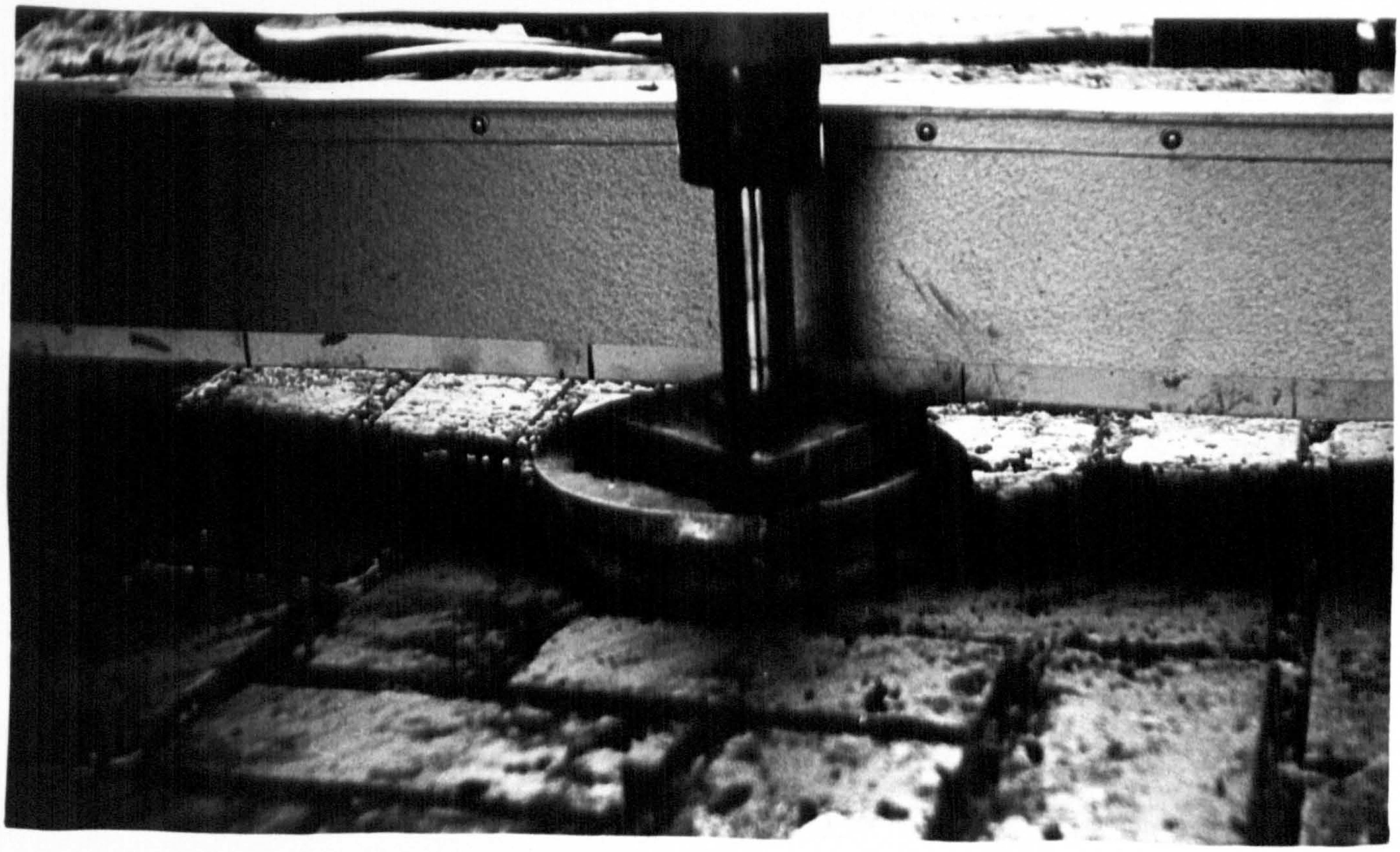
The test pavement, 6m long by 3m wide, was constructed on the centre-line of an existing asphaltic pavement excavated to a depth of 700 mm to expose a clay subgrade. Imported clay from an adjacent borrow pit was used to bring the subgrade up to the required profile. The sub-base was a type 2 granular material which, when compacted, tapered from 400 mm to 50 mm over a length of 4 metres. 80 mm thick rectangular concrete blocks were laid in a herringbone pattern in the normal manner, edge restraint being provided by precast concrete channels benched in lean concrete to prevent lateral movement.

Since it was not possible to apply any form of vehicular loading, a substitute method was used. A repetitive load of 6.4 tonnes was applied through a 200 mm diameter steel plate by an hydraulic jacking system clamped to the chassis of a heavy vehicle. The deformed profile of the pavement was recorded at regular intervals throughout each test by a straight edge profileometer to an accuracy of 0.25 mm. Seven tests were carried out with sub-base thicknesses of 50, 150, 200, 250, 300, 350 and 400 mm, and deformation profiles plotted to an exaggerated vertical scale. Loading was continued either until integrity of the pavement was destroyed or until there was an unacceptable disturbance of the adjacent testing point.

In the first test, where the sub-base was only 50 mm thick, failure occurred on the first load application, Figure 5.~~5~~4 JK (a). The area under and immediately around the loading plate was punched straight through the surfacing and into the subgrade. Three blocks were split in half with vertical shear ocurring along these breaks and along the joints, showing that interlock developed between adjacent blocks was greater than the shear strength of the concrete.

At the next testing point, where the sub-base was 150 mm thick, vertical deformation developed rapidly, but a similar failure did not occur. The deformation profiles showed that the area around the loading plate was depressed, but at greater radii the blocks arched upwards. This arching action gave the surfacing greater rigidity and improved its load spreading properties. There was slight shear along some joints around the plate and two or three blocks spalled at the corners where large stresses developed. Apart from this, structural integrity of the

Figure 5.4   Response of block surfacing to severe loading



surfacing was not affected despite the large deformations which developed.

Figure 5.~~5~~4 (b) shows the deformation at the final loading point with the load applied, the sub-base is 400 mm thick. Sufficient interlock had been developed to give a general depression with no vertical shear along joints. There was no trace of 'failure' of the surfacing, any criterion being an arbitrary limit on vertical deformation.

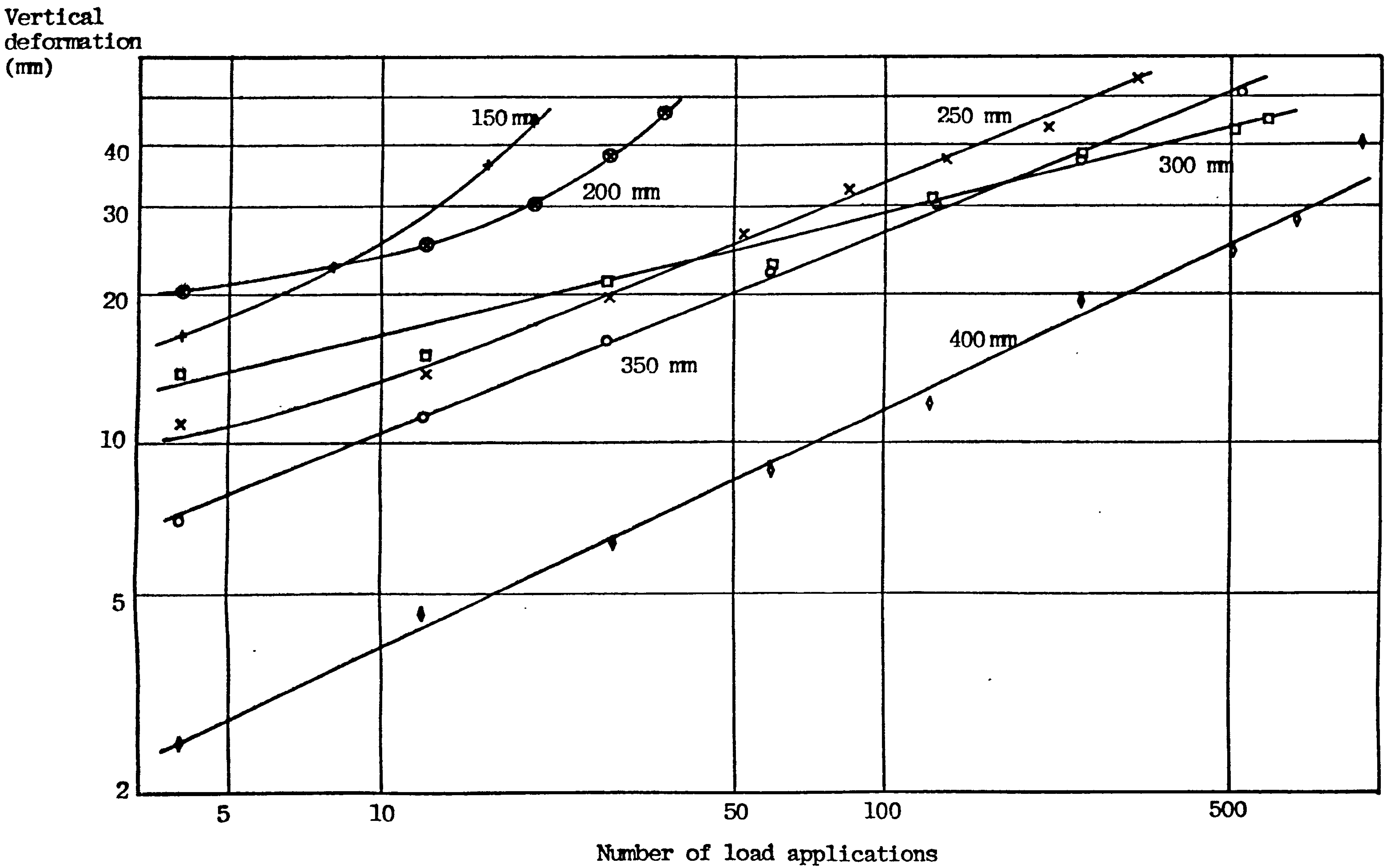
Progressive deformations for each test are plotted logarithmically against number of load repetitions in Figure 5.~~6~~5JK. It can be seen that the last four tests approximate to a series of parallel straight lines of the form :-

y :: N ** n

Linear regression for the last four tests gave an average value for n of 0.37. The constant of proportionality is dependant on the magnitude of the deformation after the initial loading cycles, but generally decreased with increasing sub-base thickness.

For a mathematical analysis, the loading was converted to an equivalent number of standard axles using the relationship

Figure 5.5 Progressive deformation in block surfacing



developed by Heukelom and Klomp, and given in Chapter 4.2. The pavement model applied is shown in Figure 5.6. Jones's tables, Ref. 5.18, were used to calculate vertical strain at the top of the subgrade under a standard axle loading condition, i.e. a 40 kN wheel load. This strain was related to estimated design life for the pavement with varying sub-base thicknesses using the relationship proposed by Brown, and given in Chapter 3.2 :-

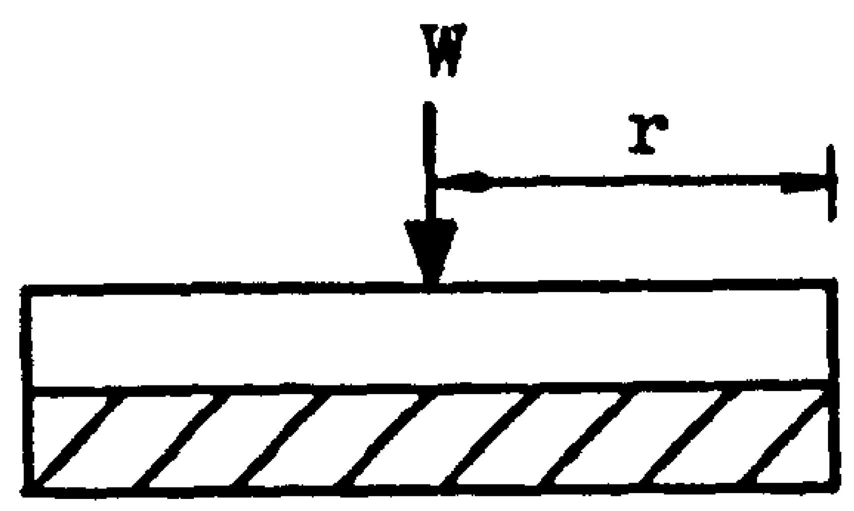
N = [21 600 / e] ** 3.571

The definition of 'pavement failure' is somewhat relative but, in the U.K., it is generally considered to be a 20 mm rut under a 2 m straight edge, this being equivalent to a surface deformation of 25 mm. The experimental and theoretical results are compared in Figure 5.7. This plot shows the design curve for a subgrade CBR of 2 % based on the structural analysis. The number of load applications required to give a 25 mm deformation at each loading point has been converted to standard axles and are also plotted. The design curves from Road Note 29 have also been added where design life exceeds 10,000 standard axles.

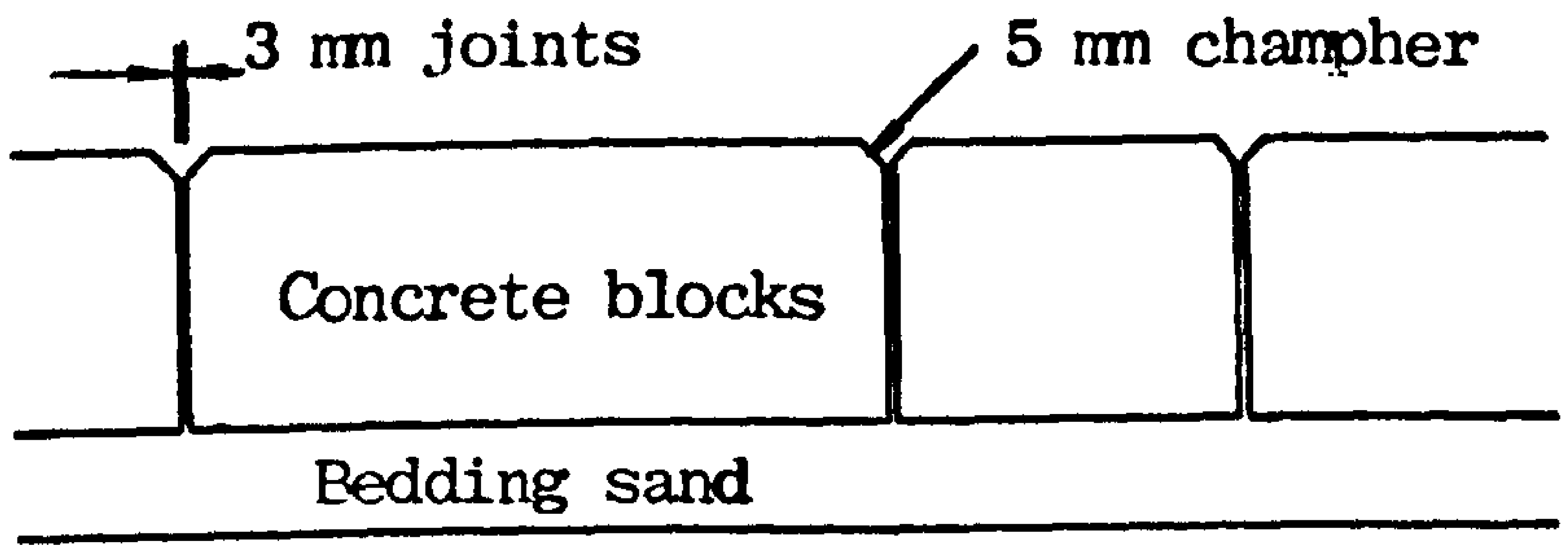
The following conclusions were drawn :-

(a) Provided concrete blocks are laid on a nominal sub-base to prevent immediate shear failure along

Figure 5.6 Model used in structural pavement analysis



Standard axle loading
$W = 40$ kN
$r = 160$ mm
$\sigma = 0.5$ N/mm^2



Layer 1
$h = 80$ mm
$E = 900$ N/mm^2

Granular sub-base

Layer 2
h = depth of sub-base
$E_2 = 2.0 \times E_3$

Subgrade CBR 2%

Layer 3
$E_3 = 20$ N/mm^2

$k_1 = E_1/E_2 = 18$

$k_2 = E_2/E_3 = 2.0$

h_2	$N=h_1/h_2$	$a=r/h_2$	$(\sigma z_2-\sigma r_3)$ [1]	tz_3 [2]	N [3]
200	0.40	0.80	0.170	4250	330
300	0.267	0.533	0.121	3025	1120
400	0.20	0.40	0.0915	2288	3030
500	0.16	0.32	0.0675	1688	8980

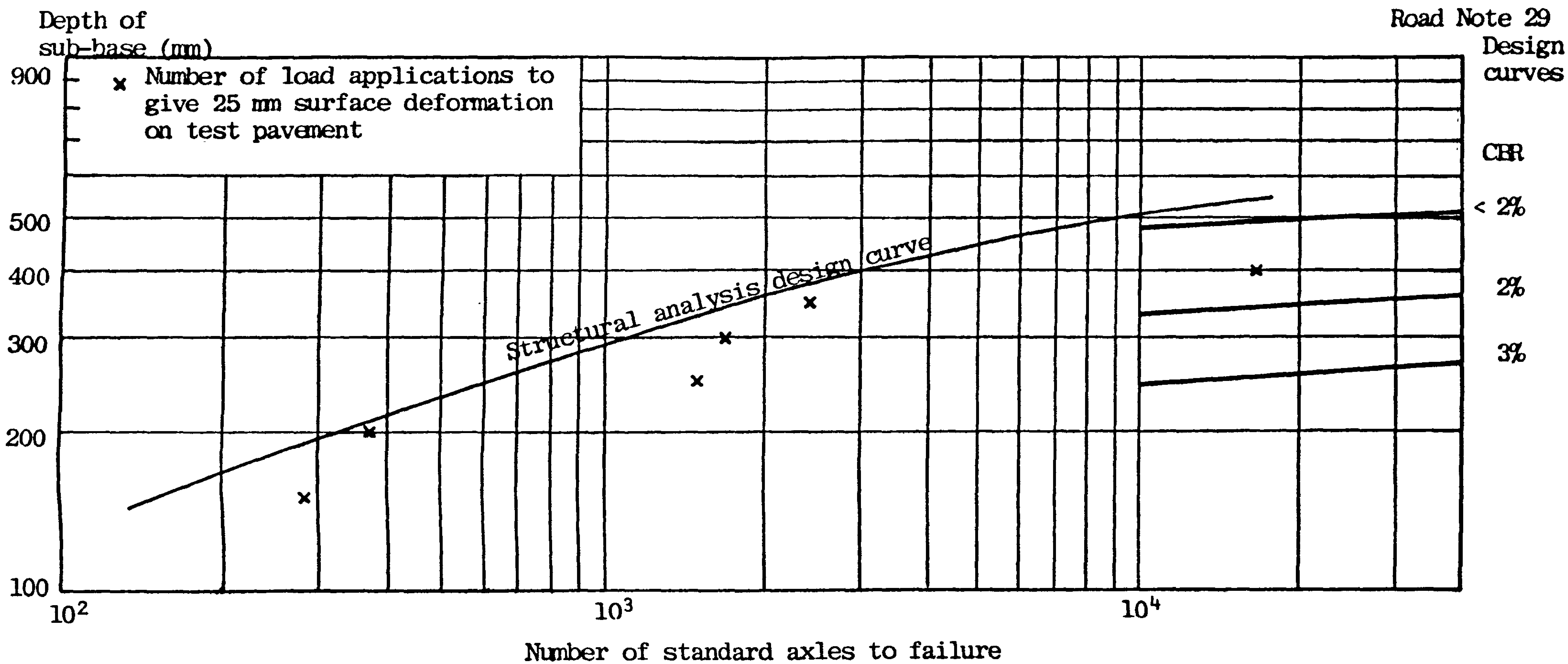
1 from graphical interpolation of Jones' tables

2 $E_{Z3} = \sigma\ (\sigma_{Z2} - \sigma_{Z3}) \times 10^6/E_3$

3 estimated design life, N

$$N = (21600/e_{Z3})^{3\cdot571} \text{ standard axle}$$

Figure 5.7 Comparison of experimental and theoretical response of block paving



the joints, the surfacing can withstand substantial elastic and permanent deformation without loss of interlock and spread very high load concentrations over a large area of the subgrade.

(b) In order to develop full interlock a certain deformation of the surface is required. The sand vibrated into the joints reduces this movement but complete compaction cannot be achieved with the plate vibrator. Trafficking and surface water quickly fill the joints with detritus and, after some initial settlement, the gross deformation, y, is related to the number of load repetitions, N, in the form y :: N**n.

It was generally indicated that current sub-base design recommendations used for block pavements, R.N. 29, were adequate, and possibly conservative. However, unlike asphaltic construction where excessive vertical movement or tensile stresses within the material will bring about a well defined failure point through cracking, a segmental surfacing behaves differently. Further research into the definition of a 'failure' criteria in terms of serviceability limits is required before a more refined design procedure can be developed.

5.8.2. An experimental investigation of the behaviour of a concrete block pavement with a sand sub-base.

The primary aim of conventional asphaltic pavement design is to limit vertical deformation so that fatigue cracking does not occur. In concrete block construction, where the surfacing is already jointed, this limit state is removed and much larger deformations can be tolerated before there is structural damage. Nearly all highway pavement design in the U.K. is based on recommendations given in R.N. 29, which attempts to cover all classes of road from culs-de-sac to urban motorways. R.N. 29 defines four categories of residential road, according to the number of public service vehicles using the road. This experiment considers the two most lightly trafficked categories.

The current practice of applying the design curves of R.N. 29 to lightly trafficked block pavements seems to be excessively conservative and the following points are considered to be the basis for more economical construction :-

(a) riding quality is relatively unimportant: speeds are low and generally vehicles are light.

(b) a block surfacing can accommodate substantial settlement without damage.

(c) if and when required, blocks can be lifted, the sub-base levelled and relaid easily.

This experiment examined the suitability of a low quality sub-base (i.e. ungraded sand) for a concrete block construction on a poor sandy clay subgrade.

The test pavement was constructed at the exit to an industrial yard, with access restricted so that offside wheels of all laden, out-going vehicles passed over the surface. In order to investigate effects of sub-base depth on deformation, the area was divided into five bays, each 2 m long, with a sub-base thickness increasing from 300 mm to 700 mm in increments of 100 mm. Two types of 80 mm concrete blocks were used, giving overall pavement dimensions of approximately 20 m by 2 m.

The existing concrete pavement was broken out and all granular material removed. It was necessary to backfill a 6 m length of the area with boulder clay to give the required profile and, although as much care as possible was taken in compacting this clay, it was not possible to remove all air voids with the equipment available. The sub-base was an unwashed, poorly graded, fine to medium sand with some coarse gravel; the grading curve is compared to the requirements for a type 2 material in Figure 5.~~9~~8.

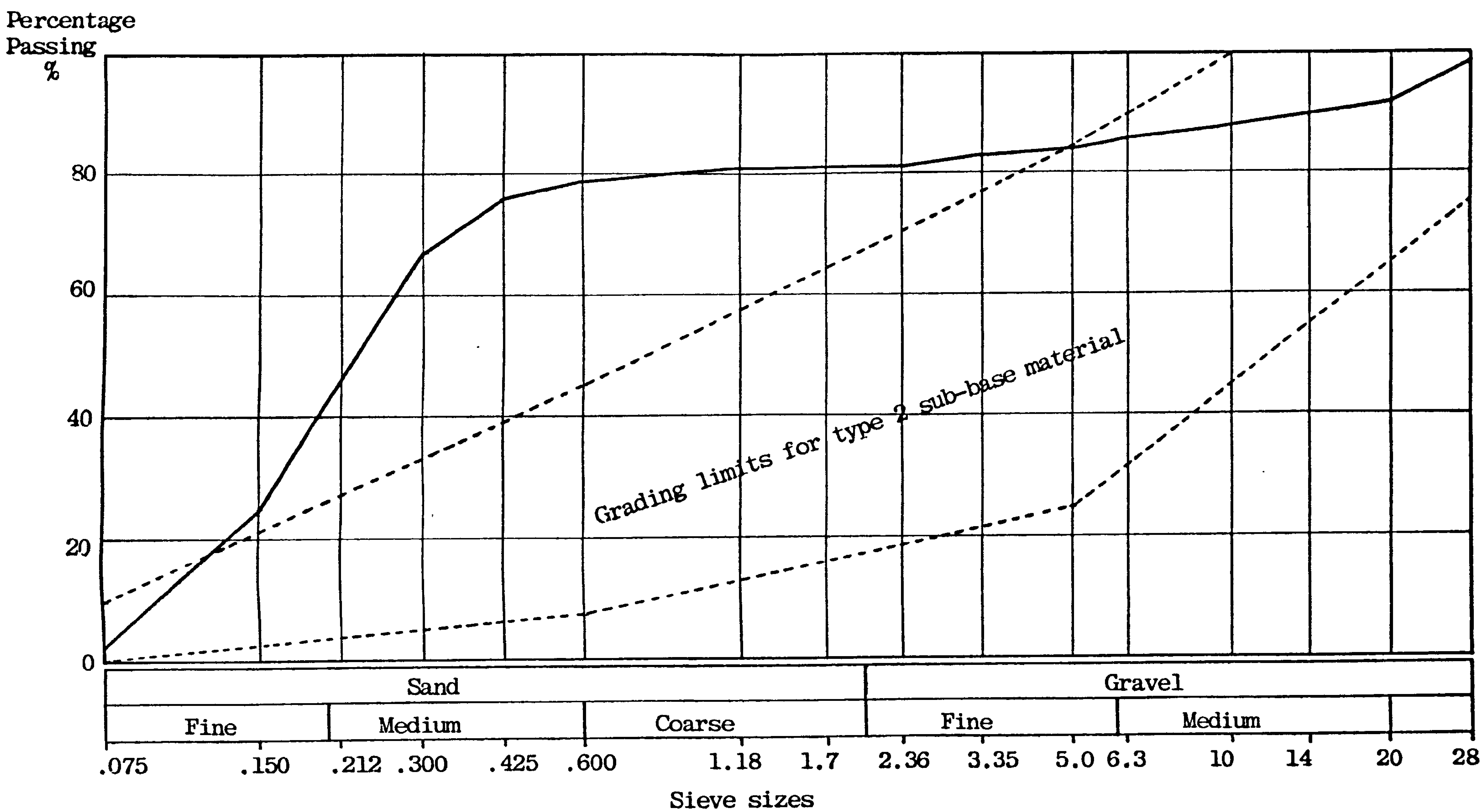
Two types of block, 100 x 200 mm rectangular and 112 x 224 mm shaped, were laid in herringbone pattern in the normal manner. The progressive deformation was measured by a series of eight surveys carried out over a period of 3 months. Scaled profiles for each of the 10 bays were plotted to show the development of rutting. Since the site was under close control, it was possible to estimate total trafficking up to the time of each survey. The overall loading was greater than that given in R.N. 29 for a class II residential road and carried over 4,000 standard axles throughout the experiment.

The following points were noted :-

(i) despite poor subgrade conditions and heavy rainfall, the surfacing maintained its integrity and showed no signs of disintegration.

(ii) large deformations, in excess of 100 mm, were recorded in areas where an imported clay subgrade was laid. The large settlement was due to compaction of boulder clay from excessive voids left during construction. However, after this initial period of

Figure 5.8 Grading curve for sand sub-base material



settlement progressive deformation reduced steadily.

(iii) the two bays which had deformed severely were lifted and relaid and, after a further month of trafficking, the deformations were similar to those of the rest of the trial pavement.

(iv) All profiles showed a general pattern of behaviour: a significant deformation over the first few days followed by a much smaller movements throughout the remaining period. Initial settlement was due to compaction of bedding sand and sub-base, and was between 10 and 20 mm for seven of the ten survey points. The results from this experiment indicate that permanent deformation accumulates linearly against a logarithmic timescale; after an initial settlement period, approximately 4 mm every time the time-scale is doubled. Initial deformations could have been greatly reduced if heavier compaction plant had been used.

(v) there was negligible difference in the behaviour of the block types used in this experiment.

It was generally concluded that the use of a lower quality sub-base material could be applied to block surfacings for residential roads. However, greater care is needed in compaction of the material to ensure initial surface deformations do not impair drainage falls etc.. Also, the sub-base should be able to retain its strength when wet since a block surfacing does not provide a waterproof covering to the pavement structure.

5.9. Other Research and Design Recommendations.

The only other large scale pavement tests on block paving that have been presented have been carried out by Shackel in Australia and South Africa, Refs. 5.19, 5.20 and 5.21.

The first test, 1978, investigated the effect of block thickness and sub-base thickness on deformation characteristics of the pavement. The load was applied through a series of adjacent plates (200 mm square) arranged beneath a loading gantry carrying seven hydraulic jacks. By transferring the load smoothly along the line of jacks a simulated wheel of 14 m/min (approximately 0.25 km/h) was obtained. Contact pressures used were 600 and 900 N/mm2, corresponding to loads of 24 and 36 kN respectively.

The pavement was constructed on an imported sandy loam of very high strength, CBR 66 %. The sub-base was a dolerite crushed rock, conforming to U.K. grading specification of a type 2 sub-base material, and laid in thicknesses of 60, 100 and 160 mm. Three block thicknesses, 60, 80 and 100 mm, of various shapes were tested. The pavement was subjected to 1,000 loading cycles and a survey of the surface was then made. A further 12,000 loading cycles were applied before a second survey was made. From these results it was possible to evaluate rutting, creep, faulting, elastic and permanent deformations and stress distribution of the pavement structure.

The following conclusions, among others, were drawn :-

(1) horizontal creep movements were insignificant when compared to the average joint width of 3 mm (c.f. results from a similar test on an asphaltic material)

(2) permanent deformation occurring in the first 1,000 loading cycles was equal to the progressive deformation over the next 12,000 repetitions. It was estimated that there was a 'shakedown' period of 10,000 load repetitions.

(3) progressive vertical deformations after the initial

'shakedown' was very small and independant of number of load repetitions.

(4) elastic deflections were large, between 1 and 2 mm, but this did not adversely affect the pavement structure.

(5) pavement performance was characterised by two factors, rut depth and curvature ratio. Rut depths were between 1 and 6 mm for all tests, whilst curvature ratios varied from 130 to 1,000.

(6) multi-linear regression techniques were adopted to obtain mathematical relationships between the experimental variables. Rut depth, curvature ratio, elastic deflection and the stress at the interface between sub-base and subgrade were all expressed as functions of block thickness and depth of sub-base.

(7) Vertical stresses throughout the pavement tended to decrease with increasing number of loading cycles, demonstrating the development of interlock under trafficking. Although there was a significant decrease in stresses under 100 mm blocks over the first 10,000 loading cycles, the was no stress

reduction under 60 mm blocks.

(8) there was a general improvement in pavement performance when block thickness was increased from 60 to 100 mm, but very little difference when 80 and 100 mm blocks were compared.

(9) between 30 and 60 % of surface deformation was attributable to compaction and lateral flow of the 30 mm of compacted bedding sand. By reducing this compacted depth to 20 mm, surface deformations reduced from 4 mm to 1 mm; elastic deflections were not affected but vertical subgrade stress increased from 50 to 70 kN/m2.

This experimental work was taken one stage further by Shackel, 1978, when he used these results to formulate a design method. Since block thickness is constant, the only variables that were considered were sub-base thickness and the two limiting criteria, rut depth, R, and curvature ratio, C; design life was not considered a critical parameter. Using multi-linear regression techniques to establish the mathematical functions linking these parameters, a series of design charts was constructed. The design criteria used by Shackel are shown in Table 5.1 (a). A simple multiplication factor was applied to the

Table 5.1 (a) Suggested design criteria for interlocking concrete block pavements (Shackel).

Application	Design criteria	
Light traffic (residential street)	rut depth	< 6 mm
	curvature ratio	> 100
	contact pressure	<0.60 N/mm2
Heavy traffic (arterial road)	rut depth	< 3 mm
	curvature ratio	> 200
	contact pressure	<0.60 N/mm2
Heavy duty industrial (ports)	rut depth	< 3 mm
	curvature ratio	> 100
	contact pressure	< 0.9 N/mm2

Table 5.1 (b) Revised design criteria.

Application	Design criteria	
Light traffic	rut depth	< 20 mm
	curvature ratio	> 50
	contact pressure	<0.60 N/mm2
Heavy traffic	rut depth	< 10 mm
	curvature ratio	> 50
	contact pressure	<0.60 N/mm2
Heavy duty industrial	rut depth	< 10 mm
	curvature ratio	> 100
	contact pressure	< 0.9 N/mm2

sub-base design thickness to accommodate lower CBR values, thus extrapolating subgrade strength from 66 % down to 2 %.

The best design parameters for pavement design are obviously the criteria of permanent deformation and subgrade pressure. Since the latter is a function of subgrade strength, and vertical strains were not recorded, the application of rut depth and curvature characterisation is generally considered satisfactory. The limiting values adopted by Shackel seemed very conservative and less severe values have been suggested as being more realistic, Table 5.1 (b), Ref. 5.22. However, these revised values are outside the range of experimental results used to establish the functions.

The occurence of a 'shakedown' condition and a design life independant of number of load repetitions is shown for the materials and loading used in this experiment, but this is contrary to general experience with weaker subgrade and sub-base materials. The extrapolation to give design thicknesses for low strength subgrades is very simplistic and results in sub-base thicknesses considerably greater than those currently used in the U.K.. Also, wheel loads used in the test are relatively low, equivalent to the single tyre load from a twin-tyred axle (i.e. one quarter of the axle load), and these have been extrapolated to include heavily loaded industrial pavements.

In view of the limited range of variables considered in the experiment, further research is needed before this design approach will be widely accepted.

A second large scale test has been carried out by Shackel, 1979. In this experiment a range of wheel loads applied to different block shapes was investigated. Wheel loads of 40, 50, 60 and 70 kN were applied to test areas of paving with a Heavy Vehicle Simulator; tyre pressure 0.6 N/mm2, wheel speed 1 m/s with the loading evenly distibuted over an area 0.9 m wide.

It was concluded that rectangular blocks were inferior to the variety of shaped blocks tested. However, the experimental procedure differed from current European block construction practice in several ways; the majority of blocks used were 60 mm thick and laid in a stretcherbond pattern, compared to the general requirement of 80 mm blocks in herringbone pattern for vehicular pavements (particularly for rectangular blocks). The conclusions presented assume that the effect of subgrade strength was negligible. However, with CBR values varying between 5 % and 68 %, with typical values of 10, 20,30 and 40 %, it is considered here that the conclusions drawn are not proven, and certainly not applicable to current construction practice in the U.K. and Western Europe.

5.10. Design of Heavy Duty Pavements.

The need for, and development of, a design method for highway pavements with a concrete block surfacing has already been discussed. There was a similar call in the U.K. for design recommendations covering pavements carrying specialised traffic, and in 1978 the C&CA published a design guide for Heavily Trafficked Roads and Paved Areas, Ref. 5.23, and an advisory data sheet, Ref. 5.24, for design of industrial pavements. Examples on the use of both these design methods is given later in Chapter 8, when the resulting thicknesses are compared to those obtained from the analysis presented in this thesis.

5.10.1. Heavily Trafficked Roads.

This publication is, in principle, an extension to the design recommendations for lightly trafficked roads discussed earlier. Here road-base design curves from Road Note 29 are used in a modified form to accommodate block surfacing.

The equivalent thickness of concrete blocks to 160 mm of hot rolled asphalt developed by Knapton, Ref. 5.11, indicates that a road-base is not reqired for pavements carrying less than 1.5 million standard axles (msa). Thus, from the design charts at this point, the following materials are assumed to be of equivalent load carrying capability :-

225 mm	lean concrete and surfacing
170 mm	dense bitumen macadam and surfacing
160 mm	rolled asphalt
80 mm	concrete blocks and bedding sand

In this design method wheel loads are converted to standard axles using equivalence factors given in R.N. 29. The sub-base and road-base are proportioned from the relevant design charts, then the thicknesses given above are deducted to allow for the load carrying capacity of the block surfacing. From a practical viewpoint, minimum thicknesses of 75 mm and 100 mm are given for bitumen bound and cement bound materials respectively. A road-base thickness chart based on these assumptions is included in the publication.

There is, however, a serious anomaly in this approach. Consider the original R.N. 29 design thickness and the proposed thickness for a lean concrete construction :-

Concrete Blocks

Design Life (msa)	1.5	10	80
Road Note 29			
Surfacing	70	115	200
Road base	155	195	215
Total	225	310	415
80 mm Blocks			
Road base	nil	(100)	190

It is clear that the equivalence at 1.5 msa is assumed to be 70 mm of surfacing plus 155 mm lean concrete roadbase. At 10 msa the nominal design of 100 mm gives an equivalence of 115 mm and 95 mm respectively; i.e. more surfacing hence the proportion of lean concrete is reduced. This is extended still further when the 80 msa point is considered, the blocks being equated to 200 mm of bituminous surfacing and 25 mm of lean concrete. However, blocks are equivalent to only 160 mm of rolled asphalt, hence the overall pavement has been severely weakened at this point.

A similar analysis applied to the other two materials considered in this design method, rolled asphalt and dense bitumen macadam, gives a similar result. The use of empirical equivalent thicknesses should not, therefore, be applied to existing design methods.

5.10.2. Heavy Duty Structures.

This design concept is based on a method developed by the

Portland Cement Association, PCA, for 'soil-cement' pavements and extended, as before, to incorporate block construction. The following parameters are used in the design :-

(i) bearing value, or reaction, of the subgrade or sub-base.

(ii) heaviest wheel load carried by the pavement.

(iii) tyre contact areas.

(iv) number of heavy wheel movements throughout design life.

Lean concrete and CBGM, as defined in Specification for Road and Bridge Works, Ref 5.25, fall well within limits specified by the PCA for soil cement, hence these materials have been substituted.

The design chart published by the C&CA is a metric version of that published by the PCA, and subgrade reactions have been converted to equivalent CBR values. Design thickness for the cement bound material is obtained in the normal way, this is then reduced to allow for the load spreading capacity of the blocks. This reduction is specified at 225 mm, regardless of block type.

The PCA method makes no specific requirement on a suitable surfaceing material. However, standard construction practice would require a bituminous surfacing of at least 75 mm, and if this is provided the method specifies a maximum reduction of 25 mm in base thickness. This has been neglected in this revised application of the design method.

It was shown in the previous section that the use of this equivalent thickness can be very misleading and, in certain circumstances, can substantially reduce the strength of the pavement. It will be shown later, in Chapter 8, that the thickness and strength of a surfacing over a cement bound base has a minimal effect on overall pavement strength, unless its stiffness is greater than approximately ten times that of the base. At a modular ratio across the surfacing / base interface of 10, the equivalent thicknesses are in the ratio of 1 to 2. Since the strength of a block surfacing will not be greater than 15,000 N/mm2, a typical value for a poor cement bound granular material, the equivalent thicknesses used here are not only illogical, but result in severe underdesign.

5.11. Conclusion.

It is clear from the brief summary of existing research into concrete blocks that a satisfactory solution to characterisation and design of block pavements has yet to be achieved. The forthcoming International Conference on Concrete Block Paving, Ref. 5.26, will bring together virtually all current knowledge on the subject from around the world, and it is hoped that the necessary research will develop from this complete appraisal.

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CHAPTER SIX

PORT PAVEMENT LOADING

Synopsis

This Chapter presents a comprehensive method of assessing vehicle wheel loading in container handling areas. A concept of equivalent damaging power for a range of axle loads is developed and this applied to a distribution of container gross weights. A suitable design unit, the Port Area Wheel Load, PAWL, is proposed. An example on the assessment of the damage factors for a particular vehicle is given, a computer program which calculates vehicle damage factors is also described. The final section discusses static surface loadings commonly found in container areas.

6.1. Introduction.

A pavement is designed to carry a specific range and number of vehicle movements throughout its design life. The characterisation of wheel loading is obviously a major section in the design process, thus an accurate and suitable assessment of wheel or axle loading is essential. Highway pavements carry very large numbers of relatively light loads which, in the U.K., are subject to a statutory maximum axle load of 11.5 tonnes, at medium speeds both safely and comfortably. Airfield pavements, runways, have to carry relatively few loads, but these are both heavy and travel at high speed. On the other hand, port pavements are subjected to large numbers of relatively large loads, speeds are low and riding quality is of only secondary importance. Perhaps the nearest comparison to pavements in container handling areas is with taxiways and hardstand areas of airfields.

It was shown in Chapter 4 that different design concepts are applied to highway and airfield pavements; highways are designed for a specific number of repetitions of a standard design unit, whilst aircraft runways are designed for a standard number of maximum gear load repetitions. However, despite extensive loading analyses available for both these classes of pavement, no detailed information for loading in port areas has been assembled. This Chapter analyses the range and magnitude of wheel loads carried by pavements in both Lo-Lo and Ro-Ro container berths.

Since all axle or wheel loads are not of the same magnitude, it is necessary to assess the complete range in terms of relative damaging power. The maximum axle loads currently used in a container handling area are significantly greater than highway loadings, approximately 90 tonnes for a front lift truck. Thus, apart from assessing the effects of load distribution for a particular vehicle, it is also necessary to define a suitable unit to ensure that the pavement structure is designed for the correct magnitude of loading. The "Port Area Wheel Load", PAWL, is defined as the standard design unit, and a method of assessing PAWL ratings of container handling vehicles described in detail. This analysis has been applied to the vehicles commonly used in container ports and ratings for these are also given, Refs. 6.1 and 6.2.

Finally in this Chapter, static surface loading from stored cargo and trailer dolly wheels are considered. Whilst these loading conditions do not affect structural design of the pavement, they are directly related to the durability of the surfacing material.

6.2. Range and Frequency of Loading.

Studies of highway traffic loading show that it is not the heaviest loads which are the most damaging to a pavement, but where magnitude and frequency combine to give a critical condition. In a port, the frequency of a particular wheel load for a given vehicle can be related to the distribution of gross weights of the containers the vehicles carry.

Annual statistics published by the National Ports Council, Ref. 6.3, give the numbers and nett weights of the various types of containers handled in U.K. ports. These figures have been converted to gross weights, and Table 6.1 summarises container weight characteristics averaged over 10 years, 1967 to 1976. It can be seen that currently approximately 60 % of all containers handled are 20 ft units, and since 30 ft units are not widely used, it is assumed that the remainder of units handled are 40 ft units. This is obviously a simplification since there are many different types of container; aluminium, refrigerated, half height, liquid tanks etc, and empty units have been omitted completely. Containers having only a one-way payload must be returned for re-use, the proportion of empty units varies widely between different trade routes, but may be as high as 1 in 6 of the total handled. These could, however, be accommodated in a

Table 6.1 Summary of container characteristics.

Size of container	Maximum gross weight (t)	Tare weight (t)	Av. nett weight (t)	Average gross wt (t)	Relative occurrence (%)
20 ft	20.170	2.00	12.36	14.36	75
30 ft	25.400	3.00	14.25	17.25	< 5
40 ft	30.480	3.40	13.44	16.84	20

more detailed breakdown if significant numbers were known to pass through the port or were handled in a particular area.

The distribution of container weights for both 20 ft and 40 ft units are plotted in histogram form in Figure 6.1 and tabular form in Table 6.2, these values being an average of the annual statistics from 1974 to 1976. The actual distribution used for the vehicle load analysis is derived from the proportion of 20 ft and 40 ft units carried by the vehicle. This is currently a 60 / 40 division in favour of 20 ft units, although general economies of scale are tending to an even division.

6.3. The Concept of Relative Damage.

It is known that heavier wheel or axle loads are proportionally more damaging to a pavement than lighter ones. The relationship between pavement damage and wheel load is generally assumed to follow an approximate fourth power law, although estimates of the power vary from 3.5 to over 6. Pavement damage is an arbitrary quantity related to stresses and strains throughout the pavement structure. Industrial pavements have to carry much larger loads than highway pavements, stresses within the pavement must be similar and it follows, therefore, that this equivalence factor would quantify relative damaging

Figure 6.1   Frequency distribution of containers by gross weight

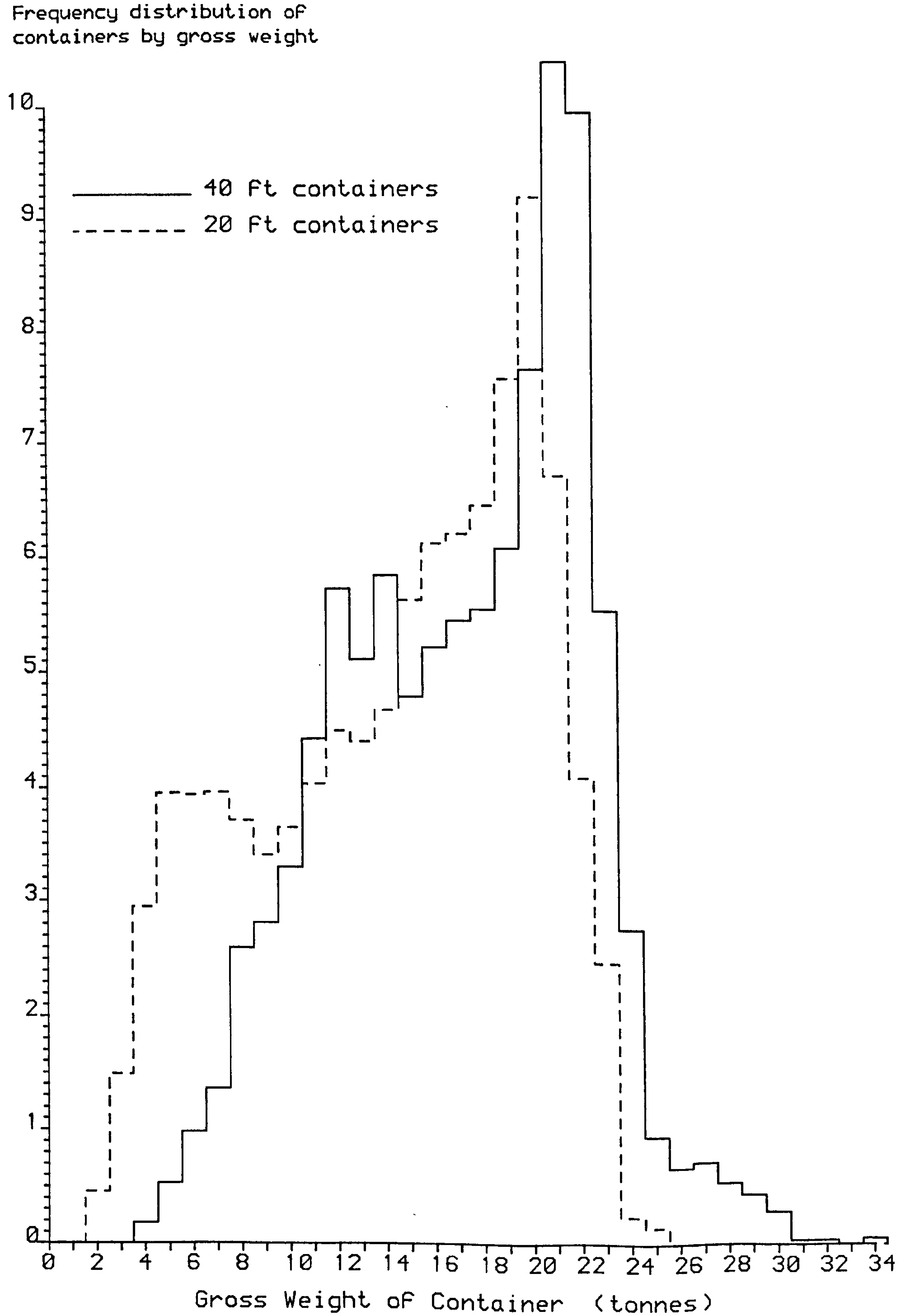


Table 6.2 Distribution of container weights by gross weight.

Cont. Weight	Proportion of 40 ft/20 ft containers 100/0	60/40	50/50	40/60	0/100
0.	0.00	0.00	0.00	0.00	0.00
1.	0.00	0.00	0.00	0.00	0.00
2.	0.00	0.18	0.23	0.28	0.46
3.	0.00	0.60	0.74	0.89	1.49
4.	0.18	1.29	1.57	1.84	2.95
5.	0.53	1.90	2.25	2.59	3.96
6.	0.98	2.17	2.46	2.76	3.94
7.	1.37	2.41	2.67	2.93	3.97
8.	2.60	3.05	3.16	3.27	3.72
9.	2.82	3.05	3.11	3.17	3.41
10.	3.30	3.44	3.48	3.52	3.66
11.	4.43	4.28	4.24	4.20	4.04
12.	5.73	5.24	5.12	4.99	4.50
13.	5.12	4.83	4.76	4.69	4.41
14.	5.85	5.38	5.26	5.14	4.67
15.	4.78	5.12	5.21	5.29	5.63
16.	5.22	5.58	5.67	5.76	6.13
17.	5.45	5.75	5.83	5.91	6.21
18.	5.55	5.91	6.00	6.10	6.46
19.	6.08	6.68	6.83	6.98	7.58
20.	7.67	8.28	8.43	8.58	9.19
21.	10.40	8.93	8.56	8.19	6.72
22.	9.95	7.60	7.02	6.43	4.08
23.	5.53	4.31	4.00	3.69	2.47
24.	2.75	1.75	1.50	1.25	0.24
25.	0.95	0.63	0.55	0.47	0.15
26.	0.67	0.40	0.33	0.27	0.00
27.	0.72	0.43	0.36	0.29	0.00
28.	0.53	0.32	0.27	0.21	0.00
29.	0.43	0.26	0.22	0.17	0.00
30.	0.28	0.17	0.14	0.11	0.00
31.	0.03	0.02	0.02	0.01	0.00
32.	0.03	0.02	0.02	0.01	0.00
33.	0.00	0.00	0.00	0.00	0.00
34.	0.05	0.03	0.02	0.02	0.00

power of the range of axle loads found in ports. Direct extrapolation of current highway design recommendations, Ref. 6.4, is not practicable, numbers given in "standard axles" would become too large, hence a mathematical relationship has been adopted here. Heukelom and Klomp, Ref. 6.5, proposed the following equation giving pavement damage, D, in terms of the wheel load, W, and contact stress (or tyre pressure), p, :-

D :: W ** 3.75 x p ** 1.25

Highway design is based on a single standard loading condition, "standard axle", defined as an 80 kN axle load, tyre pressure is usually assumed to be 0.50 N/mm2. However, with larger loads covering a much wider spectrum, it is not practicable to adopt a single standard for design of industrial pavements. A basic unit is necessary to quantify damaging power, and for this reason a new standard is proposed. This is defined as the Port Area Wheel Load, or PAWL, as follows :-

Wheel load	12.0 tonnes
Contact pressure	0.80 N/mm2

Use of the above equation shows that one PAWL (an axle load of 24.0 tonnes) is equivalent to approximately 100 standard axles.

When analysing the loading characteristics of a series of vehicles, two quantities have to be established :-

1) the average damage rating for each vehicle so that the total number of PAWLs a pavement will carry can be established.

2) the most damaging combination of frequency and wheel load, the "critical" load, this is then adopted as the design unit in pavement design calculations.

The calculation of PAWL ratings is best explained by a simple example.

6.4. Example of Evaluation of Damage Ratings.

The basic dimensions of two front lift trucks (FLTs) typical of those used in container handling, the Barton 20/CH and Knapper 40/CH, are given in Table 6.3. Both machines are fitted with a toplift frame, a fixed 20 ft frame on the smaller machine and a telescopic 20 - 40 ft frame on the larger vehicle. The weights of these two lifting frames are 2 tonnes and 6 tonnes respectively. In this example, the Barton handles only 20 ft units whilst the Knapper spends approximately 80 % of its time

Table 6.3 Vehicle specifications for Barton and Knapper FLTs.

	Barton FLT 20/CH	Knapper FLT 40/CH
Lifting capacity (t)	24.00	37.5
Unladen Wt. with Toplift	40.00	55.00
Wheelbase (m)	4.750	5.250
Load Centre (m)	2.250	2.500
Unladen Wheel Loads (t)		
Front	11.00	15.13
Rear	9.00	12.38
Proportion of 20ft/40ft Containers Handled	100 / 0	20 / 80
PAWL Ratings		
Unladen	1.06	3.50
Maximum or Critical	19.45	36.70
Average	10.77	24.56

on 40 ft units.

From simple statics, wheel loads for each vehicle can be found for a given load on the frame. The front and rear wheel loads for both vehicles are plotted in Figure 6.2. In these calculations the load on twin tyres is assumed to act on the pavement as a single load with a contact stress equal to the true tyre pressure. Although true loading on the surfacing is not as severe as this, the difference between twin tyres and a single contact area at lower levels in the pavement is small and negligible.

The wheel loads given in Figure 6.2 have been converted to damaging powers, in terms of PAWLs, by use of the above equation, these are plotted in Figure 6.3. It can be seen that the rear wheel loads for both FLTs do relatively little damage compared to the front wheels.

Figure 6.4 shows the proportional distribution of the container weights handled by the two machines; 100 % 20 ft containers for the Barton and a 20 % - 80 % mix of 20 ft - 40 ft containers for the Knapper.

The last two figures have been multiplied together to give the combined effect of wheel load and frequency, Figure 6.5.

Figure 6.2 Wheel loads for Barton and Knapper FLTs.

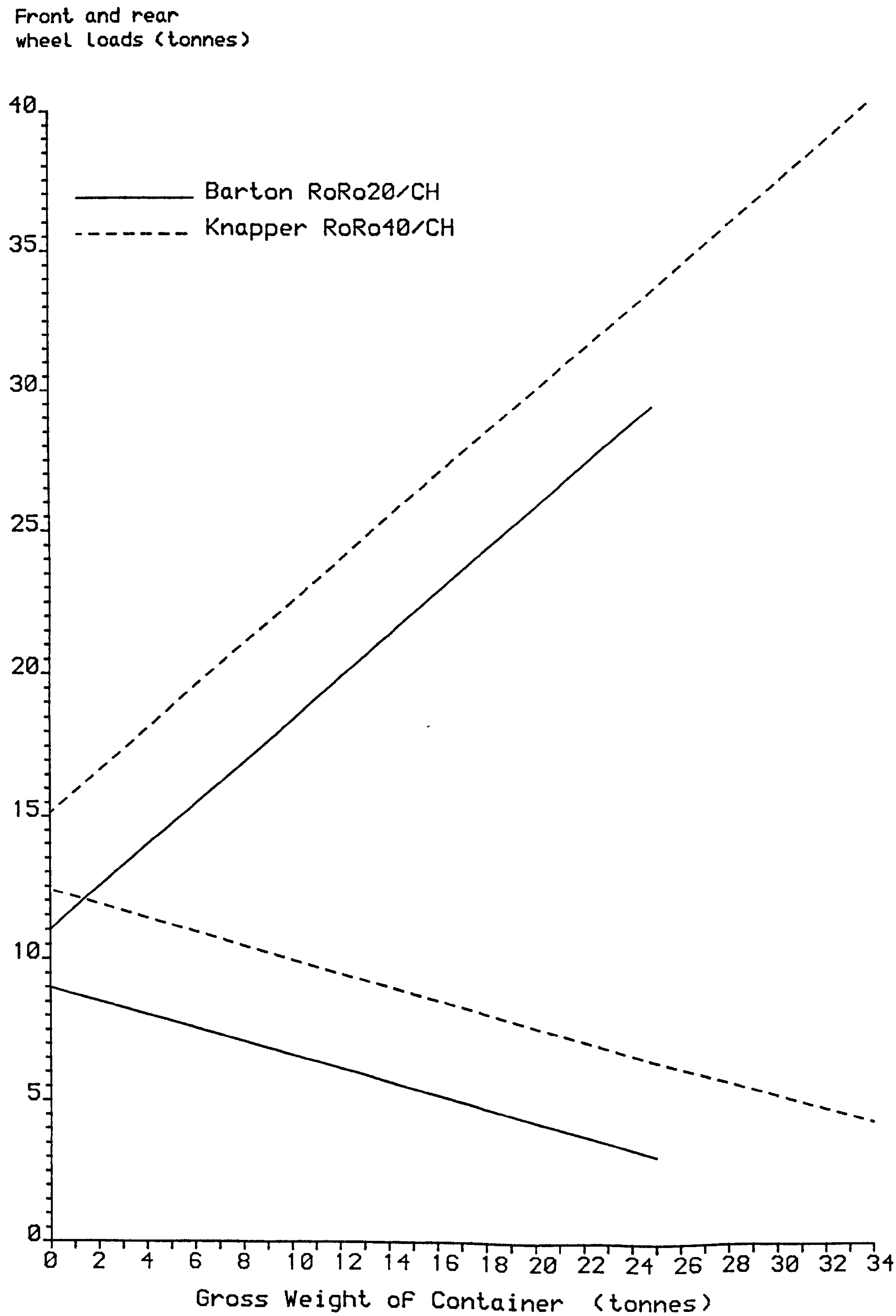


Figure 6.3 Wheel damage factors for Barton and Knapper FLTs.

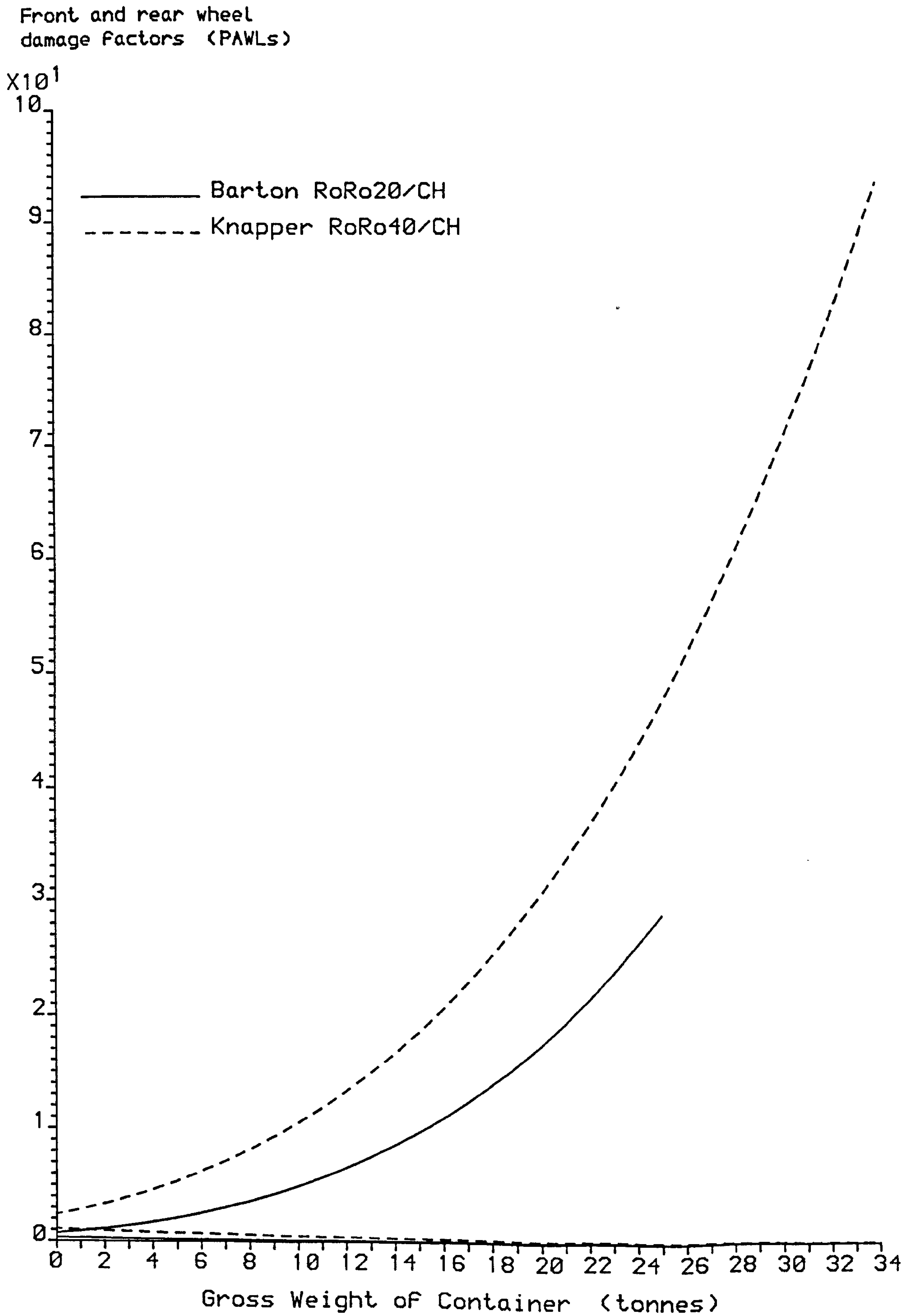


Figure 6.4 Container weight distributions for Barton and Knapper FLTs.

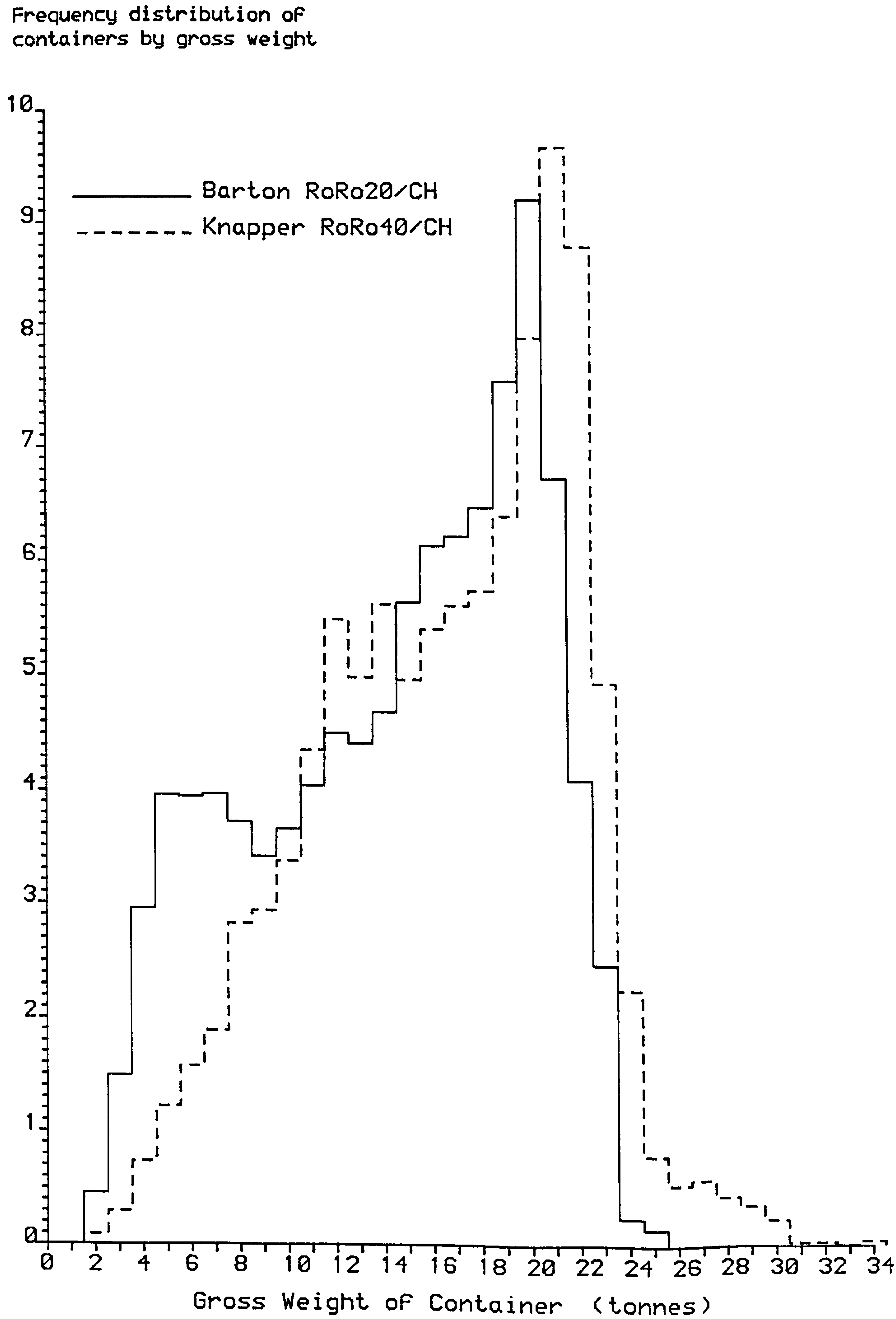
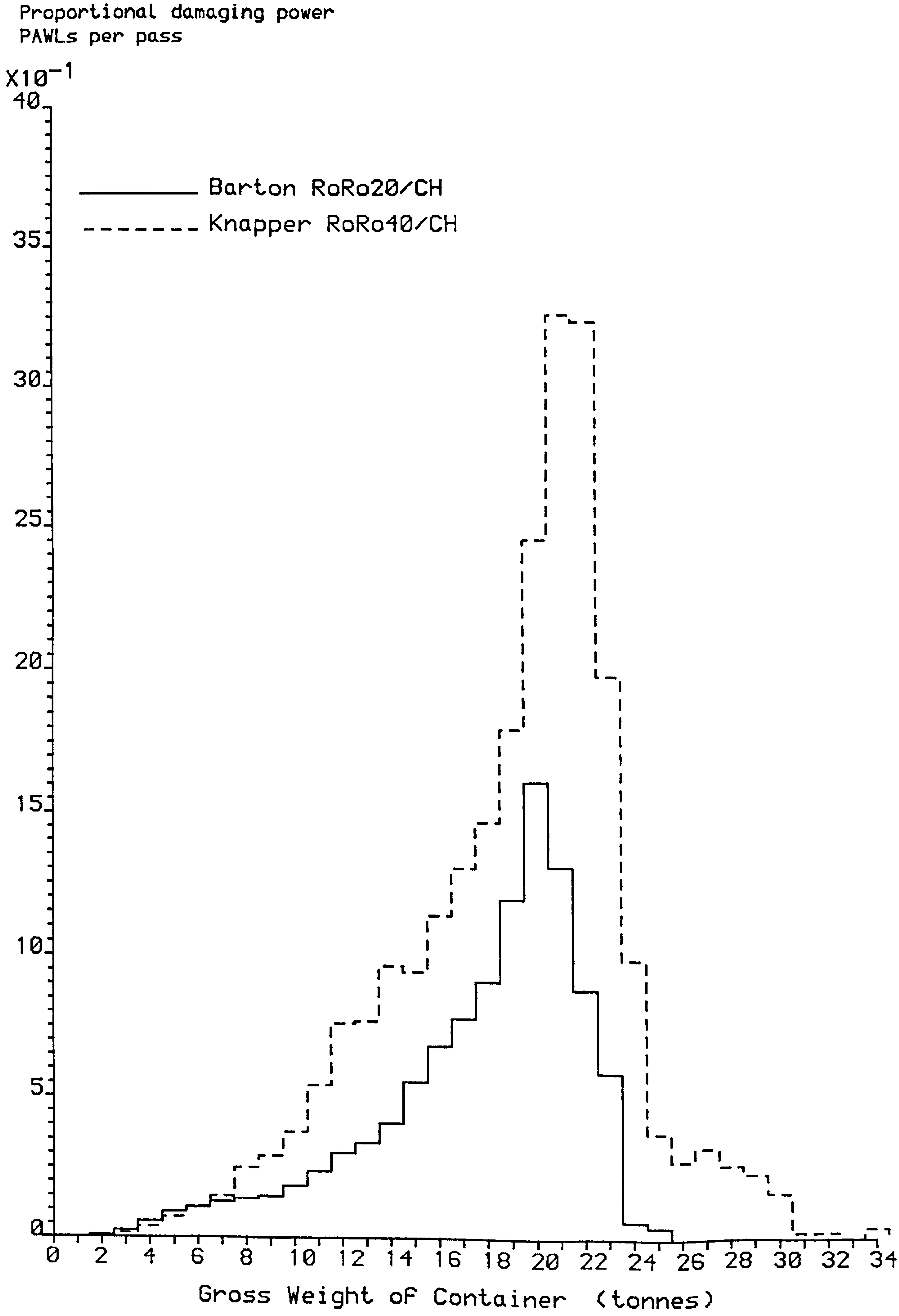


Figure 6.5 Proportional damage factors for Barton and Knapper FLTs.



This plot clearly shows that the larger vehicle is significantly more damaging to the pavement than the lighter vehicle. The area under each histogram in this Figure represents the total damaging effect of 100 container handling movements. Thus, summing all columns gives the average damage rating for each vehicle, these values are included in Table 6.3.

It has already been established that for a satisfactory design procedure, the magnitude of the loading must be established. For this, the most damaging wheel load in the complete load spectrum, the critical wheel load, is adopted. In this example, this coresponds to a 22 tonne container on the Knapper. From Figure 6.3, this is equivalent to a PAWL rating of 36.7. A detailed analysis of the calculations for the Knapper is given in Table 6.4.

From these PAWL ratings, the total number of PAWLs a pavement has to carry and the critical design unit can be established. The equivalent number of repetitions of the design wheel load, or design life, is calculated simply by dividing the total number of PAWLs by the critical PAWL rating from all vehicles under consideration.

Table 6.4 Detailed analysis of damage rating calculations for Knapper FLT 40/CH.

Container Weight	Cont. Distr.	Wheel Loads Front	Wheel Loads Rear	Wheel Damage Factors Front	Wheel Damage Factors Rear	Total Prop. Damage
0.0	0.00	15.13	12.38	2.38	1.12	0.000
1.0	0.00	15.86	12.14	2.85	1.04	0.000
2.0	0.09	16.60	11.90	3.38	0.97	0.004
3.0	0.30	17.34	11.66	3.98	0.90	0.015
4.0	0.74	18.08	11.42	4.65	0.83	0.040
5.0	1.22	18.82	11.18	5.40	0.77	0.075
6.0	1.57	19.55	10.95	6.24	0.71	0.109
7.0	1.89	20.29	10.71	7.17	0.65	0.148
8.0	2.82	21.03	10.47	8.20	0.60	0.248
9.0	2.94	21.77	10.23	9.33	0.55	0.290
10.0	3.37	22.51	9.99	10.57	0.50	0.373
11.0	4.35	23.24	9.76	11.93	0.46	0.540
12.0	5.49	23.98	9.52	13.42	0.42	0.759
13.0	4.98	24.72	9.28	15.03	0.38	0.767
14.0	5.61	25.46	9.04	16.79	0.35	0.962
15.0	4.95	26.20	8.80	18.68	0.31	0.941
16.0	5.40	26.93	8.57	20.74	0.28	1.135
17.0	5.60	27.67	8.33	22.95	0.25	1.300
18.0	5.73	28.41	8.09	25.33	0.23	1.465
19.0	6.38	29.15	7.85	27.89	0.20	1.793
20.0	7.97	29.89	7.61	30.63	0.18	2.456
21.0	9.66	30.63	7.38	33.56	0.16	3.259
22.0	8.78	31.36	7.14	36.70	0.14	3.233
23.0	4.92	32.10	6.90	40.04	0.13	1.976
24.0	2.25	32.84	6.66	43.61	0.11	0.983
25.0	0.79	33.58	6.42	47.40	0.10	0.375
26.0	0.53	34.32	6.18	51.42	0.08	0.275
27.0	0.57	35.05	5.95	55.69	0.07	0.320
28.0	0.43	35.79	5.71	60.22	0.06	0.257
29.0	0.35	36.53	5.47	65.01	0.05	0.225
30.0	0.23	37.27	5.23	70.08	0.04	0.159
31.0	0.03	38.01	4.99	75.42	0.04	0.020
32.0	0.03	38.74	4.76	81.07	0.03	0.021
33.0	0.00	39.48	4.52	87.01	0.03	0.000
34.0	0.04	40.22	4.28	93.27	0.02	0.037

Average PAWL rating for vehicle = 24.563

6.5. Calculation of PAWL Ratings.

This analysis is obviously suited to computer operation and a program has been developed which calculates PAWL ratings for any type of handling vehicle. This program, like the analysis programs described in the next Chapter, provides full prompts for the operator and all data entry is in free format. Detailed results are given in both tabular and graphical form, whilst principal values are displayed immediately on the terminal.

The data required by the program is :-

1) number of axles on vehicle
2) maximum load capacity (tonnes)
3) axle loads, both unladen and laden (tonnes)
4) tyre pressure (N/mm2)
3) distribution of container weights

The distribution of container weights is based on the number and proportion of 20 ft and 40 ft units handled. These are stored within the program and five options are given :-

(i) 20 ft containers only
(ii) 40 ft containers only
(iii) 20 ft and 40 ft containers

(iv) multiple handling of 20 ft units

(v) multiple handling of 40 ft units

The third option requests the percentage of 40 ft units; (iv) and (v) request the number of containers carried by the vehicle.

Average PAWL ratings for each axle, together with critical and unladen ratings, are displayed immediately. Tabular output lists container weights, proportional distribution, wheel loads, wheel damage factors and proportional damaging powers, as well as critical PAWL ratings for each axle with a separate page for each axle. Graphical output is in the form of 4 graphs, similar to those given above. Examples on both forms of output are shown in Figures 6.6 and 6.7 respectively, where a 40 ft tractor/trailer unit carrying two 40 ft containers is analysed.

6.6. Tyres and Dynamics.

In this analysis it is assumed that surface contact pressure is equal to tyre pressure and that the contact area is circular. For many of the larger machines the tyres have heavy treads which are designed for operation on soft ground, the treads sinking into the surface. This clearly does not happen on a well constructed pavement and the load is concentrated on the tread

Figure 6.6 Typical tabular output from PAWL rating program.

Straddle Carrier. 8 wheels. 60 % 40 ft containers

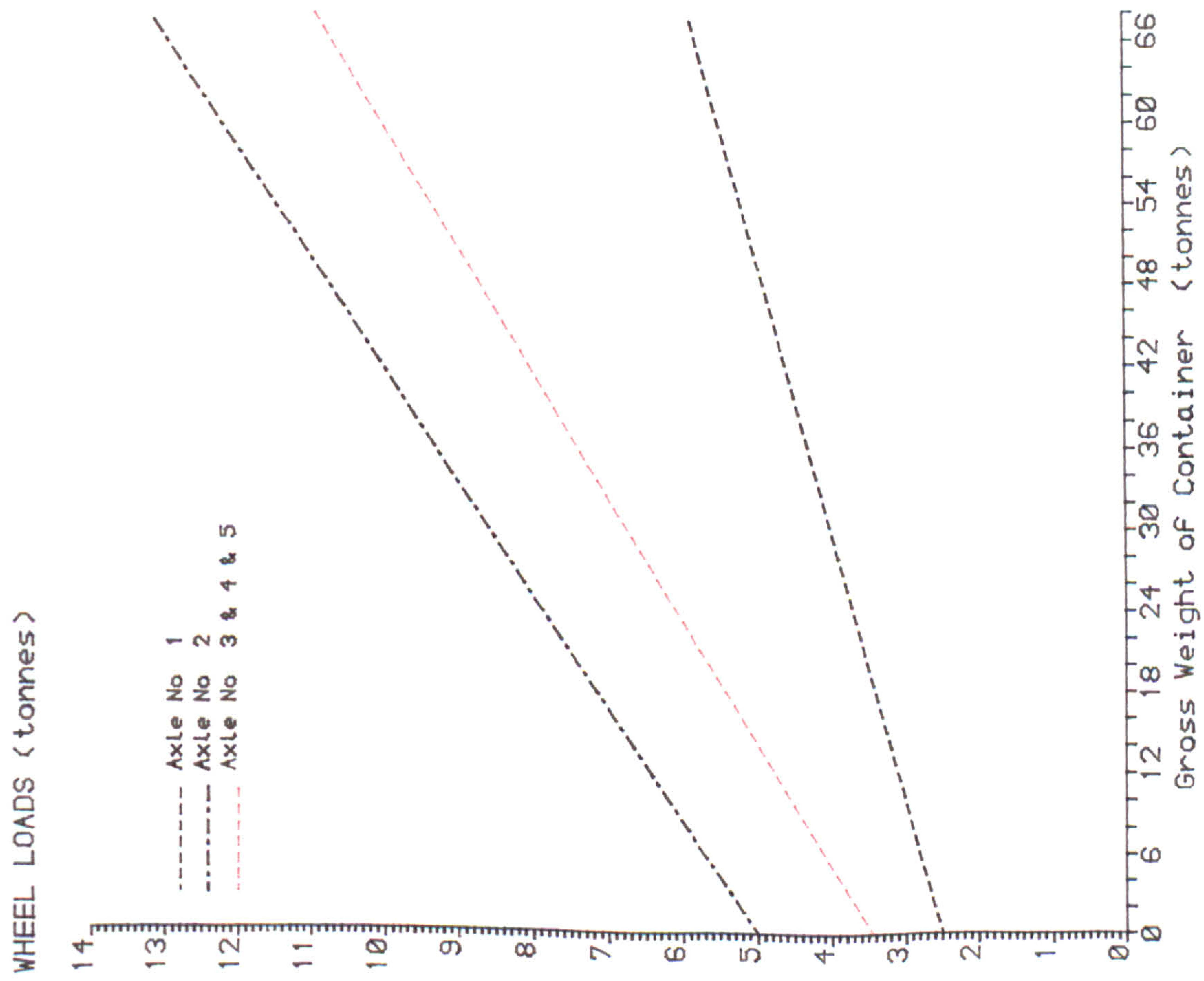
Axle number 1 of 4; 4 axles similar

Container Weight	Proport. Distr.	Wheel Load	Wheel Damage	Proport. Damage
0.	0.00	6.75	0.12	0.000
1.	0.00	6.88	0.12	0.000
2.	0.18	7.00	0.13	0.024
3.	0.60	7.13	0.14	0.084
4.	1.29	7.25	0.15	0.195
5.	1.90	7.38	0.16	0.307
6.	2.17	7.50	0.17	0.372
7.	2.41	7.63	0.18	0.439
8.	3.05	7.75	0.19	0.591
9.	3.05	7.88	0.21	0.629
10.	3.44	8.00	0.22	0.753
11.	4.28	8.13	0.23	0.991
12.	5.24	8.25	0.25	1.286
13.	4.83	8.38	0.26	1.255
14.	5.38	8.50	0.27	1.476
15.	5.12	8.63	0.29	1.485
16.	5.58	8.75	0.31	1.707
17.	5.75	8.88	0.32	1.856
18.	5.91	9.00	0.34	2.011
19.	6.68	9.13	0.36	2.393
20.	8.28	9.25	0.38	3.119
21.	8.93	9.38	0.40	3.537
22.	7.60	9.50	0.42	3.166
23.	4.31	9.63	0.44	1.883
24.	1.75	9.75	0.46	0.802
25.	0.63	9.88	0.48	0.303
26.	0.40	10.00	0.50	0.202
27.	0.43	10.13	0.53	0.227
28.	0.32	10.25	0.55	0.177
29.	0.26	10.38	0.58	0.151
30.	0.17	10.50	0.61	0.103
31.	0.02	10.63	0.63	0.013
32.	0.02	10.75	0.66	0.013
33.	0.00	10.88	0.69	0.000
34.	0.03	11.00	0.72	0.022

Total Axle Damage	0.32	PAWLs
Total for 4 similar axles	1.26	PAWLs
Average Rating for Whole Vehicle	1.26	PAWLs
Unladen Damage Rating	0.46	PAWLs
Most Damaging Rating	0.40	PAWLs

Figure 6.7 (a) Typical graphical output from PAWL rating program.

Terminal trailer. 2 x 40 Ft containers.



Terminal trailer. 2 x 40 Ft containers.

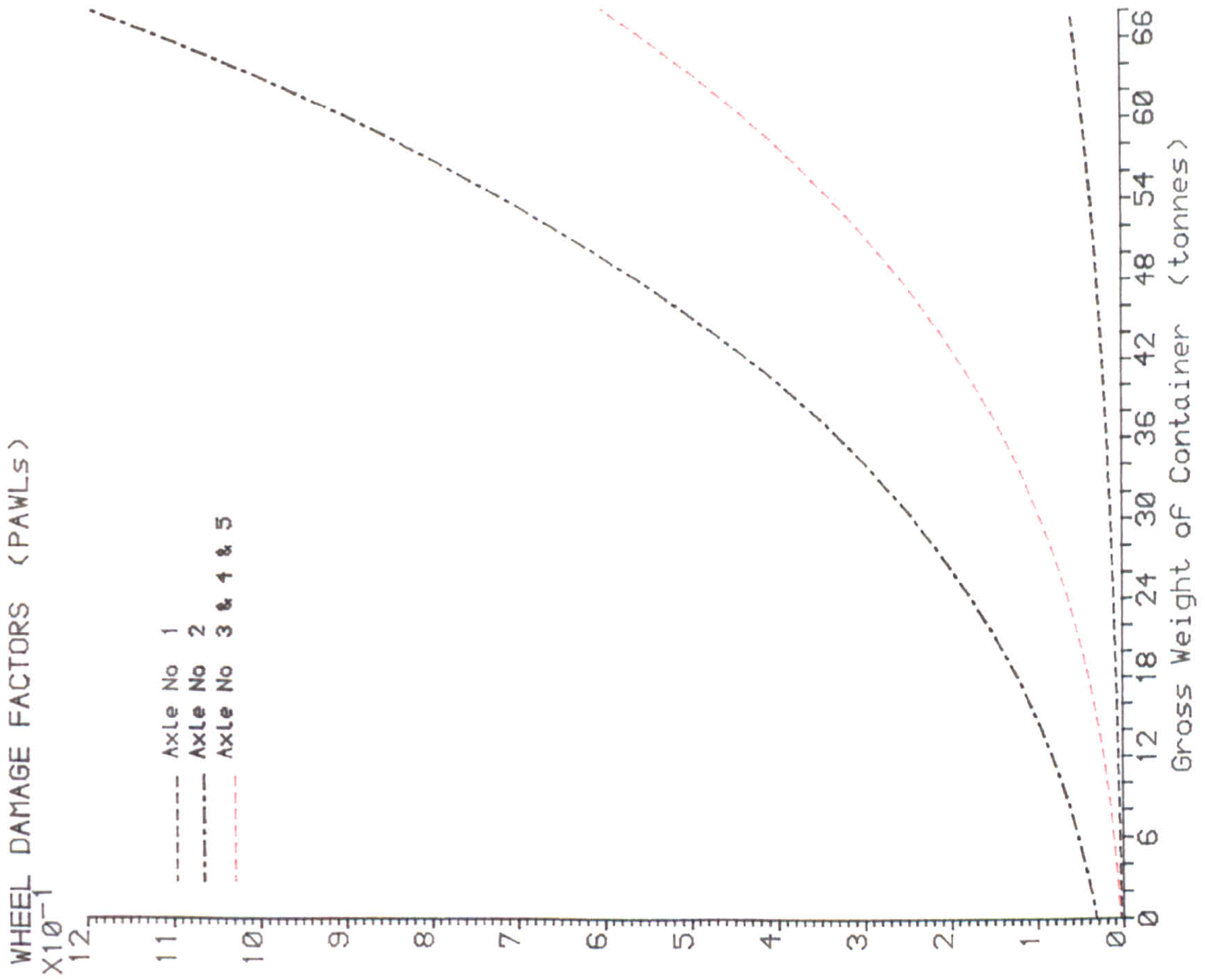
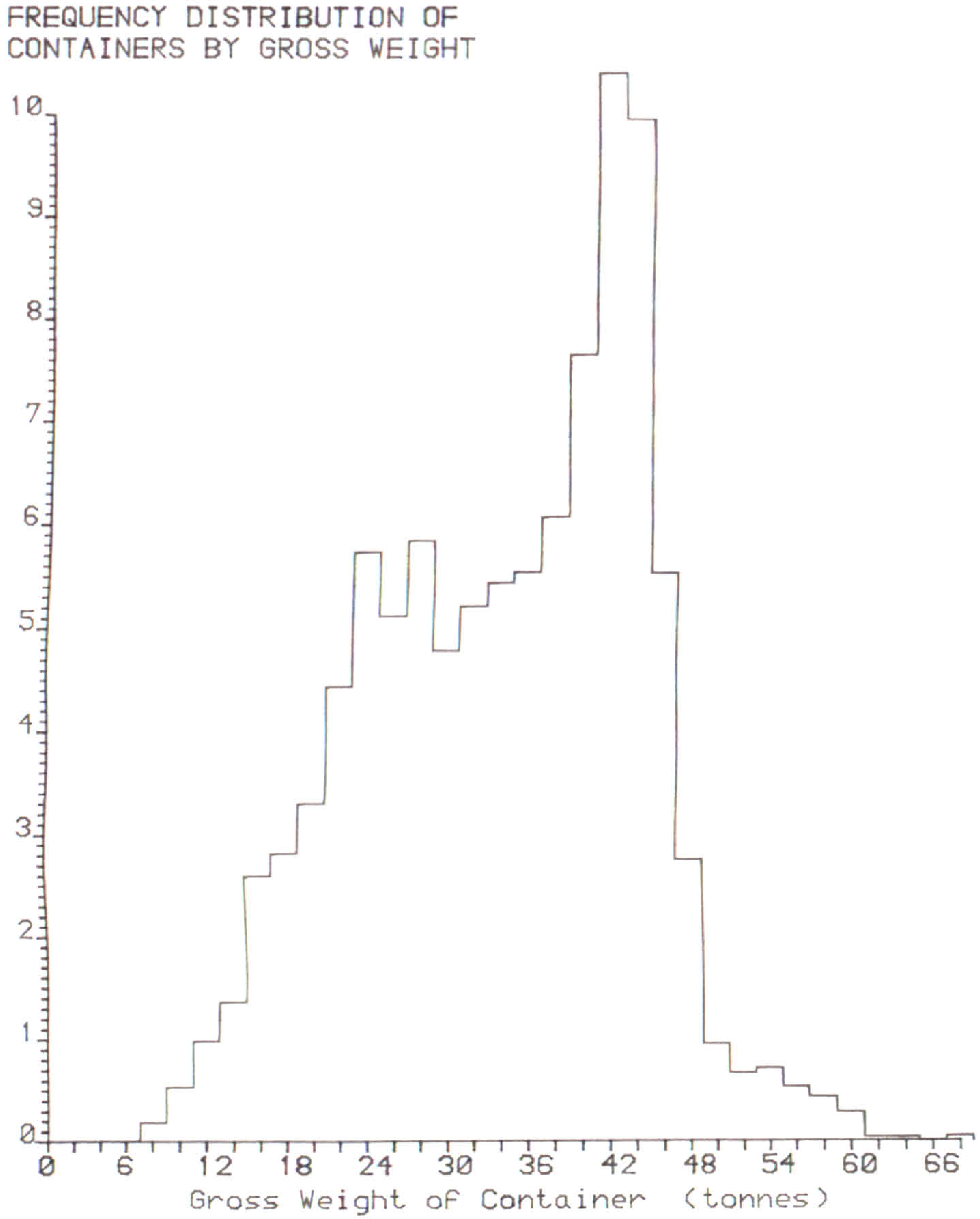
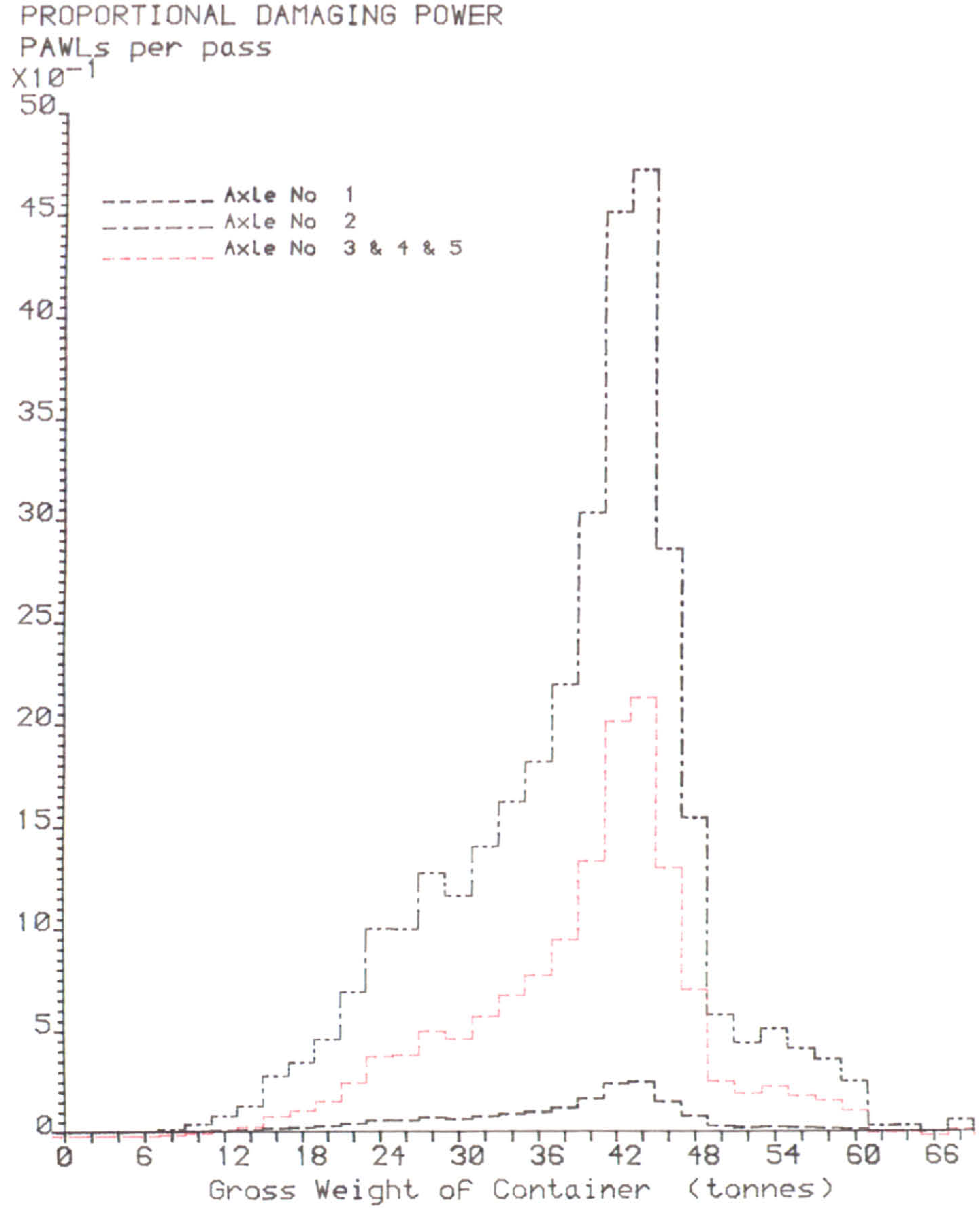


Figure 6.7 (b) Typical graphical output from PAWL rating program.

Terminal trailer. 2 x 40 ft containers.



Terminal trailer. 2 x 40 ft containers.



bars, thus the loading pattern is not circular and the actual surface stress can be much higher than the tyre presure. Although this increases stresses within the surfacing material, the stress concentrations are substantially dissipated when considering the materials lower in the pavement structure, and the effect on structural design parameters is negligible.

The effects of dynamic loading induced by cornering, acceleration, braking and surface uneveness are much more difficult to ascertain, this is a topic in need of fundamental research. Some container handling vehicles travel at relatively high speeds and have high centres of gravity, hence the load can be constantly swinging from one wheel to another. However, until specific data is available, it is assumed that dynamic effects are similar for all handling vehicles, and all analysis covered here is based on static loading conditions of the vehicles.

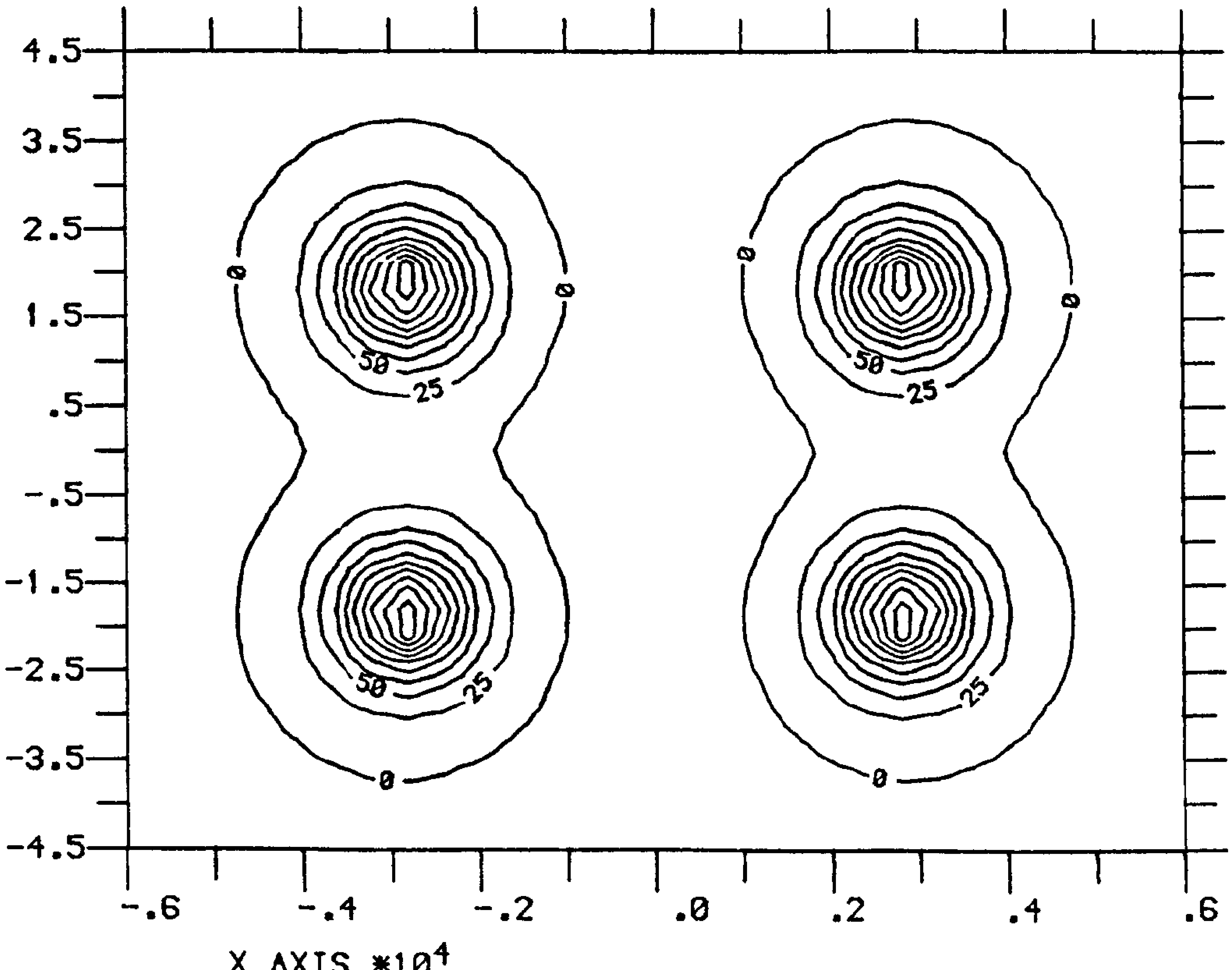
6.7. Axle Spacing and Gear Configurations.

The example on calculation of PAWL ratings assumed that the wheel load was equal to half the axle load, and was represented by a single contact area. However, heavily loaded axles usually have twin tyres and, particularly for trailers, there are two or more axles closely spaced down the vehicle. There is obviously overlap of stresses within the pavement, and these should be considered in the loading characterisation. The degree of interaction between adjacent axles increases as axle spacing decreases and strength of the pavement structure increases. This is clearly demonstrated when a 4-wheeled and an 8-wheeled straddle carrier are compared.

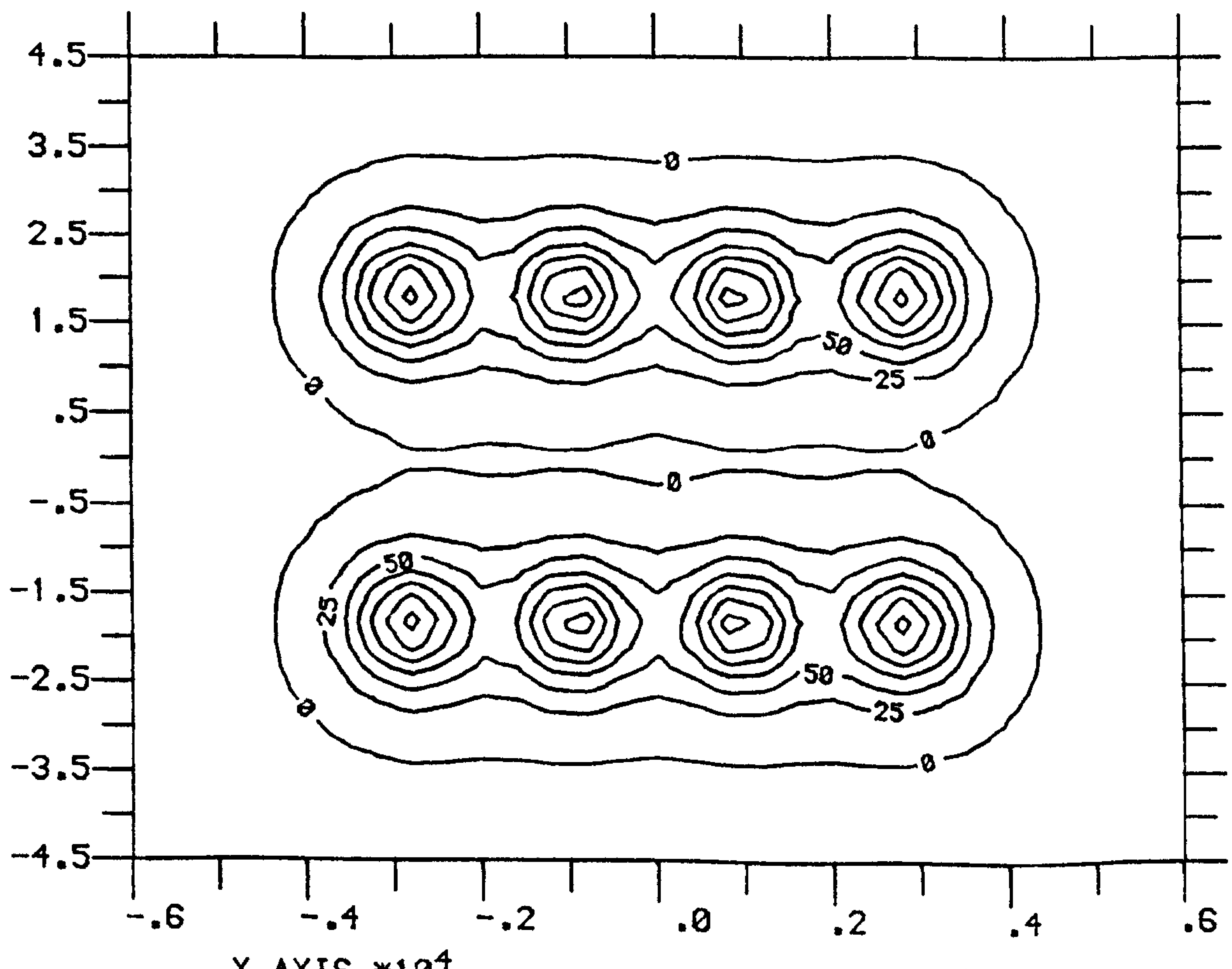
The inclusion of extra wheels and heavy suspension increases the gross weight by 20 tonnes to 108 tonnes, thus the wheel loadings are significantly reduced, but wheel spacing is reduced by 66 %. The contour plots shown in Figure 6.8, (a) and (b), show the tensile radial strain at the bottom of a lean concrete base material for both an under-designed and an over-designed pavement. On the weaker pavement the advantage of more wheels to spread the load is clearly shown, the four distinct peaks are only 67 % of those under the 4-wheeled model. However, when the stronger pavement is considered, stress overlap has increased giving one large load pulse under the 8-wheeled vehicle. In this case the maximum stress is lower for the lighter vehicle, although there are twice as many load pulses for each vehicle pass.

A similar effect is encountered when twin tyre loading conditions are used. The difference in critical vertical subgrade strain between a twin contact area and an assumed

Figure 6.8 (a) Contour plots of tangential strain under straddle carriers on an underdesigned pavement.

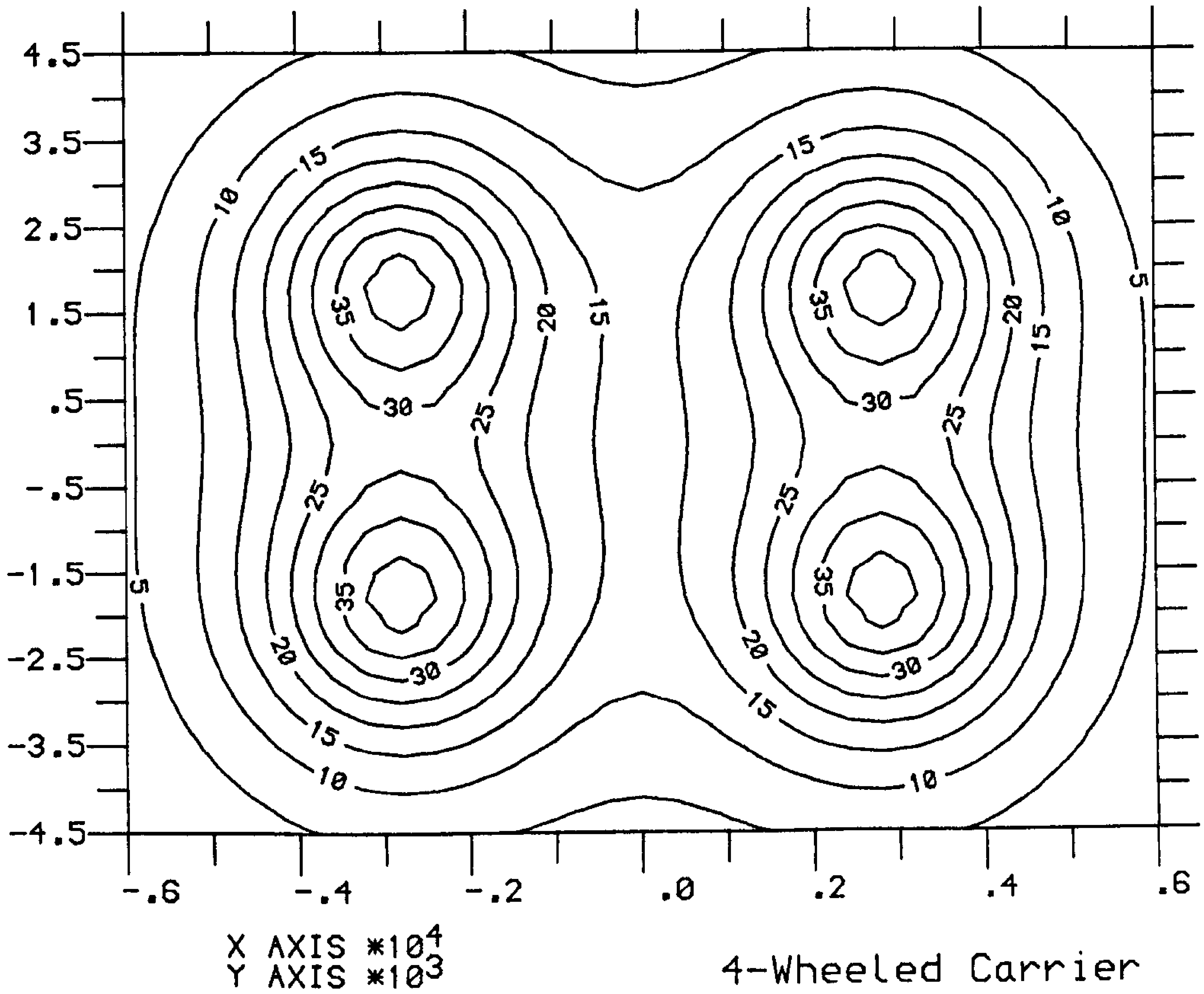


4-Wheeled Carrier

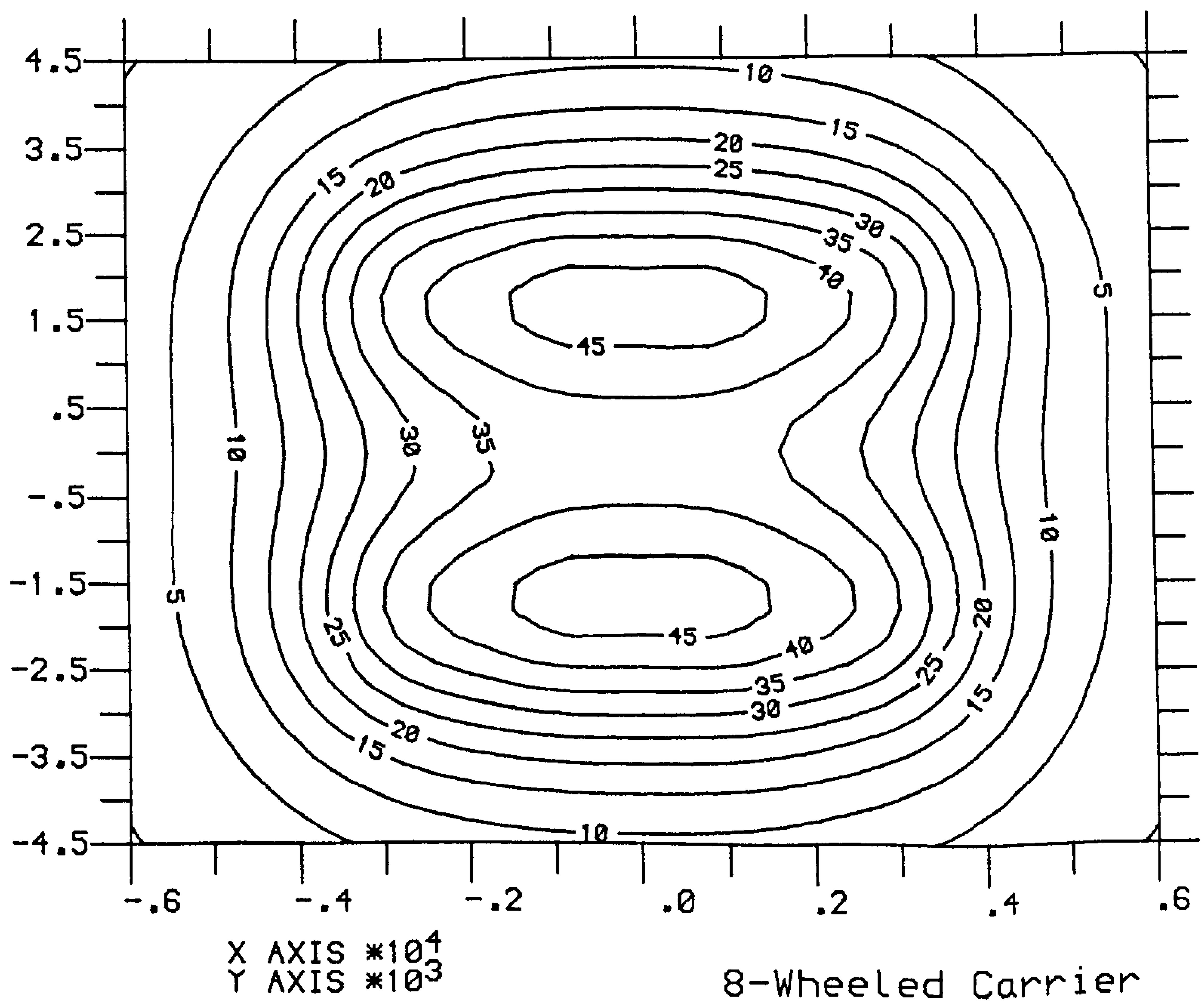


8-Wheeled Carrier

Figure 6.8 (b) Contour plots of tangential strain under straddle carriers on an overdesigned pavement.



4-Wheeled Carrier



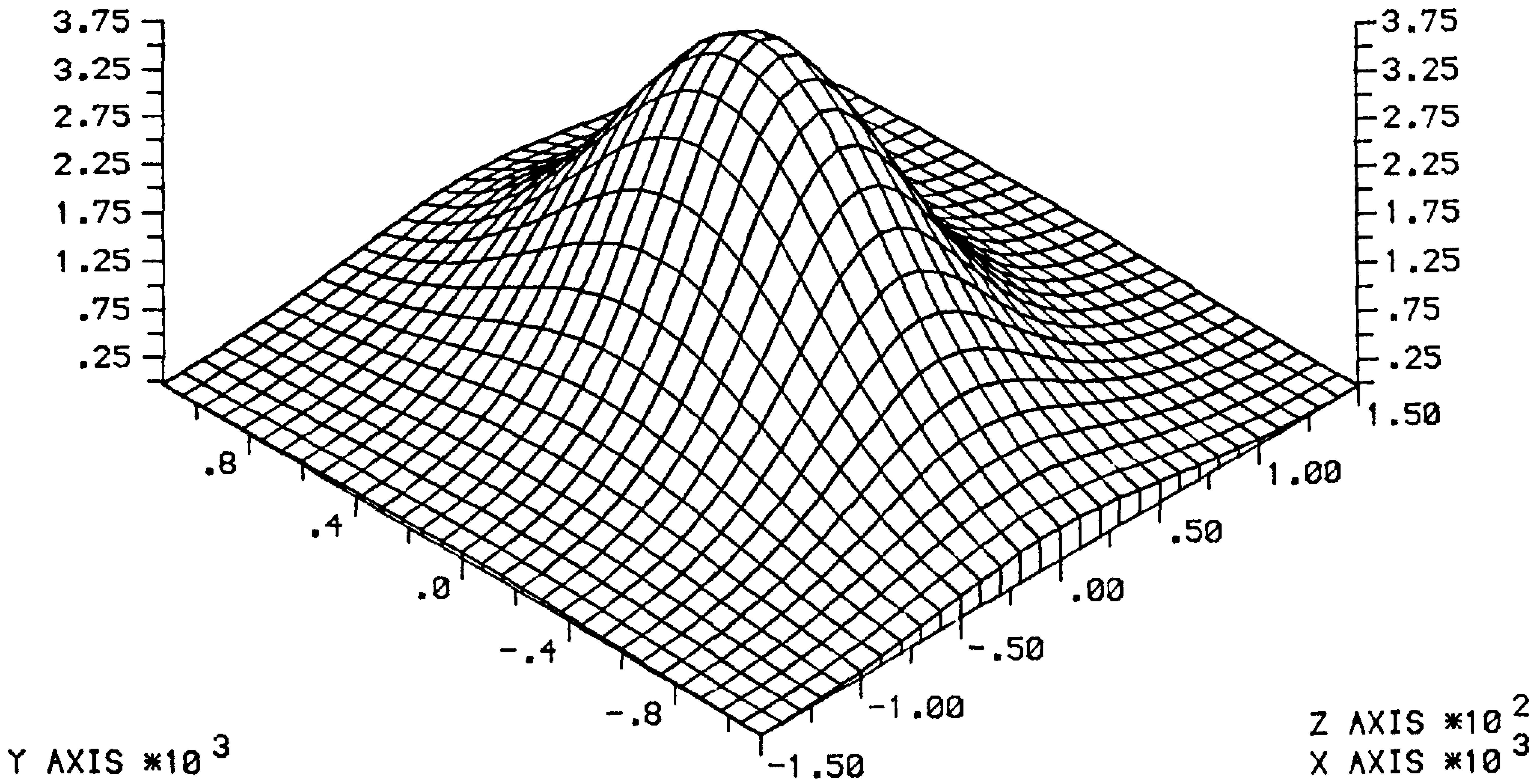
8-Wheeled Carrier

single contact area on a pavement of moderate strength is shown in Figure 6.9. The difference in magnitude between the true loading and the analysis model is small and negligible.

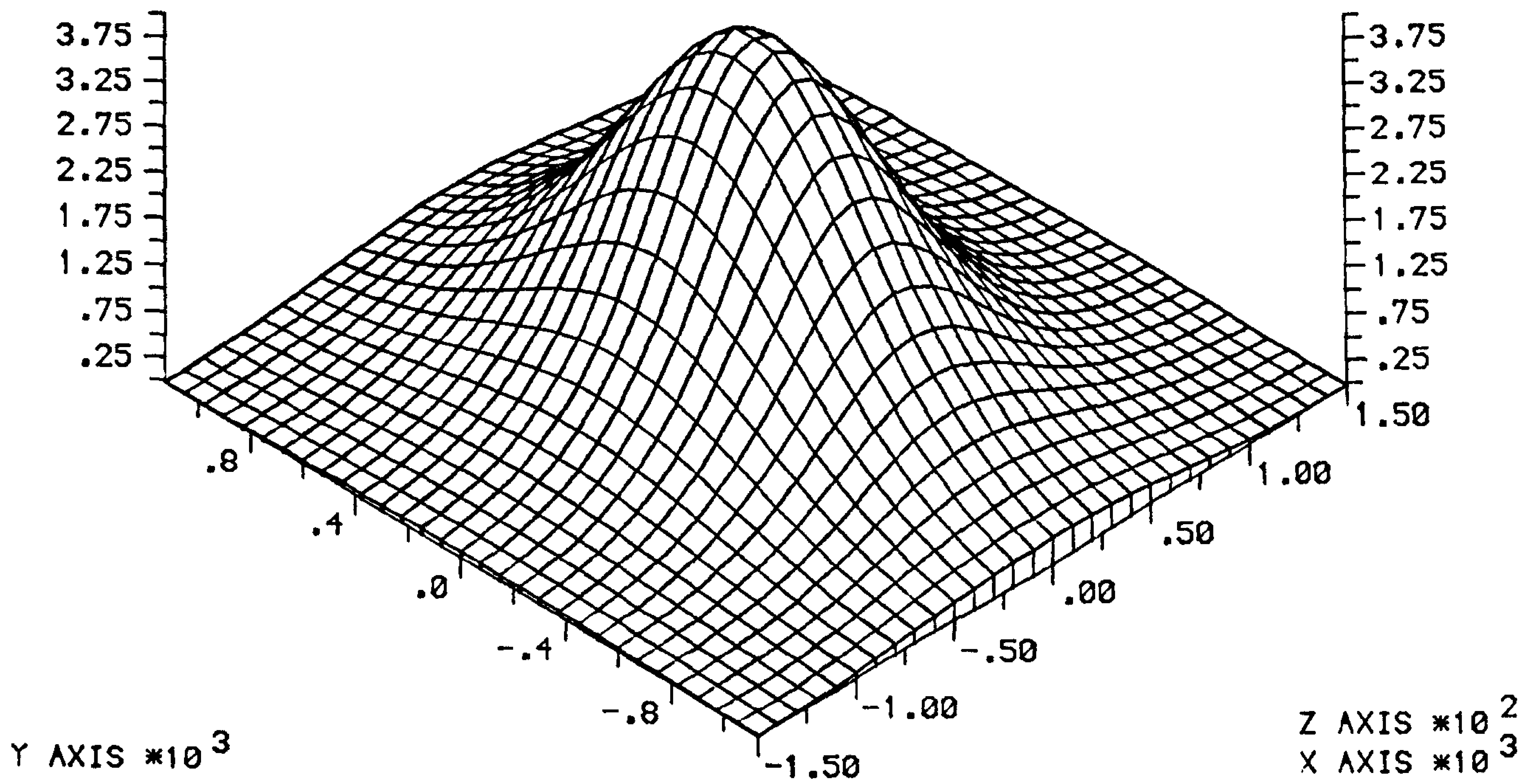
Terminal trailers have one set of wheels towards the rear of the unit, these are small solid rubber tyres arranged on a series of simple pivot axles, the arrangement of these axles is given later in Figure 6.13. The interaction between adjacent wheels increases with depth below the surface. This is clearly shown for terminal trailers in Figure 6.10 where vertical stress has been plotted for a series of effective depths. Because the contact areas are so close together, the wheels are considered to act as a single axle with a contact stress equal to that under one of the rubber tyres.

The problem of axle spacing and gear configuration is not limited to the superposition of critical stresses and strains. Stress overlap within the structure increases the length of the pulse load considerably, although the maximum value is of similar magnitude. However, the stiffness of bituminous materials is a function of the inverse of loading time, thus pavement stiffness is reduced, which in turn affects the stress distribution within the structure. To attempt to accommodate this range of parameters in a general characterisation of wheel loading would be impossibly complex, the assumptions adopted

Figure 6.9 Comparison of vertical subgrade strain under a twin and a single tyred wheel

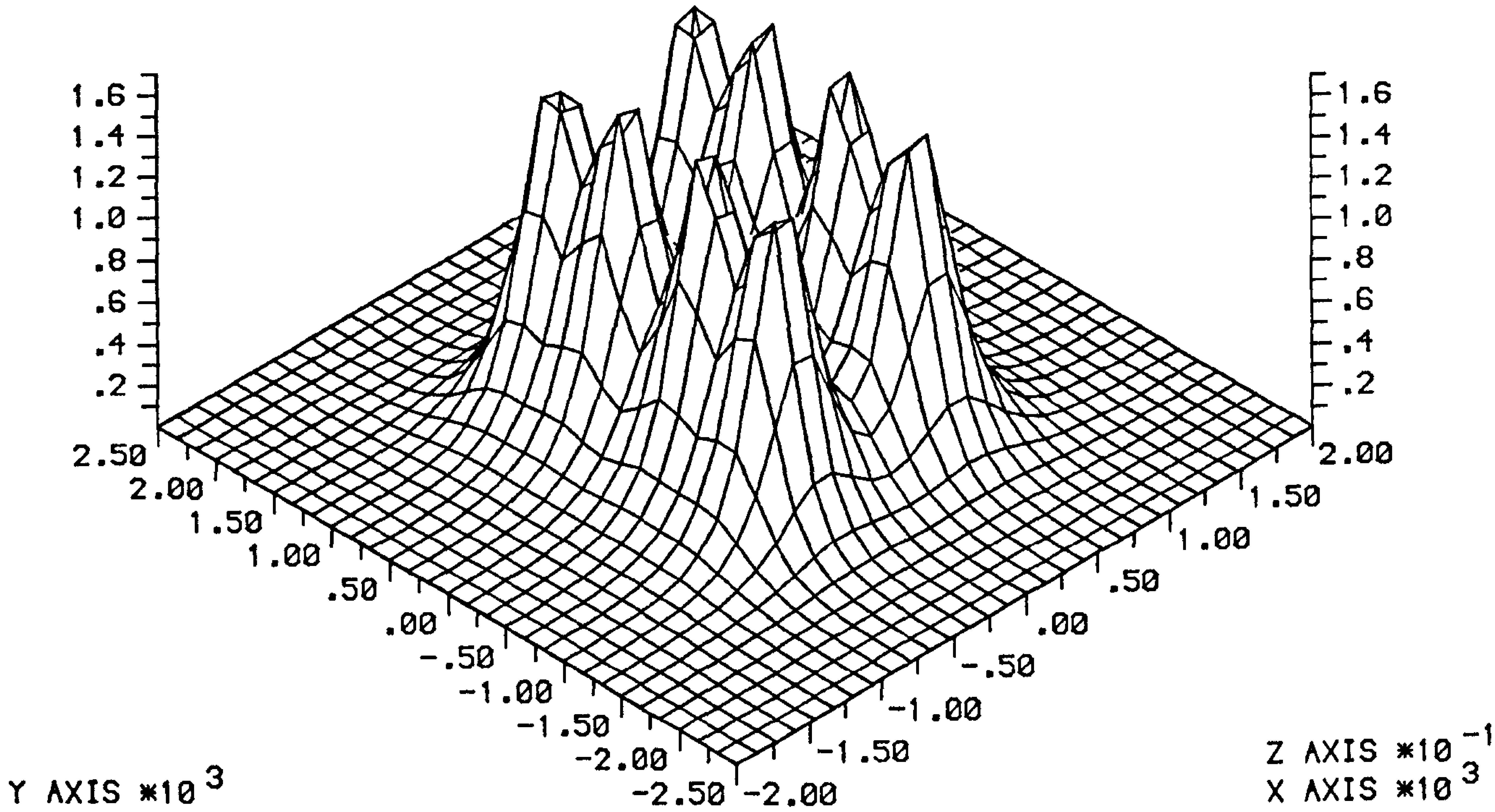


Twin Tyred Wheel
(tyre spacing 320 mm along Y axis)

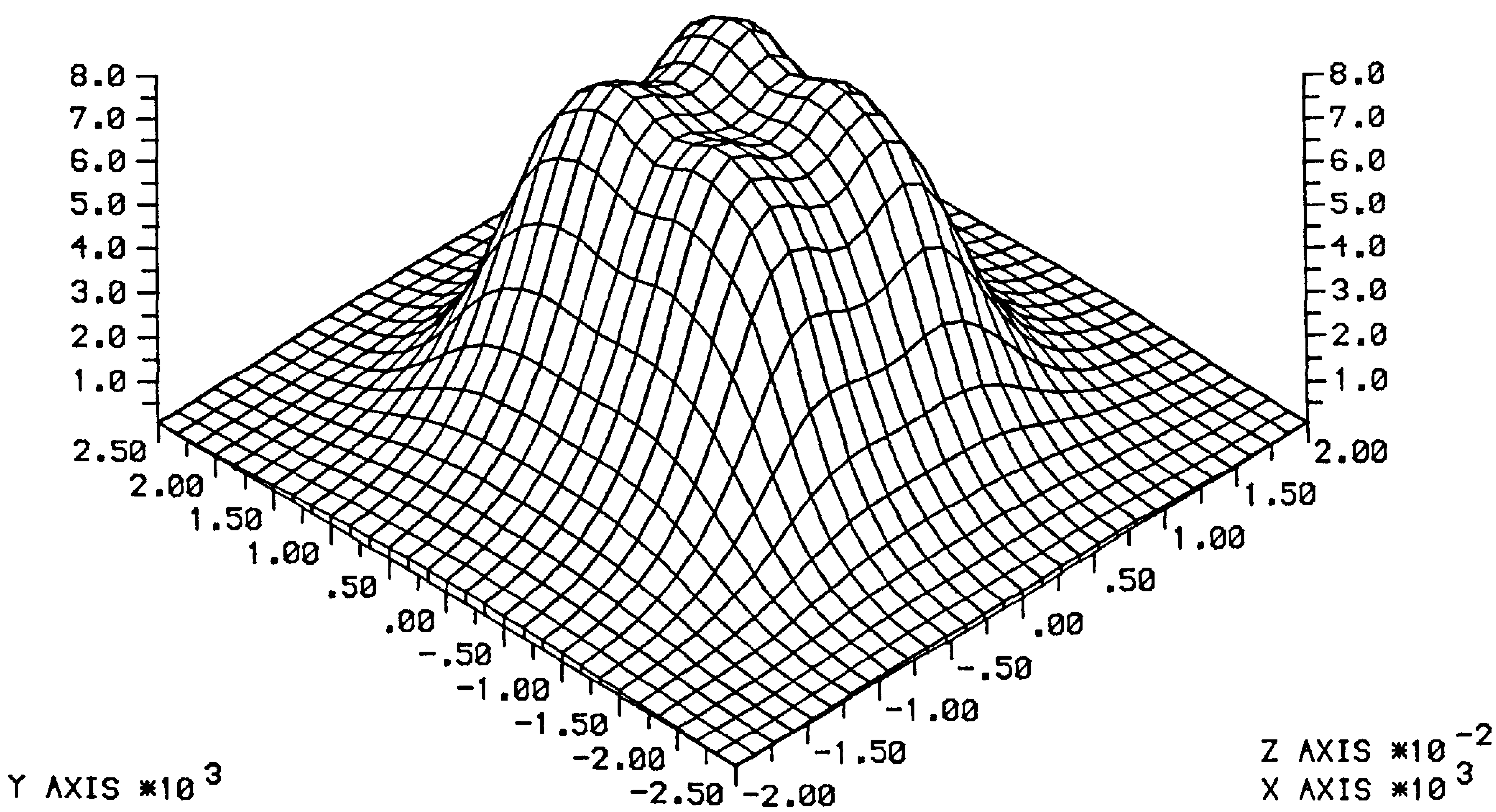


Single Tyred Wheel

Figure 6.10 (a) Vertical stress at different depths under a fully laden 40 ft. terminal trailer

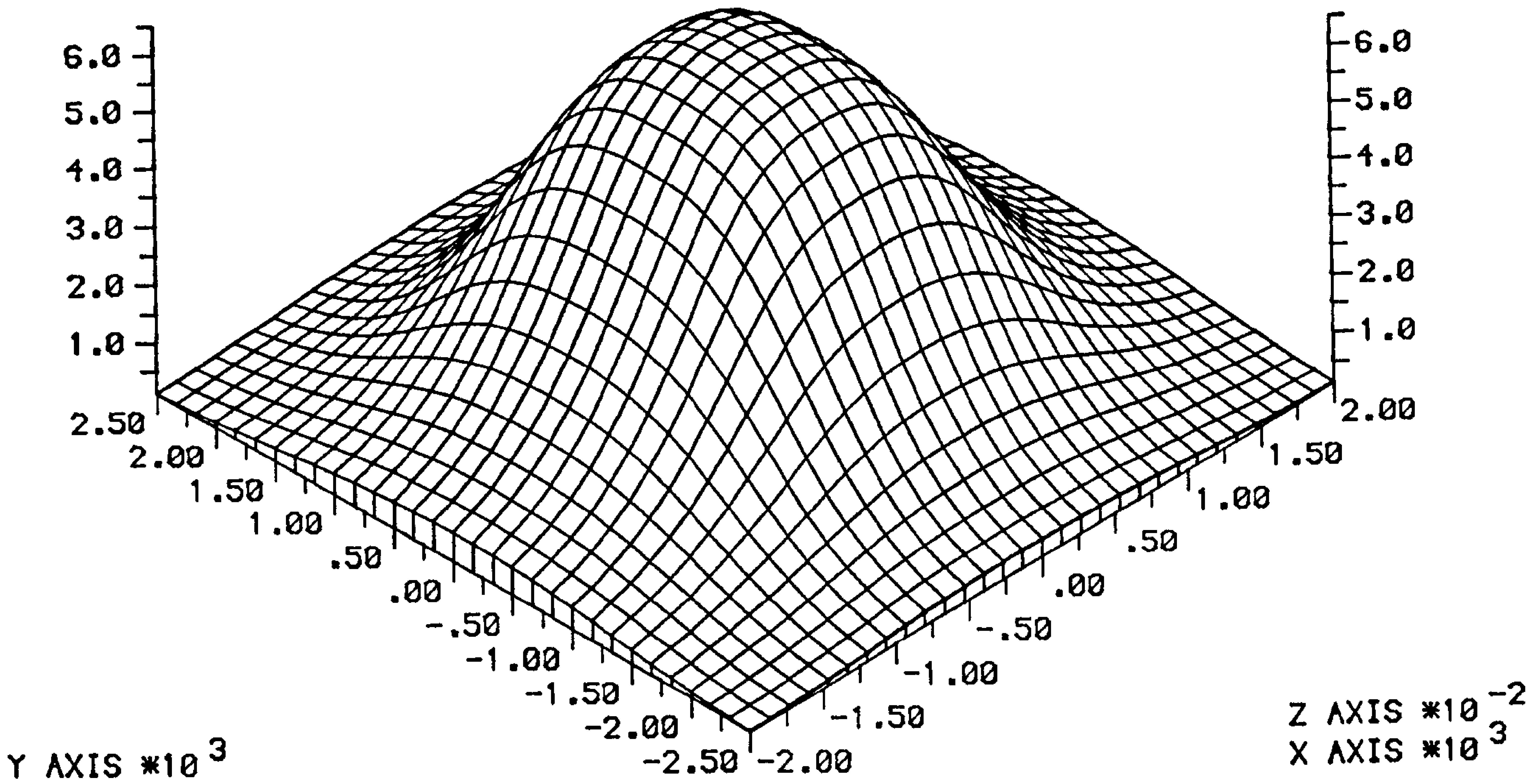


Effective depth 400 mm

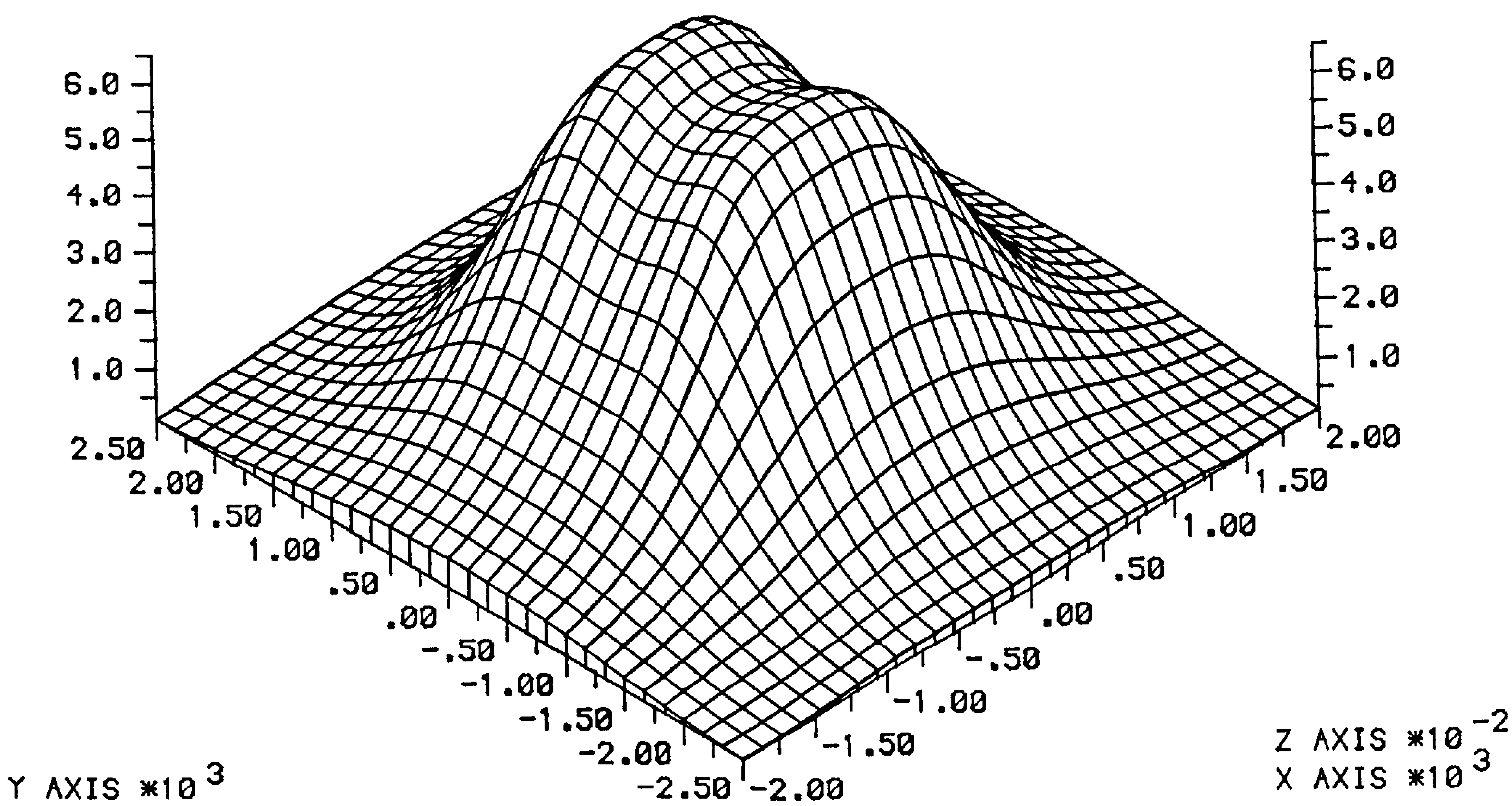


Effective depth 800 mm

Figure 6.10 (b) Vertical stress at different depths under a fully laden 40 ft. terminal trailer



Effective depth 1200 mm



Simplified Double Axle Representation
Effective depth 1200 mm

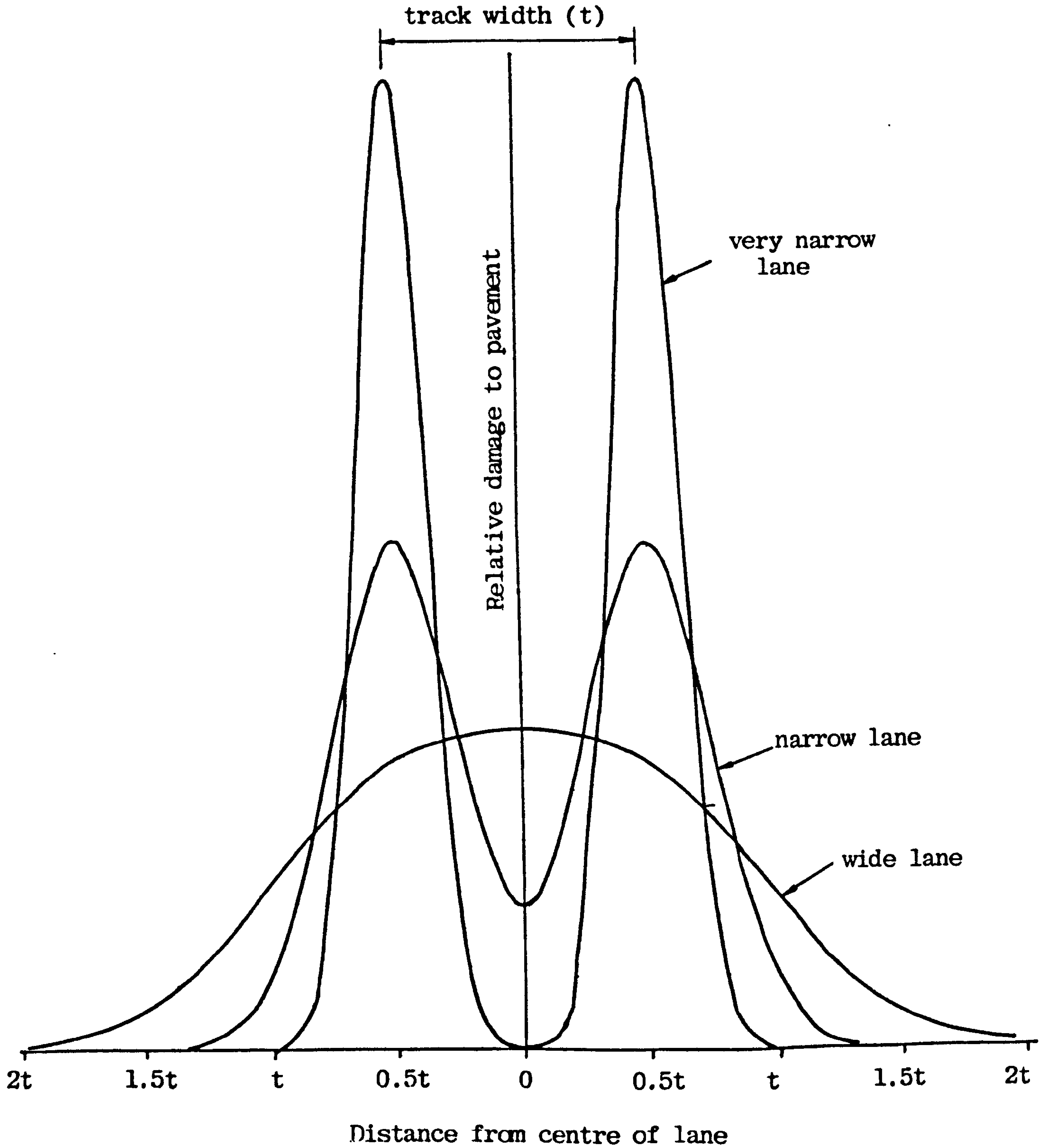
here are considered to be sufficiently accurate in the absence of more detailed information.

6.8. Lane Channelling.

Vehicles moving across a wide pavement do not follow exactly the same course. The variation, or wander, varies significantly with the type of pavement under consideration. Where vehicle movements are restricted with a lane width of the same order as the trackwidth, channelling is significant and, as in highway pavement design, each axle can be considered as a single repetition. For wide pavements, however, this assumption would result in significant over-design.

The concept of vehicle distribution across a pavement has been used in airfield design where different standard deviations have been applied for runway take-off, landing and taxiways. The standard deviation defines the distribution of wheels across a pavement and provides an estimate of the proportion of traffic covering a particular point. Figure 6.11 shows the transition from a very narrow lane, where confined rutting occurs, to a wide pavement where the maximum loading occurs at the centre-line.

Figure 6.11 Effect of increased lane width on pavement damage



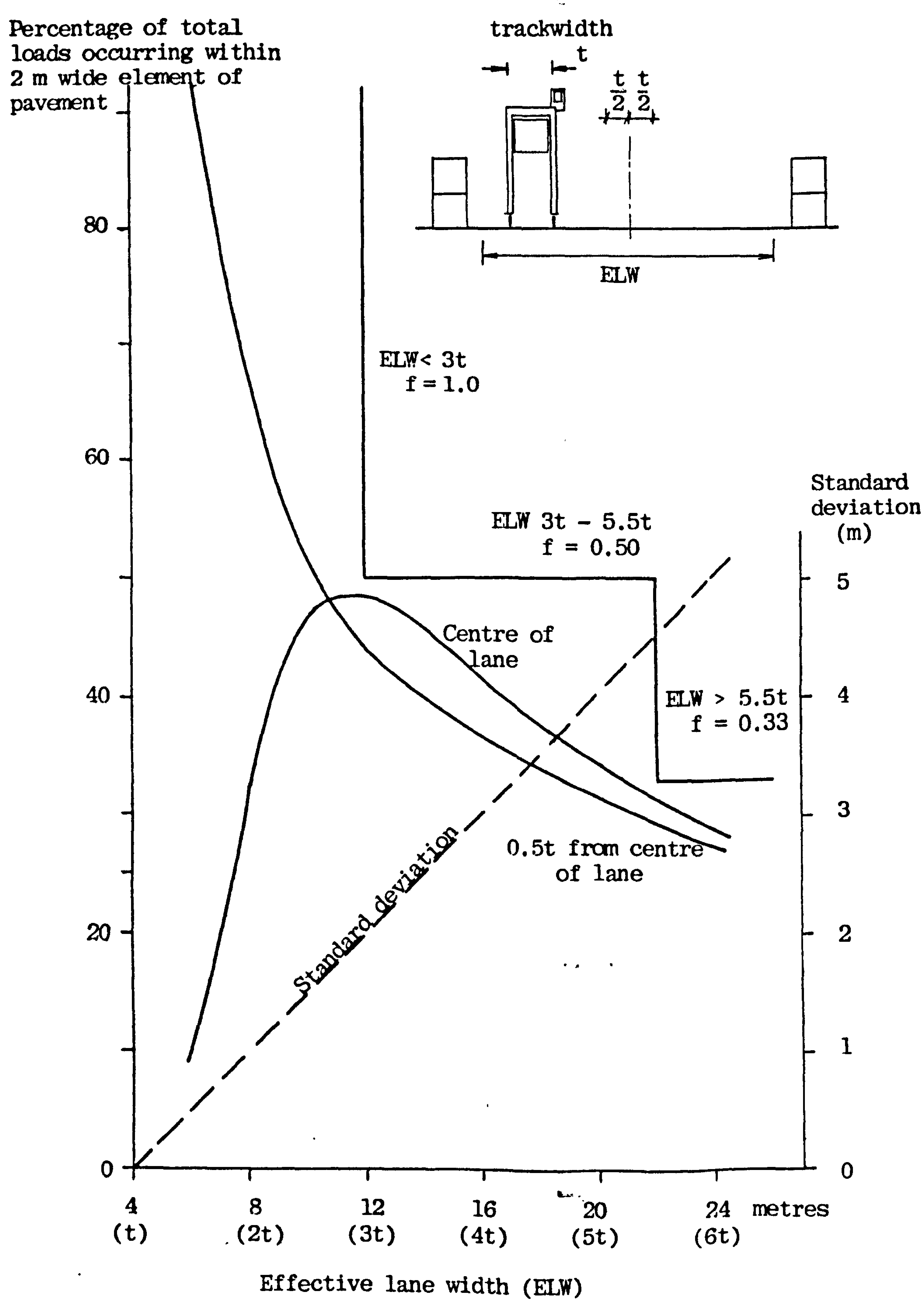
Standard deviation is defined by the effective lane width, that is, the area within which 95 % of vehicle movements occur. This can be estimated from minimum turning radii into the lane, or a fixed reduction of one or two metres at either side, or can be assumed to be the true pavement width.

The pavement is divided into a series of strips of such a width that any wheel passing within the bounds of a given strip will stress the pavement substructure sufficiently to constitute one load repetition. Ideal strip width depends on the actual pavement structure, but a value of 2 metres is used here.

Figure 6.12 shows the effect of increasing lane width on number of wheel loads passing over two critical points on the pavement, on the centre-line and at a distance of half trackwidth from it. This Figure is calculated for a typical straddle carrier and shows that the maximum effect changes from under the wheel to the centre-line when lane width is 11 metres, or just under three times the trackwidth.

Also plotted on Figure 6.12 is a design curve giving suggested reduction factors for wide pavements, the effective lane width is given in terms of the vehicles trackwidth. The factor varies from 1.0 for narrow lanes less than three times the trackwidth, down to 0.33 for wide lanes greater than five

Figure 6.12 Reduction factors for wide pavements



and a half times the trackwidth. The actual values to be used depend on the layout and operation of the handling area under consideration.

In straddle carrier operation, containers are stacked in long rows separated by narrow lanes approximately 1.75 m wide. In these lanes the wheels are restricted to exactly the same path and severe rutting results, particularly in asphaltic pavements. Often it is in this area that the worst rutting occurs as there is no "ironing out" effect. Thus, this criteria should be studied carefully during design by establishing the maximum number of wheel passes, and checking this against the strength of the pavement.

6.9. Damage Ratings for Container Handling Vehicles.

The example given above outlined the analysis of a typical, but fictitious, container handling vehicle. A similar analysis has been undertaken for all handling vehicles commonly used in British ports and the results are summarised in Table 6.5.

6.9.1. Straddle Carriers.

A wide range of straddle carriers is available, varying from

Table 6.5 PAWL damage ratings for container handling vehicles.

Vehicle	Unladen Weight	Load Capacity	PAWL Ratings Unladen	PAWL Ratings Average	PAWL Ratings Critical
STRADDLE CARRIERS ("")					
Valmet 3097	50.0	30.0	0.29	0.84	0.7
3597	54.0	35.0	0.39	1.04	0.4
4097	58.0	40.0	0.51	1.28	0.1
Nellen 902 & 903	52.0	35.0	0.52	1.42	0.6
Ferranti					
Karricon 3042 (′)	32.7	30.5	0.53	2.40	1.9
3043 (′)	35.6	30.5	0.72	2.97	2.6
Clarks Van Carrier					
830L (20 ft only)	54.79	30.5	0.78	.36	0.6
830L (20 + 40 ft)	54.79	30.5	0.78	.40	0.6
FRONT LIFT TRUCKS					
Hyster (Challenger)					
H520B (")	37.81	23.5	0.86	0.58	17.5
H620B (")	39.18	28.0	0.97	0.34	15.5
800	55.08	37.0	5.21	0.75	44.4
Lancer Boss (DD)					
52/48CH (")	27.45	23.5	0.26	4.86	8.3
62/48CH (")	40.67	28.0	1.14	1.43	18.6
82/48CH	50.95	37.0	2.68	9.56	29.1
Lansing Henley (Hermes)					
56/20CH (")	40.28	25.0	0.92	9.47	15.9
68/25CH (")	40.46	30.0	1.19	0.97	17.0
90/35CH	61.23	40.0	8.75	3.61	62.2
SIDELOADERS					
Lancer Boss 2000 (")	29.49	23.5	0.34	1.61	1.7
2500 (")	37.21	25.4	0.72	2.60	1.1
3500	47.64	36.8	2.25	6.72	4.3
Climax 560 (")	39.70	25.4	0.98	3.30	2.7

Table 6.5 (contd.)

PAWL damage ratings for container handling vehicles.

Vehicle	PAWL Ratings Unladen	Average	Critical
TRAILER / TRACTOR SYSTEMS			
Typical Highway Trailer	0.00	0.02	0.02
20 ft Terminal Trailer			
1 x 20 ft Container	0.01	0.10	0.14
2 x 20 ft Container	0.01	0.94	1.55
40 ft Terminal Trailer			
1 x 40 ft Container	0.01	0.05	0.02
2 x 20 ft Container	0.01	0.26	0.18
2 x 40 ft Container	0.01	0.39	0.25
4 x 20 ft Container	0.01	2.62	2.01

("") All Straddle Carriers are fitted with telescopic 40 ft frame.
(´) Straddle Carrier has only four wheels.
(") Handles only 20 ft containers or lightly loaded 40 ft containers

low profile models to gantry cranes spanning two or three rows of stacked containers. Although smaller vehicles are used for Ro-Ro operation and palletised loads, only the machines capable of stacking containers two or three high are included here.

Lift heights vary between models, allowing "one over one" or "one over two" operation, there are two principal groups; those with four wheels and those with eight wheels. The latter machines are generally 15 - 20 tonnes heavier, but since the load is carried on more axles, the overall damaging effect is lower, despite the greater number of loading repetitions.

6.9.2. Front Lift Trucks.

Front lift trucks (FLTs) are one of the most versatile forms of handling equipment and hence very widely used. Lifting capacities range from under 1 tonne to 42 tonnes, and most ports have a wide selection for various operations. The vehicles included in Table 6.5 are only those recommended for carrying and stacking of loaded containers. Since a very wide range of truck modifications are offered by various manufacturers, the data presented in Table 6.5 assumes that all FLTs are fitted with a 20 ft mast and a special toplift frame. As in the example, the toplifts are assumed to weigh 2 t. and 6 t. for a fixed 20 ft and a telescopic 40 ft frame respectively. All trucks capable of lifting loaded 40 ft units, i.e. a lifing capacity in excess of 35 tonnes, are fitted with telescopic frames.

Most FLTs have twin tyres on the front axle and single tyres on the rear axle. Some models are available with two extra tyres on the front axle, giving a six wheel drive axle, to improve load distribution, particularly for on-ship operation where deck strength must be considered. However, although this will reduce the stresses within the pavement surfacing, it will have minimal effect in lower layers of the pavement structure.

The results shown in Table 6.5 clearly demonstrates the damaging power of a vehicle carrying the load outside its wheelbase. Maximum axle loads reach 90 tonnes, and are likely to increase as larger vehicles are developed, giving very high PAWL ratings.

6.9.3. Sideloaders.

Sideloaders are an alternative form of handling used in areas where straddle carriers or FLTs are not ideally suited. These vehicles are capable of stacking containers three high, giving better random selection to stacks than straddle carriers, but at the cost of requiring wider access lanes. Since the load is carried evenly on two axles, wheel loading is lower than in the case of FLTs.

All vehicles included in Table 6.5 have two axles, but the number of tyres vary. In larger models, 4 tyres are spaced equidistantly across the axle, giving better load distribution. However, the vehicle is analysed with a single contact area at either end of each axle. Although these vehicles are not as heavy as straddle carriers, the lack of four axles to carry the load results in larger PAWL ratings.

6.9.4. Tractor and Trailer Units.

Numerous trailer systems have been developed for both specific and general use in a wide variety of heavy industries. Only those commonly used in ports, particularly Ro-Ro operation, are considered here, there are two groups :-

1) Normal road haulage trailer - here tyres are pneumatic and the vehicle is subject to a statutory maximum gross weight of 32.5 tonnes and a maximum axle load of 11.5 tonnes.

2) Terminal trailer systems for on site operation - here tyres are solid rubber and there is no upper

statutory limit on axle loading.

Normal highway trailers are common on short sea routes where Ro-Ro operation is prelavent. The trailers are usually handled by a special fleet of tractors with elevating fifth wheels. Axle loads are of the same magnitude as highway traffic, around 0.01 PAWLs or 1.0 "standard axle".

With terminal trailer systems the situation is slightly different, these can be pre-loaded to give high loading and unloading rates. With the development of faster handling times, trailers are being increasingly loaded with more than one container, i.e. two 20 ft units on a 20 ft trailer and two 40 ft or four 20 ft units on a 40 ft trailer. With a minimum number of closely spaced, small diameter wheels the damage ratings very quickly increase to values aproaching those of other handling vehicles. Trailers are generally the least damaging of all handling vehicles due to their low self-weight and uniform load distribution.

The data for trailers given in Table 6.5 assumes the distribution of container weights shown in Figure 6.1. The true distribution could well be different if other forms of cargo are handled, and a modified analysis may be necessary.

6.10. Static Loading.

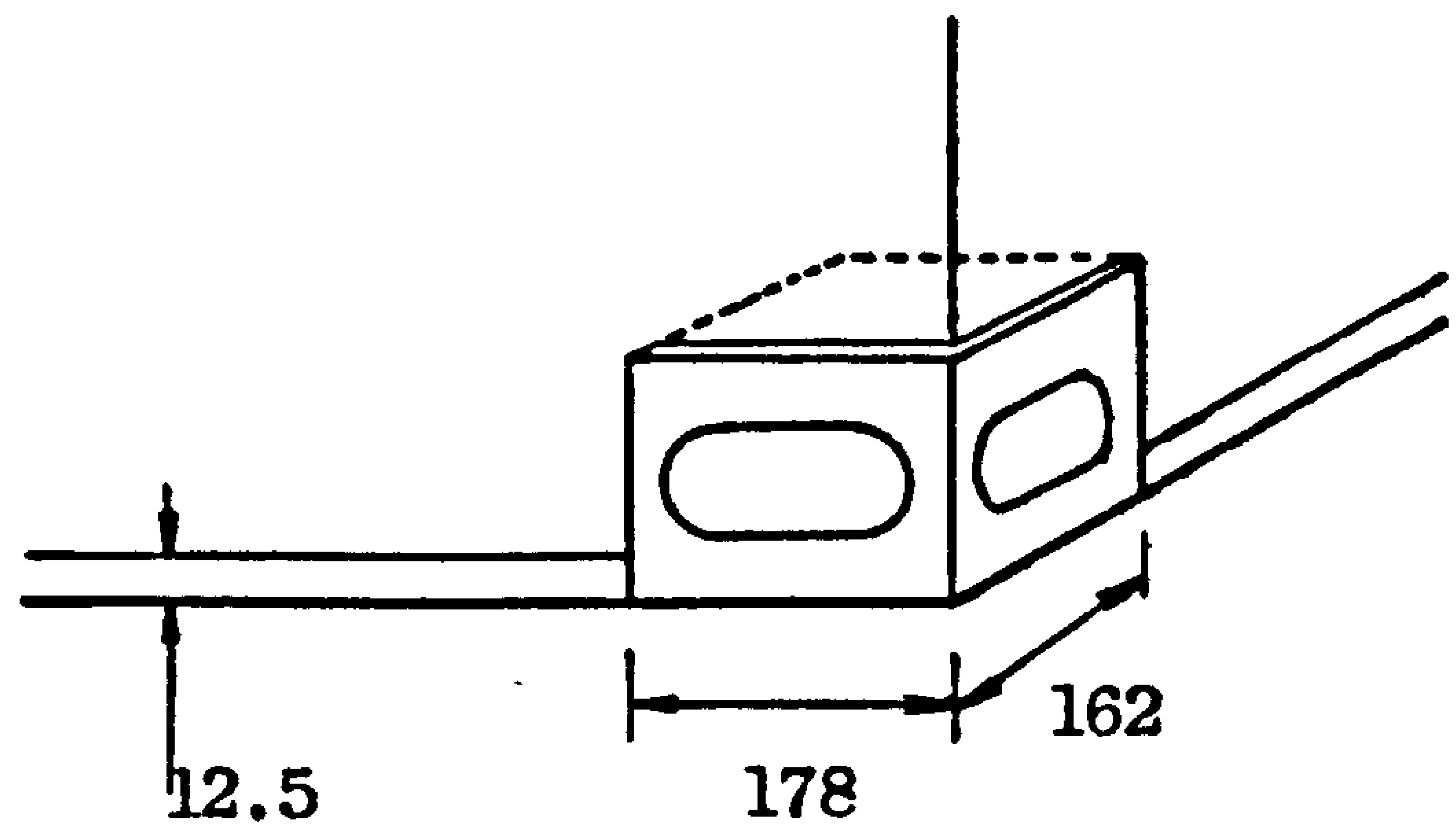
So far the analysis has covered only wheel loading, however a pavement also has to withstand very high static loads associated with cargo storage. There are three primary loads to be considered, as shown in Figure 6.13 :-

1) Container stacking
2) Trailer dolly wheels
3) Stabilising jacks of vehicles, e.g. sideloaders

Apart from ensuring the pavement has sufficient strength to carry the loading without overstressing the subgrade or any of the layers within the structure, the pavement surfacing must withstand these contact stresses. Existing pavements show that if a pavement can carry repetitive wheel loads, it can carry the static loads without structural failure. However, there have been many problems with surface disintegration under the action of corner castings and trailer dolly wheels. Once the surface has been broken, water can penetrate and frost action leads to further break up. It is a critical part of overall specification that the pavement surfacing is designed to withstand this loading.

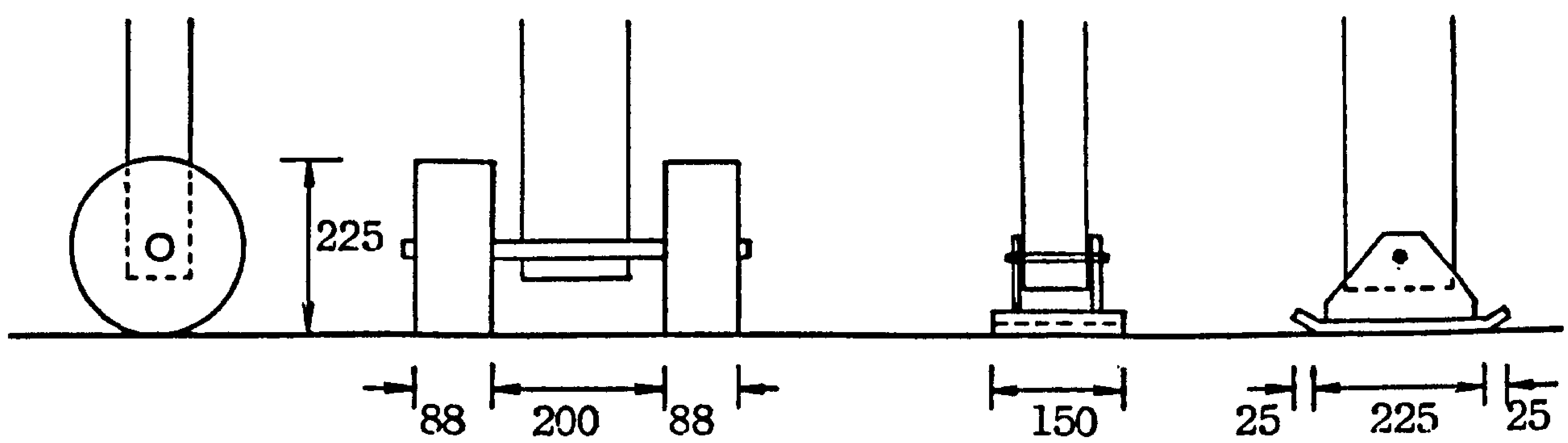
Figure 6.13 Static loading conditions on port pavements

(a)

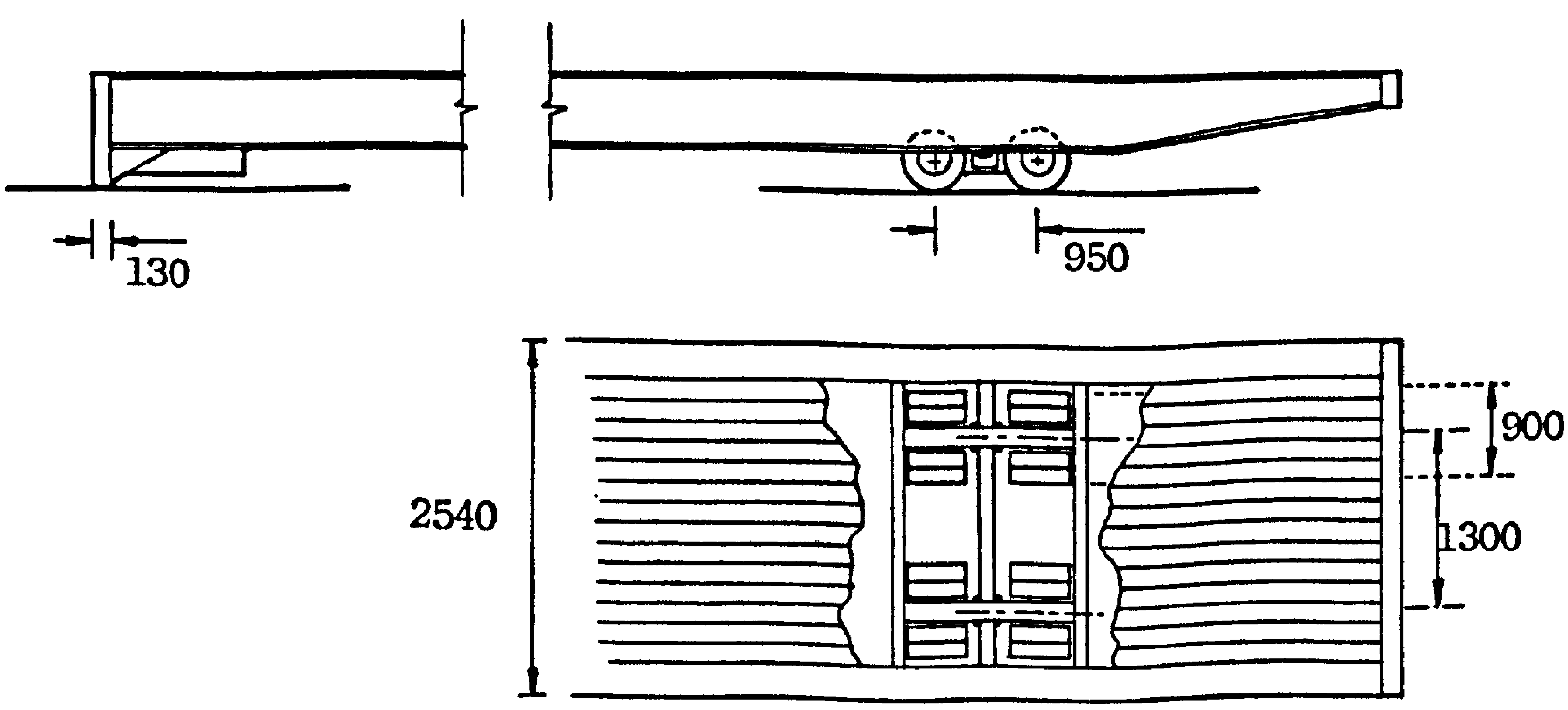


Container corner costing

(b) Trailer dolly wheels and plates



(c) Terminal trailer



6.10.1. Corner castings.

For intensive handling, containers are stacked in rows or blocks two, three, four or five high. Container corner castings protrude 12.5 mm below the bottom of the unit to allow for central deflection when fully laden. The plan dimensions of these castings is only 178 x 162 mm, giving a very high load concentration.

Table 6.6 gives maximum loads and stresses for most stacking arrangements. Since it is unlikely that fully laden 40 ft units will be stacked on top of one another, maximum gross weights are reduced as shown in column two of the Table. With the maximum reduction of 40 %, the 18.3 t design value is still above the average gross weight of 16.8 t.

6.10.2. Trailer dolly wheels.

Trailer dolly wheels are an old and notorious problem to the port engineer. When parked, the front of the trailer is supported by two posts, each having two 225 mm diameter by 88 mm wide wheels. With a fully laden trailer the load on these wheels can be as high as 14 tonnes which, assumming no penetration, gives an applied stress of 40 N/mm2 over two contact areas of 10 mm by 88 mm.

Table 6.6 Pavement Loads from Container Stacking.

Stacking Height	Reduction in Gross Weight %	Contact Stress N/mm2	Load on Pavement (tonnes) Stacking Arrangement Singly (a)	Rows (b)	Blocks (c)
1	0	2.59	7.62	15.24	30.48
2	10	4.67	13.72	27.73	54.86
3	20	6.23	18.24	36.58	73.15
4	30	7.27	21.34	42.72	85.34
5	40	7.78	22.86	45.72	91.44

(a) Contact area 178 x 162 mm.
(b) Contact area 356 x 162 mm.
(c) Contact area 356 x 324 mm.

Some trailers have pivot plates instead of steel dolly wheels. These give a much larger contact area, usually 150 mm wide by 225 mm long. With the same load carried by the plate the contact stress is only 2.0 N/mm2. This reduced contact stress is very apparent in existing pavements, where dolly wheels have left 50 mm indentations and plates have made no mark at all. This pavement maintenance problem would be reduced greatly if plates were fitted to all trailers in preference to dolly wheels.

6.10.3. Stabilising jacks.

In order to maintain stability when lifting a container, a sideloader has to lower a series of jacks. Because of the large cantilever effect under maximum reach, the jacks carry 70 % of combined vehicle weight and maximum load. The number of jacks varies between different models, generally two on smaller machines and four on 40 ft handling models. Although overall loads are large, contact stresses are modest, 0.4 to 1.3 N/mm2. These stresses are lower than those associated with dolly wheels, but the jacks are usually placed in exactly the same position and this has induced some damage to relatively soft surfacings.

6.11. References.

6.1. BARBER S.D. and KNAPTON J. Port pavement loading. The Dock and Harbour Authority, Vol. LIX, No. 701, April 1979. London. pp 379-386.

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6.4. ROAD RESEARCH LABORATORY A guide to the structural design of pavements for new roads. 3rd Edn. HMSO. London. 1970. Road Note 29.

6.5. HEUKELOM W. and KLOMP A.J.G. Consideration of calculated strains at various depths in connection with the stability of asphalt pavements. Proc. 2nd Int. Conf. on the Struct. Design of Asphalt Pavements, Ann Arbor, 1967, Univ. Michigan. pp 155-168.

CHAPTER SEVEN

DEVELOPMENT OF DESIGN PROGRAMS

Synopsis

This Chapter describes the development and structure of the analysis program "PEPAP" for multi-layered pavements. The analysis method is based on the simplified method developed by Ullidtz and Peattie and it is formulated to give full interactive operation. The construction of design charts based on this analysis is also described and the required data input specified.

7.1. Introduction.

The primary aim of this research program has been to develop and propose a suitable design method for heavily loaded pavements found in Port handling areas. In recognition of the fact that each design situation is unique, the design method must be capable of accommodating a wide variety of variables. It was also considered essential that it should be formulated in a way which preserves the engineers' freedom to specify material types and thicknesses for the structure; it would be wrong to remove this choice since local experience must always be the final mediator in selecting a form of construction.

The basic requirement, therefore, is for a simple presentation which can be followed easily, with an emphasis on the engineer following the design process. It should be possible to arrive at a series of designs which can then be taken forward for an economic appraisal. It was clear that graphical presentation would be desirable, but care was needed in handling the increased number of variables compared to those used in highway design. Graphical interpolation between adjacent pages was also to be kept to an absolute minimum, although this increased the number of charts needed to cover all plausible design criteria. In general, therefore, each design chart is constructed for a particular site.

From data presented in previous Chapters, four primary steps in the design process were isolated :-

(i) proportioning of strength and thickness of structural components of the pavement.

(ii) inclusion of assessment of wheel loading.

(iii) prevention of cracking in bound layers of the pavement.

(iv) prevention of over-stressing of the subgrade.

It will be noted that selection and design of the surfacing material is omitted since it is dictated by surface loading conditions and required surface durability, it having little effect on the design of structural elements within the pavement.

The construction of a suitable design chart requires the analysis of a large number of pavement structures, with incremental variations in each of the variables. The design charts presented here require a maximum of 512 different structures to be analysed, the results being stored and sorted before they are used to compile the chart. Since it is not practicable to publish a complete range of charts, the use of rigorous analysis methods is not possible since the computer resources required would too great. Therefore, the simplified analysis method developed by Ullidtz and Peattie has been adopted, Ref. 7.1. This method uses a principle of effective thicknesses with Boussinesq equations for a homogeneous elastic half-space. Correlation between this method and larger, more rigorous analyses has been checked by Ullidtz and Peattie and found to be satisfactory.

This simplified approach offers many advantages for design work. Since arithmetic calculations are straight-forward, computer usage is kept to a minimum and the programs can be made available for use on desk top computers. Each design chart can be compiled from basic input data within a few minutes.

The analysis programs were originally written for use on a Texas TI 59 programmable calculator. Due to limited capacity, the suite of programs had to be stored on several magnetic cards and full interactive design was not possible. Here, all programs have been combined in a single package, with various options and material choices included to give full interative operation. All programs are written in Fortran IV and run on a 256K GEC 4070 computer system, graphical routines use GINO, GINO-SURF and GINO-GRAF system libraries.

The analysis program, named 'PEPAP', is made up of a series of subroutines and can be used to analyse specific pavement structures in their own right. The program is described in detail in this Chapter.

In order to apply the basic analysis program for the contruction of design charts, a third program is required to collect relevant data and arrange it in a suitable format with incremental variations in the variables. This ensures a fast operation time since manual data entry is kept to an absolute minimum and, with the use of suitable prompting by the program, the operator has a clear understanding at each stage.

7.2. The 'PEPAP' Analysis Program.

This program incorporates all elements required to proportion the components of a pavement. It is written in the form of an analysis of a proposed structure, rather than the design of a structure from design criteria. However, an editing facility allows repeated modification until a satisfactory structure has been obtained. Ideally the program should be used at a VDU terminal, with hard copy of the results being taken as required.

The complete program is split into a series of 15 subroutines, each covering a particular segment of the analysis.

These are called by the main program in the order required by the operator. The program is fully interactive; all data input is in free format with suitable prompts displayed.

The program can be used to analyse structures of up to 5 layers, including the subgrade, and gives vertical stress, radial and vertical strains at each interface on a line through the centre of the loaded area. Surface deflection and an equivalent single elastic modulus for the complete structure are also calculated. It is also possible to calculate these parameters for points away from the centre-line of the load, thus making it possible to establish effects of stress overlap where wheels are closely spaced.

The program is shown in schematic form in Figure 7.1, the function of each subroutine is described in detail below :-

1) KEEP - specifies the type of output of results required.

2) LIFE - gives allowable strains for pavement materials against specified design life.

3) LOADIN - input of surface loading conditions.

4) STRUCT - input of all structural data for the pavement.

Figure 7.1 (a) Schematic Representation of PEPAP Analysis Program.

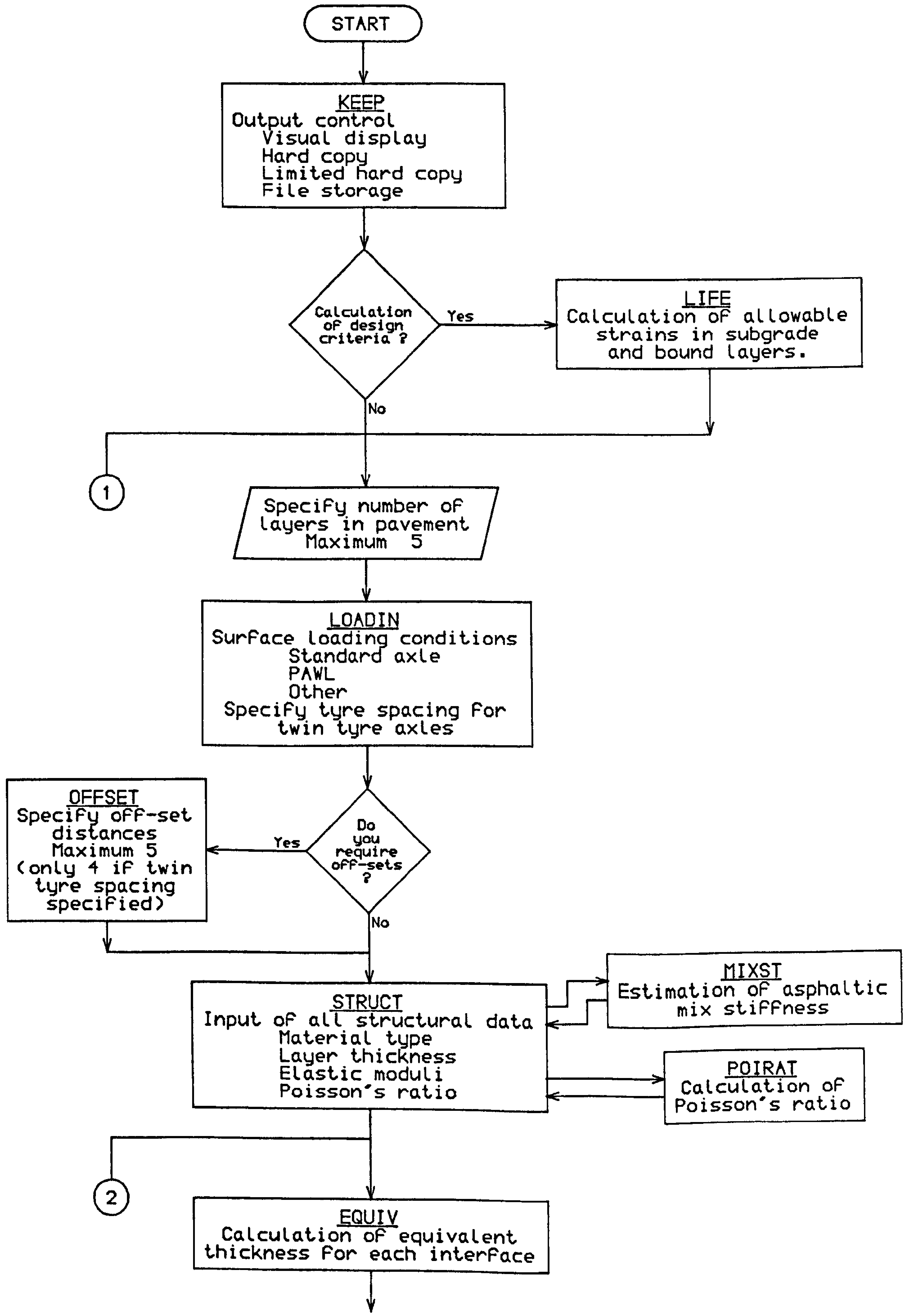
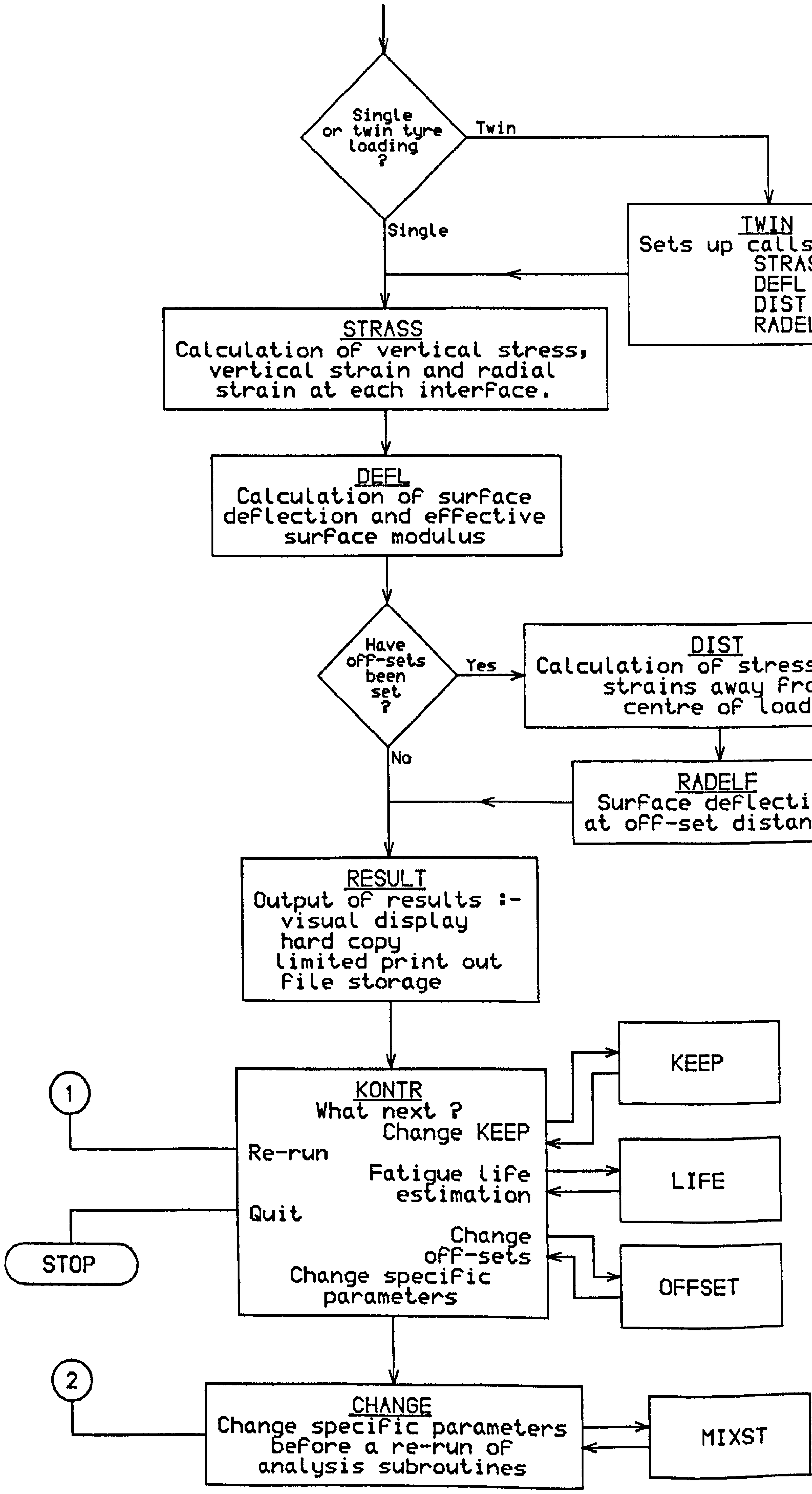


Figure 7.1 (b) Schematic Representation of PEPAP Analysis Program.



5) POIRAT - specification of Poisson's ratio for each layer.

6) MIXST - estimation of asphaltic mix stiffness.

7) OFFSET - stores required off-set distances.

8) EQUIV - calculation of equivalent thicknesses for each interface.

9) STRASS - calculation of stresses and strains at each interface.

10) DEFL - calculation of surface deflection and equivalent surface modulus.

11) TWIN - calls subroutines to accommodate dual wheel loading conditions.

12) DIST - calculation of stresses and strains at each interface at specified off-set distances.

13) RADELF - calculation of surface deflection and equivalent surface modulus at specified off-set distances.

14) RESULT - output of results in required format.

15) KONTR - calls the next operation required by the operator.

16) CHANGE - editing facility making it possible to change specific parameters before re-running the program.

7.2.1. KEEP - Control of output destination.

Output destination can be selected from one of six options: limited display on VDU; full output on line printer, with a full page for each structure; both these options together; a reserve judgement option, where hard copy selection is delayed until the results have been displayed on the screen; full output on the terminal; and limited print-out on line printer.

There is also another, hidden, option which is used when the program is called for the compilation of a design chart. Here all results are written out to file storage in a fixed format, thus forming a data file for the next program in the suite. This subroutine can also be recalled after the analysis of each structure, thus giving full flexibility in obtaining the required output.

7.2.2. LIFE - Calculation of allowable strains.

All materials within the pavement structure exhibit fatigue characteristics, thus allowable strains used for design are a function of the number of load repetitions, or design life. This subroutine estimates allowable strains for each of the materials used in pavement construction :-

7.2.2.a. Allowable tensile radial strain for an asphaltic mix is a function of mix proportions and bitumen characteristics. The inputs required are penetration and percentage volume of bitumen for the original asphaltic mix. The equations are shown in Figure 7.2 (a).

7.2.2.b. No specific guide lines have been developed relating tensile strain of a cement bound material to number of load repetitions to fracture. The following approximate method has been applied to a lean concrete material for this subroutine. Elastic modulus and compressive strength are related by the function given in Figure 7.2 (b). Williams, Ref. 7.2, also suggested that, under the action of repetitive loading, a fatigue life in excess of one million repetitions would be obtained if flexural stress was limited to 50 % of static strength; flexural strength of lean concrete being approximately one sixth (16.7 %) of

Figure 7.2 Equations for calculation of fatigue strains in paving materials

N	number of load repetitions
E_r	limiting radial tensile strain (microstrain)
E_v	limiting vertical compressive strain (microstrain)

(a) Asphaltic materials

Pen_i	initial penetration of bitumen
SP_i	initial softening point of bitumen (^{o}C)
W_{bit}	bitumen content by weight (%)
W_{agg}	aggregate content by weight (%)
V_v	volume of voids in mix (%)

Specific gravity of aggregate 2.70

Specific gravity of bitumen 1.02

$$W_{agg} = 100 - W_{bit}$$

$$V_{bit} = (100 - V_v)\,\frac{W_{bit}/1.02}{W_{bit}/1.02 + W_{agg}/2.70}$$

where V_{bit} is bitumen content by volume (%)

$$SP_i = 99.13 - 26.35 \log Pen_i + c$$

where c is a correction factor from American Standard Technical Measurement (ASTM) to British Standard (BS)

$$c = -1.5\ ^{o}C$$

$$E_r = \text{anti log}\ \frac{14.39 \times \log V_{bit} + 24.2 \times \log SP_i - 40.7 - \log N}{5.13 \times \log V_{bit} + 8.63 \times \log SP_i - 15.8}$$

Figure 7.2 (contd.)

Equations for calculation of fatigue strains in paving materials

(b) Lean Concrete

f_e compressive strength (N/mm^2)

f_f flexural strength (N/mm^2)

E elastic modulus (N/mm^2)

$$E = 16{,}800 \times f_c^{0\cdot25} \qquad f_c > 7$$

$$E = 4 \times f_c \qquad f_c < 7$$

$$f_f = f_c/6$$

$$\frac{\sigma}{E} = \frac{E^{1\cdot02}}{0.994}$$

Fatigue life is 10^6 repetitions if flexural stress is limited to 50% of static strength

$$E_r = \frac{f_c \times 993{,}500}{6 \times E^{1\cdot022}} \times N^{-0\cdot0502}$$

(c) Subgrade

$$E_v = \frac{21{,}600}{N^{0\cdot28}}$$

compressive strength.

Two further relationships are required: stress against strain for lean concrete, and limiting strain against design life. The former was derived from design charts in the Shell Pavement Design Manual, Ref. 7.3, using a curve fitting program to establish the constants, Ref. 7.4. The latter relationship is derived from the two fixed points, one million repetitions at 50 % ultimate strain and one repetition at 100 % ultimate strain, assuming a similar linear logarithmic relationship to that adopted for fatigue characterisation of other paving materials. The values of function constants are also given in Figure 7.2 (b).

7.2.2.c. Limiting vertical compressive strain for the subgrade is expressed as a function of design life, Brown, Ref. 7.5; as shown in Figure 7.2 (c).

7.2.3. LOADIN - Surface Loading Conditions.

The surface contact area is assumed to be circular with a contact stress equal to the tyre pressure. Three methods of specifying surface loading are offered :-

(i) Standard Axle - wheel load is specified as 40 kN and tyre pressure as 0.5 N/mm2.

(ii) Port Area Wheel Load (PAWL) - from the PAWL rating entered, the actual wheel load is calculated and displayed; tyre pressure is specified as 0.8 N/mm2.

(iii) Other - wheel load, kN, and tyre pressure, N/mm2, are entered by the operator.

All three loading conditions can be treated as a single or a twin tyre axle, the centre to centre tyre spacing is specified by the operator in millimetres. A spacing of zero, 0.0, assumes a single contact area whilst higher values, through the use of subroutine "TWIN", allows for interaction of twin tyres.

7.2.4. STRUCT - Input of Structural Data.

This subroutine collects the basic properties for each of the pavement layers in turn, four parameters are required for each :-

(i) Material type

(ii) Layer thickness (mm)

(iii) Layer strength or stiffness (elastic modulus, N/mm2)

(iv) Poisson's ratio

For the surfacing material, the first layer, two materials are offered, asphaltic and block paving. With the latter, 80 mm concrete blocks, thickness is specified at 130 mm (blocks plus 50 mm compacted bedding sand) and the elastic modulus is specified as 3500 N/mm2. In all cases where an asphaltic material is selected, elastic modulus may be specified directly or calculated from mix proportions through "MIXST".

For all other layers, except the subgrade, the following materials are offered :-

asphaltic

cement bound granular material (CBGM)

lean concrete

soil cement

granular

Also, material type may be "unspecified", in which case all parameters are specified by the operator.

If the penultimate layer, i.e. the sub-base, is granular, then the stiffness of that layer, Eg, is calculated from the thickness of that layer, Hg, and subgrade stiffness, En :-

$$Eg = k \times En$$

where $k = 0.2 \times Hg ** 0.45$ $\quad 2 < k < 4$

Subgrade stiffness is assumed to be equal to ten times the CBR (N/mm2), and, naturally, layer thickness is not specified. For all materials, Poisson's ratio is specified through the subroutine "POIRAT".

7.2.5. POIRAT - Poisson's ratio.

Poisson's ratio for each layer are specified as follows :-

asphaltic	0.4
lean concrete	0.2
CBGM	0.2
block surfacing	0.2

Poisson's ratio, U, for granular and soil cement materials and the subgrade are calculated from :-

$$U = 0.82 - 0.10 \times LOGe(E) \qquad 0 < U < 0.5$$

where E is the stiffness or elastic modulus.

7.2.6. MIXST - Asphalt mix stiffness estimation.

The stiffness, or elastic modulus, of an asphaltic bound material is calculated from the basic mix proportions and material properties. The calculations are based on the Shell ~~Francaise~~ Amsterdam JK method, and the equations are given in full in Figure 7.3. The result is displayed and returned to the main program.

7.2.7. OFFSET - Storing of off-set distances.

Stresses and strains can be calculated for a series of off-set distances, this subroutine stores up to five values to reduce manual data entry in a long series of calculations. If twin tyre loading has been specified only four off-sets can be stored. Off-set distances are specified in millimetres. This subroutine can be recalled and the values modified from KONTR after each run.

7.2.8. EQUIV - Calculation of equivalent thicknesses.

This, and the next four subroutines, use the above data to calculate the stresses and strains for each interface. The layers above that interface are converted to a single layer of equivalent stiffness, but with an elastic modulus equal to that of the material below the interface. Thus, using the properties

Figure 7.3 Equations for calculation of bituminous mix stiffness

Pen_i	initial penetration of bitumen
Pen_r	penetration of bitumen after laying
SP	softening point (^oC)
W_{bit}	bitumen concent by weight (%)
Wagg	aggregate content by weight(%)
V_{bit}	bitumen content by volume (fraction)
Vagg	aggregate content by volume (fraction)
V_v	volume voids in mix (fraction)
v	vehicle speed
T	temperature (^oC)
S_B	stiffness of bitumen (N/mm^2)
S_m	stiffness of mix (N/mm^2)

Specific gravity of aggregate 2.70

Specific gravity of bitumen 1.02

Time of loading, $t = 1/v$ secs. $0.01 < t < 0.1$

Figure 7.3 (contd.)

Equations for calculation of bituminous mix stiffness

For bitumen:-

$$Pen_r = 0.65 \times Pen_i$$

$$SP = 99.13 - 26.35 \log (Pen_r)$$

$$PI = \frac{20 \times SP + 500 \times \log (Pen_r) - 1951.55}{SP - 50 \times \log (Pen_r) + 120.15}$$

$$S_B = 1.157 \times 10^{-7} \times t^{-0.368} \times e^{-PI} \times (SP - T)^5$$

For bituminous mix:-

$$S_m = S_B \left[1 + \frac{2.5 \times C_v'}{n(1 - C_v')}\right]^n$$

where

$$n = 0.83 \log \left[\frac{40000}{S_B}\right]$$

$$C_v' = \frac{C_v}{0.97 + V_r}$$

$$C_v = \frac{V_{agg}}{V_{agg} + V_{bit}} = \frac{W_{agg}/2.70}{W_{agg}/2.70 + W_{bit}/1.02}$$

of the lower layer, elastic modulus with the equivalent modulus, the Boussinesq equations for a homgeneous half space are applied. The notation used in all following equations is given in Figure 7.4.

The calculation of equivalent thicknesses for each interface uses the summations shown in Figure 7.5. The first summation is applied to pavements where layer stiffnesses decrease down the structure. A correction factor, f, is applied to calculated equivalent thicknesses as follows :-

$f = 1.0$ for the first interface

$f = 0.8$ for all other interfaces

For pavements where the ratio of elastic moduli across an interface is less than 2.0 (e.g. for a pavement with a cement bound base material), the second equation for equivalent thickness is used; f' is another correction factor, equal to 0.9 for all interfaces. It should be noted that for any one pavement structure, equivalent thicknesses are calculated from only one of the two equations, they are not mixed.

7.2.9. STRASS Stress / strain calculations.

This subroutine calculates radial strain, vertical strain and vertical stress at each interface below the centre of the

Figure 7.4 Notation for use in structural equations

h_i	thickness of i'th layer (mm)
E_i	elastic modulus i'th layer (N/mm)
μ_i	Poisson's ratio of i'th layer
h_{en}	equivalent thickness to interface (mm)
W	wheel load (N)
σ_0	contact pressure, or tyre pressure (N/mm^2)
R	radius of circular contact area (mm)
N	number of layers in pavement
σ_z	vertical stress (N/mm^2)
t_z	vertical strain (microstrain)
E_r	radial strain (microstrain)
δ_0	surface deflection under centre of load (mm)
E_0	equivalent surface modulus under centre of load (N/mm^2)
y	horizontal offset distance from centre of load (mm)
Z_i	corrected depth to i'th interface at an offset of y (mm)
σ_{zy}	vertical stress at interface at offset y (N/mm^2)
E_{zy}	vertical strain at interface at offset y (microstrain)
E_{ty}	tangential strain at interface at offset y (microstrain)
δ_y	surface deflection at offset y (mm)
E_y	equivalent surface modulus at offset y (N/mm^2)

Figure 7.5 Equations for calculation of equivalent thicknesses

(a) Summation for pavement structures where E_i/E_{i+i} is greater than 2.0

$$h_{en} = f \sum_{i=i}^{n-1} \left[h_i \cdot \sqrt[3]{\frac{E_i}{E_{i+i}}} \right]$$

where f is a correction factor as follows:-

f = 1.0 for first interface

f = 0.8 for all other interfaces

(b) Summation for pavement structures were E_i/E_{i+i} is less than 2.0

$$h_{en} = 0.5 \times h_{e,n-1} \left[1 + \sqrt[3]{\frac{E_{n-1}}{E_n}} \right] + f' \times h_{n-1} \sqrt[3]{\frac{E_{n-1}}{E_n}}$$

where f′ is another correction factor equal to 0.9.

Figure 7.6 Equations for calculation of stresses and strains under centre of load

Vertical stress at n'th interface

$$\sigma_{zn} = \left[1 - \left(\left(\frac{R}{h_{en}}\right)^2 + 1\right)^{-3/2}\right] \sigma_0 \quad N/mm^2$$

Vertical strain at n'th interface

$$E_{zn} = \frac{1}{E_{n+1}} \left[\left(\frac{h_{en}}{R} \times K^{-3/2}\right) - \left(\frac{h_{en}}{R} \times K^{-1/2} - 1\right) \times (1 - 2\mu_{n+1}) \right]$$

$$\times \sigma_0 \times (1 + \mu_{n+1}) \times 10^6 \text{ microstrain}$$

where

$$k = \left[\left(\frac{h_{en}}{R}\right)^2 + 1\right]$$

Radial strain at n'th interface

$$E_{rn} = \left[\frac{(\sigma_{zn} - E_{zn} \times E_{n+1}) \times (1 - \mu_n)}{2\mu_n} - \mu_n \times \sigma_{zn}\right] \times 10^6 \text{ microstrain}$$

NOTE:- E_{rn} is calculated using the Poisson's ratio of the material above the interface.

loaded area using the equations shown in Figure 7.6. The elastic modulus and Poisson's ratio used here are those of the material below the interface, with the exception of radial strain where the Poisson's ratio of the material above the interface is used. Stresses are given in N/mm2 and strains in microstrain.

In a three dimensional stress analysis, three stress components have to be considered; vertical, radial and tangential. The vertical component is, in this case, related to the subgrade, whilst tangential and radial strains are related to tensile cracking of bound layers. Under the centre of the load, the radial and tangential components are equal, but when off-set distances are considered, it is found that tangential strain is the critical component; the analysis is adjusted accordingly. For clarity, however, the term "radial" strain has been adopted in all text output.

7.2.10. DEFL - Surface deflections.

This subroutine sums the elastic compaction of each layer to give the surface deflection under the centre of the wheel load. It also calculates an equivalent elastic modulus for an homogeneous half-space which would give the same surface deflection as the pavement structure. The equations used are shown in Figure 7.7.

Figure 7.7 Equations for calculation of surface deflections under centre of load

Surface deflections

$$\delta_0 = \frac{C(1 + \mu_N) R\sigma_0}{E_N} + \sum_{i=1}^{N-1} \frac{(K - K')(1 + \mu_i) R\sigma_0}{E_i}$$

where,

$$C = \left[\sqrt{\left(\frac{h_{eN-1}}{R}\right)^2 + 1} - \frac{h_{eN-1}}{R}\right] x\ (1 - 2\mu_N) + \left[\sqrt{\left(\frac{h_{eN-1}}{R}\right)^2 + 1}\right]^{-1}$$

and

$$K = \left[\sqrt{\left(\frac{h_{ei-1}}{R}\right)^2 + 1} - \frac{h_{ei-1}}{R}\right] x\ (1 - 2\mu_i) + \left[\sqrt{\left(\frac{h_{ei-1}}{R}\right)^2 + 1}\right]^{-1}$$

and

$$K' = \left[\sqrt{\left(\frac{h_{ei-1}}{R}\right)^2 + 1} - \left(\frac{h_{ei-1}+h_i}{R}\right)\right] x\ (1 - 2\ _i) + \left[\sqrt{\left(\frac{h_{ei-1} + h_i}{R}\right)^2 + 1}\right]^{-1}$$

Equivalent stiffness

$$E_0' = \frac{2\ R\ \sigma_0\ (1 - \mu_N^2)}{\delta_0}$$

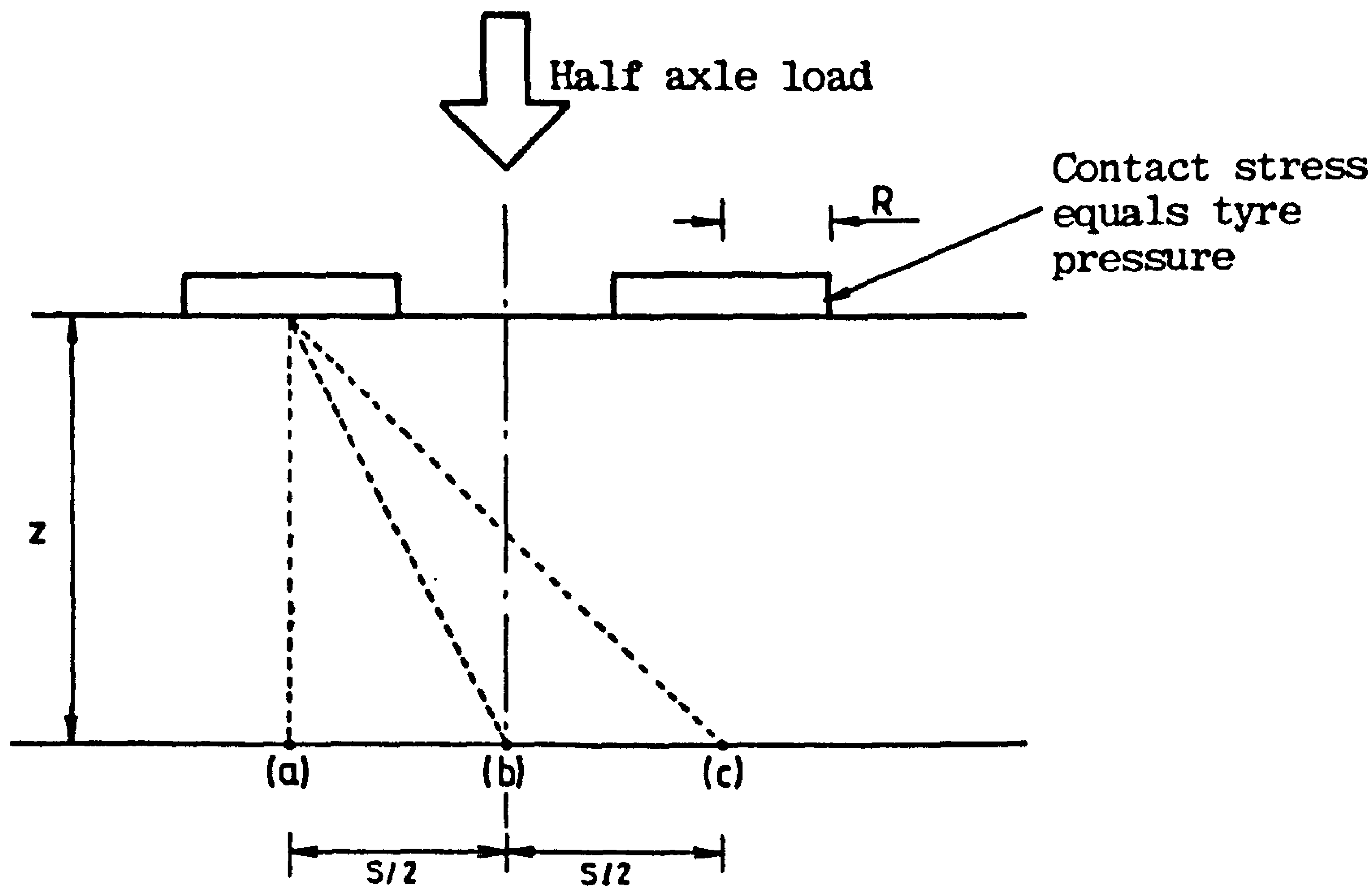
7.2.11. TWIN - Twin tyre loading.

The majority of heavily loaded axles are made up of twin tyres on each axle and this subroutine, by use of subroutines "DIST" and "RADELF", calculates stresses under a twin tyred axle. Figure 7.8 shows the loading condition under consideration. In this case two points are of interest: stresses and strains under the centre of the gear assembly, b, and under the centre of either of the single tyres, a or c.

The stresses, strains and deflections for a single tyre (i.e. half wheel load) are calculated for the centre-line and at off-sets of S and 0.5 x S, where S is the centre to centre tyre spacing. Then, by doubling the results for point b, and summing those for a and c, the results for both critical points are calculated and given in the output.

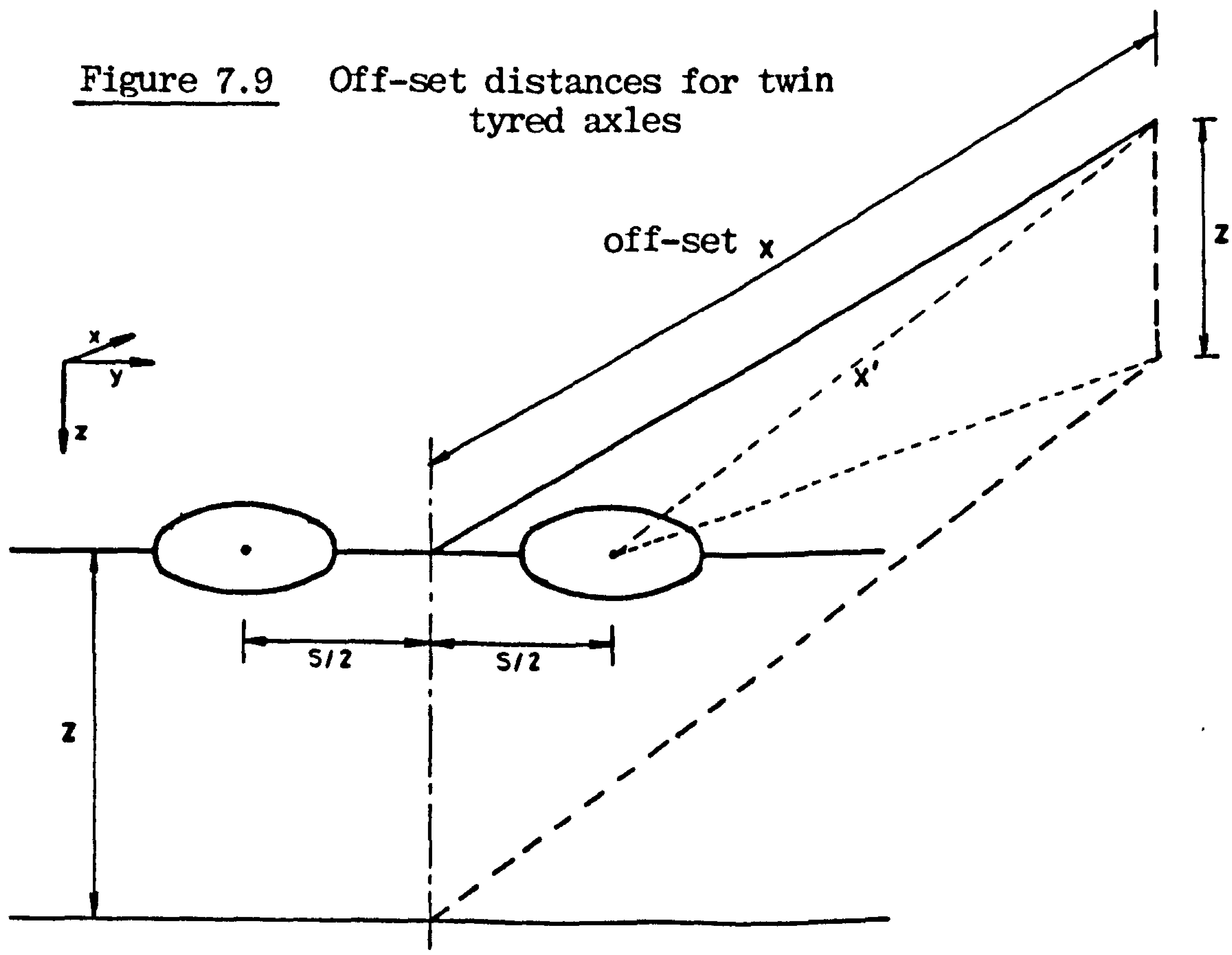
A similar operation is applied for off-set distances required in the line of travel of the axle, Figure 7.9. The specified off-set, x, is converted to true distance between the required point and the centre of one of the twin tyres, x', before "DIST" and "RADELF" are called. The results for the single tyre at a quarter axle load are then doubled to give the required results.

Figure 7.8 Geometry of twin tyred axle loading conditions



S = twin tyre spacing

Figure 7.9 Off-set distances for twin tyred axles



7.2.12. DIST and RADELF - Off-set distances.

These two subroutines calculate stresses, strains and deflections for points away from the centre-line of the loaded area. Up to five off-sets can be accommodated in each run of the program, the distances being recalled from subroutine "OFFSET". The equations used by both these subroutines are given in Figure 7.10.

7.2.13. RESULT - Output of results.

The form of output is specified by subroutine "KEEP". The final print-out for a typical four layer structure with twin tyre loading conditions is shown in Figure 7.11. The temporary display of results on a VDU is presented in a condensed format because of limited space.

7.2.14. KONTR - Program control.

Having completed the analysis of the pavement structure, the program can be re-run from various points; i.e. with or without the same number of layers and loading conditions. The subroutines KEEP, OFFSET, LIFE and CHANGE may also be called from this point.

Figure 7.10 Equations for stresses, strains and deflections for points away from centre of load

(a) Stresses and strains

$$Z_i' = h_{ei} + \frac{0.6\ R^2}{h_{ei}}$$

$$V = \sqrt{Z_i'^2 + y^2}$$

$$\theta = \tan^{-1} (y/Z_i)$$

Vertical stress

$$\sigma_{zy} = 3/2 \left(\frac{R}{V}\right)^2 \sigma_0 \cos\theta$$

Vertical strain

$$E_{zy} = (3\cos^3\theta - 2\mu_{i+1}\cos\theta)\left(\frac{R}{V}\right)^2 \frac{\sigma_0}{2} \frac{(1 + \mu_{i+1})}{E_{i+1}} \times 10^6$$

Tangential strain

$$t_{ty} = \left[\frac{1 - 2\mu_i}{1 + \cos} - \cos\theta\right] \times \frac{R}{V} \frac{\sigma_0}{2} \frac{(1 + \mu_i)}{E_{i+1}} \times 10^6$$

Figure 7.10 (contd.)

Equations for stresses, strains and deflections
for points away from centre of load

(b) Deflections

Surface deflection

$$\delta_y = \frac{K_N}{E_N} + \sum_{i=1}^{N-1} \left(\frac{K'}{E_i} - \frac{K''}{E_i} \right)$$

where

$$K = \left[\left(\frac{X}{Y} \right)^2 + 2(1 - \mu_i) \right] \frac{(1 + \mu_i)\sigma_0 R}{2 \left[\frac{0.6R^2}{Y} + Y \right]}$$

$$Y = \sqrt{x^2 + y}$$

x takes the following values:-

for K' $\quad x = h_{e,i-1}$

for K'' $\quad x = h_{e,i-1} + h_i$

for K_N $\quad x = h_{eN}$

Equivalent surface modulus

$$E_y' = \frac{2(1 - \mu_N)^2 \sigma_0 R^2}{2\delta_0 y}$$

Figure 7.11 Typical output of PEPAP results.

Original specification, 1969 - 1970.

```
      WLOAD= 208.9 kN
      PAWL = 8.00
      TPRES= 0.80  N/MM2
      RAD  =  204. MM
      SPACE=  600. MM

         VVVV  VVVV
 -------------------------

LAYER NO.1 ASPHALTIC
                                                    OFFSET DISTANCE
THICKNESS =  240.  MM
MODULUS   =  2300. N/MM2                                     300.
POISS RAT =  0.40
                          RADIAL STRAIN =  -301.                -298.
 -------------------------
                          VERT STRESS   = 0.1947 N/MM2       0.2579
                          VERT STRAIN   =   450.                 644.
LAYER NO.2 ASPHALTIC

THICKNESS =  230.  MM
MODULUS   =   375. N/MM2
POISS RAT =  0.40
                          RADIAL STRAIN =  -231.                -216.
 -------------------------
                          VERT STRESS   = 0.1400 N/MM2       0.1354
                          VERT STRAIN   =   468.                 454.
LAYER NO.3 GRANULAR

THICKNESS =  150.  MM
MODULUS   =   300. N/MM2
POISS RAT =  0.24
                          RADIAL STRAIN =  -201.                -177.
 -------------------------
                          VERT STRESS   = 0.0924 N/MM2       0.0840
                          VERT STRAIN   =   623.                 560.
       SUBGRADE
MODULUS   =   150. N/MM2
POISS RAT =  0.31

                      SURFACE DELECTION  = 0.8112  MM        0.8644
                      EFF SURFACE MODULUS=   515. N/MM2        484.
                      EFF POISSONS RATIO =  0.31
```

7.2.15. CHANGE - Editing facility.

This subroutine makes it possible to change specific parameters in a program without re-entering all data. The current pavement structure, including wheel loading conditions, is displayed on the screen in the format shown in Figure 7.12. The element to be changed, e.g. material type, layer thickness, elastic modulus or Poisson's ratio, is selected and the program then prompts for layer number and replacement value.

The subroutine has a left to right priority across the material characteristics; e.g. changing material type to asphaltic will also change Poisson's ratio to 0.4. Thus, if a different ratio is required it should be specified after material type and elastic modulus. The characterisation of a granular sub-base as a function of layer thickness and subgrade strength is also maintained.

In certain cases a single Poisson's ratio for all materials is required. This can be specified by returning a layer number greater than the number of layers in the structure, the value then entered is applied to all pavement layers.

The loading conditions, wheel load, tyre pressure and twin

Figure 7.12 Editing facility within 'PEPAP' program.

```
      WHAT NEXT ::
        1  COMPLETE RE-RUN
        2  RE-RUN WITH SAME NLAY
        3  RE-RUN/SAME NLAY AND LOADING
        4  REPLACE SPECIFIC  PARAMETERS
        5  CHANGE K80
        6  CALCULATE PERMISSIBLE STRAINS
        9  QUIT PROGRAM
> 4

           H         MATE       EM         POR

      2   100.        A        4300.      0.40
      2   200.        G         217.      0.27
      3               E         100.      0.35

         WLOAD       SPACE     TPRES      QUIT
           173.7     550.       0.80     CHANGE

      ENTER ITEM REQUIRED  E.G. H/E/W/D ; USE  Q  TO QUIT
> M
      WHICH LAYER?
> 1
      WHAT MATERIAL TYPE FOR SURFACING
                ASPHALTIC
                BLOCK PAVING (80 MM)
                OTHER
> B

      ENTER ITEM REQUIRED  E.G. H,E/W/D ; USE  Q  TO QUIT
> H
      WHICH LAYER?
> 2
      ENTER NEW VALUE
> 300
           H         MATE       EM         POR

      1   120.        B       15000.      0.20
      3   200.        G         260.      0.25
      3               E         100.      0.35

         WLOAD       SPACE     TPRES      QUIT
           173.7     550.       0.80     CHANGE

      ENTER ITEM REQUIRED  E.G. H/E,W,D ; USE  Q  TO QUIT
> Q
```

Return to main program.

tyre spacing can be modified in a similar manner. When all changes have been made "Quit" will terminate the subroutine and return to the main program.

7.3. 'CHART' - The Design Chart.

Each design chart is made up of four parts :-

(i) Base Thickness Chart

(ii) PAWL Chart

(iii) Base Tension Chart

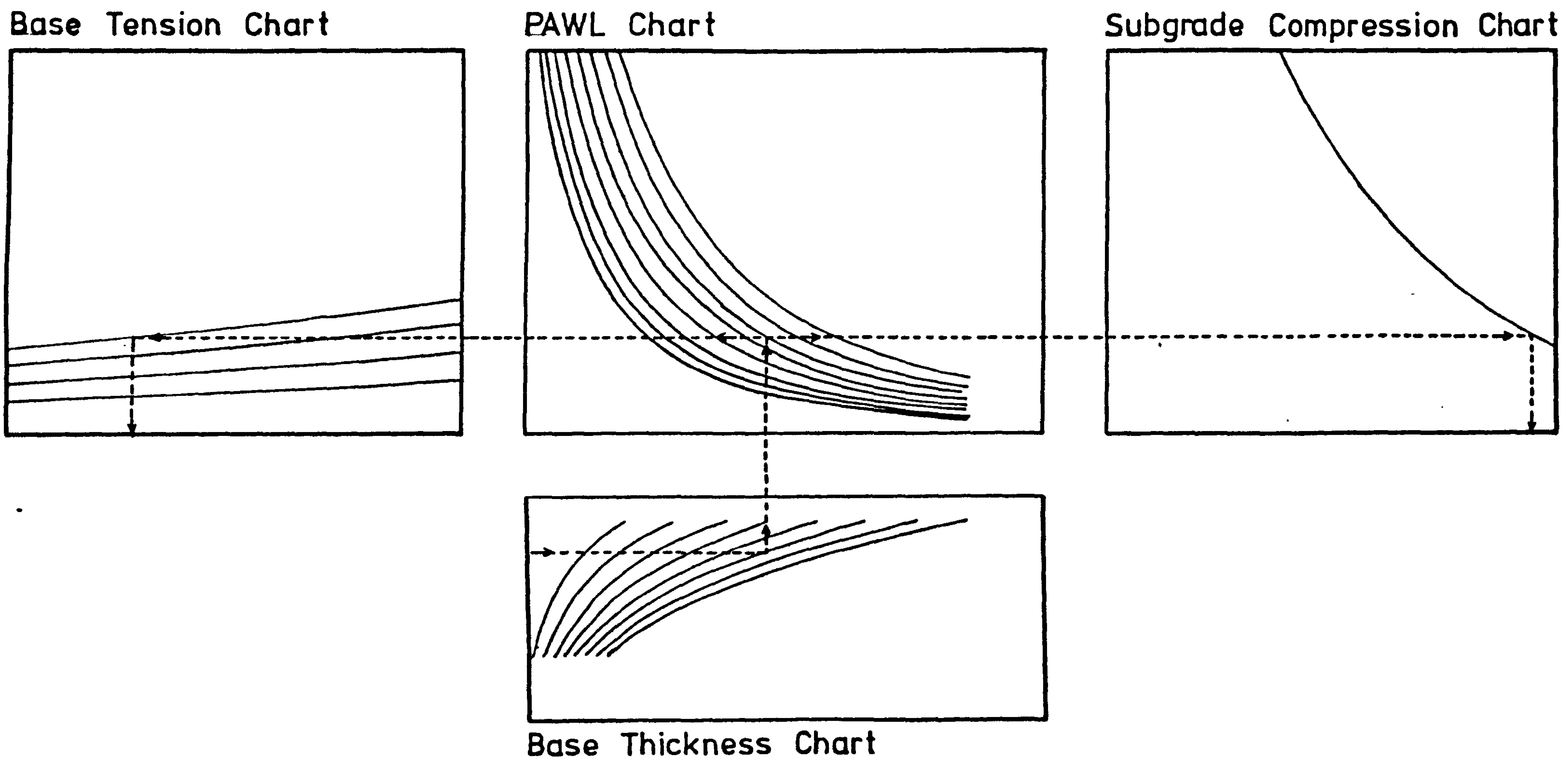
(iv) Subgrade Compression Chart.

These are drawn on a single sheet with common scales to ease interpretation, as shown in Figure 7.13. It is not possible to handle all pavement variables on a single sheet and maintain the clarity and ease of use required. Therefore, the following variables are held constant :-

SURFACING - material type
thickness
elastic modulus

BASE - material type

Figure 7.13 Complete port pavement design chart



SUB-BASE - material type
thickness
elastic modulus

SUBGRADE - strength (California Bearing Ratio)

The sub-base may be omitted, hence the pavement may consist of three or four layers. This leaves four variables :-

BASE - thickness
- elastic modulus

LOADING - magnitude of loading
- design life (number of repetitions)

This limitation of variables poses no problem to design since the surfacing, sub-base or subgrade can be varied independantly on a series of charts.

The complete chart can be used either to check that a trial structure is not overstressed by a particular loading condition, or to determine the thickness of base material required for given design criteria. Whilst the latter approach is more useful for design, the structure of the complete chart is best explained in the reverse order.

7.3.1. Base Thickness Chart.

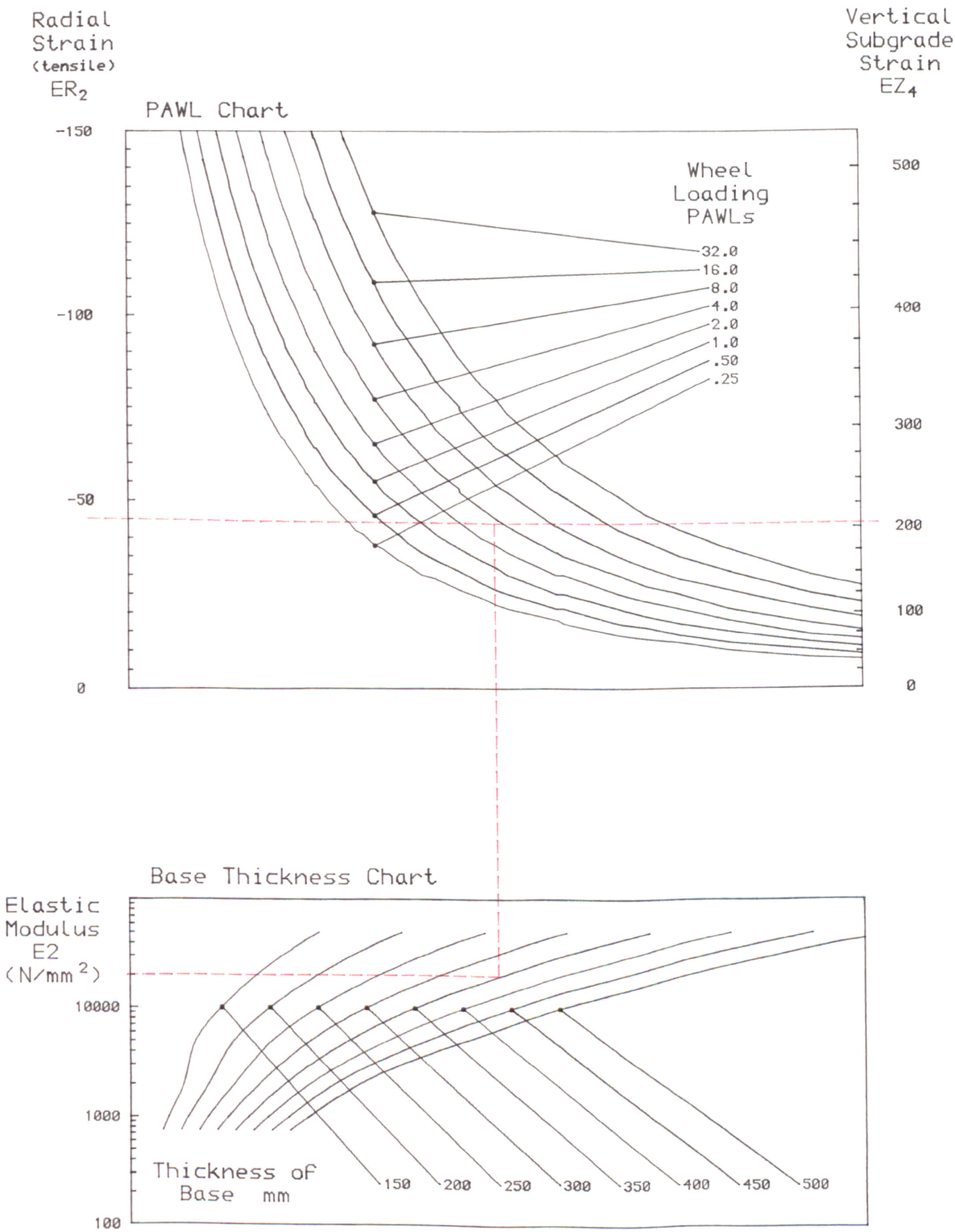
This chart, Figure 7.14, brings the two characteristics of the base layer together. Strength, elastic modulus, is the ordinate, a logarithmic scale from 100 to 100,000 N/mm2, whilst thickness, millimetres, is shown by a series of curves across the graph increasing in magnitude from left to right.

The abscissa for this chart and the PAWL chart situated above is equivalent thickness to the bottom of the base layer. Since the true value of this is of no relevance to the design process, the finished design chart does not show this scale. Hence, equivalent thickness is given for the second interface from the elastic modulus and thickness of the material. This is projected vertically onto the PAWL chart.

7.3.2. PAWL Chart.

This chart, also shown in Figure 7.14, brings the magnitude of wheel loading into the design process. The graph has two ordinate scales, the one on the left is tensile radial strain at the bottom of the bound layer, and the right hand scale is compressive vertical strain at the top of the subgrade. Both are scaled in microstrain.

Figure 7.14 Base Thickness and PAWL Charts



Radial strain is plotted against equivalent thickness to the second interface, for given surface loading conditions. Wheel loadings are given in PAWLs. Up to 8 wheel loading conditions can be included on any one chart and, if specific values are not given, default values of 0.25 to 32.0 PAWLs, increasing in multiples of two, are plotted.

Since the sub-base, if there is one, is constant on each design chart, then it is possible to express vertical compressive strain in the subgrade, EZ, as a function of radial strain in the base, ER. The program uses the following relationship :-

$$ER = EZ / (A \times EZ + B)$$

where A and B are constants found from a least squares, straight line curve fitting subroutine, Ref. 7.5. From this, the right hand ordinate is scaled and also applied to the adjacent chart, the "subgrade compression chart". Common scales for the three upper graphs makes it possible to check fatigue characteristics of base material and the subgrade against the two strains directly.

7.3.3. Subgrade Compression Chart.

This part of the design chart, Figure 7.15, plots allowable vertical subgrade strain against the number of load repetitions to pavement "failure", i.e. the design life. The curve plotted is that proposed by Brown, given earlier in Chapter 3, Ref. 7.6.

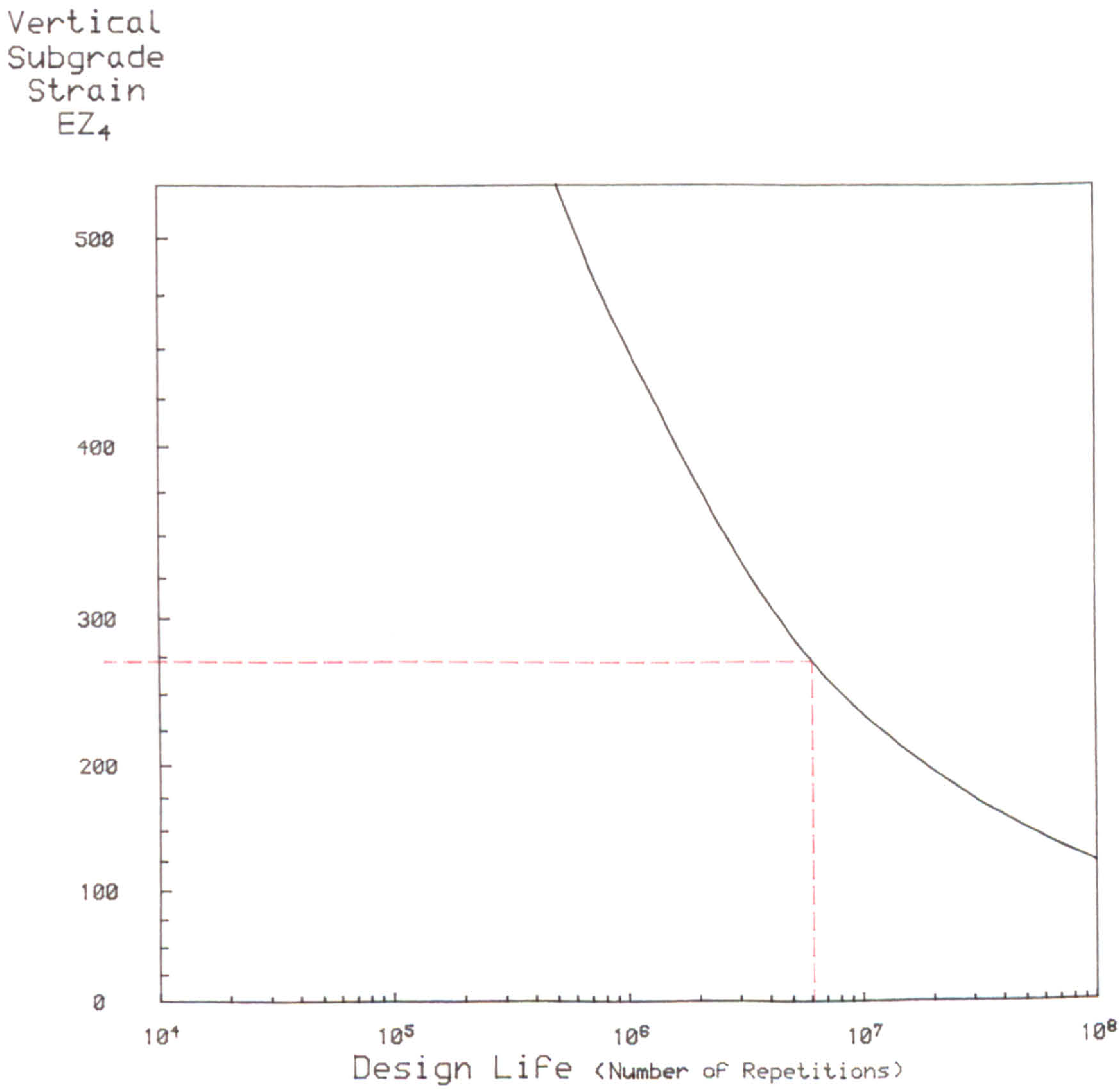
The abscissa to this graph is scaled logarithmically from 10,000 to 100,000,000 load repetitions. The ordinate scale is adjusted to suit the relationship between EZ and ER, as outlined above.

7.3.4. Base Tension Chart.

This chart is very similar to the subgrade compression chart on the opposite side of the page. It is a plot of allowable tensile radial strain against number of load repetitions to "failure". The allowable strain criterion is obviously dependant on material type. Two materials commonly used for road bases are lean concrete and asphaltic bound material, and either of these can be constructed here, Figure 7.16 (a) and (b).

If the base material is cement bound then fatigue curves for four typical mixes are given; mixes with compressive strengths of 24, 18, 12 and 6 N/mm2. If, however, the base material

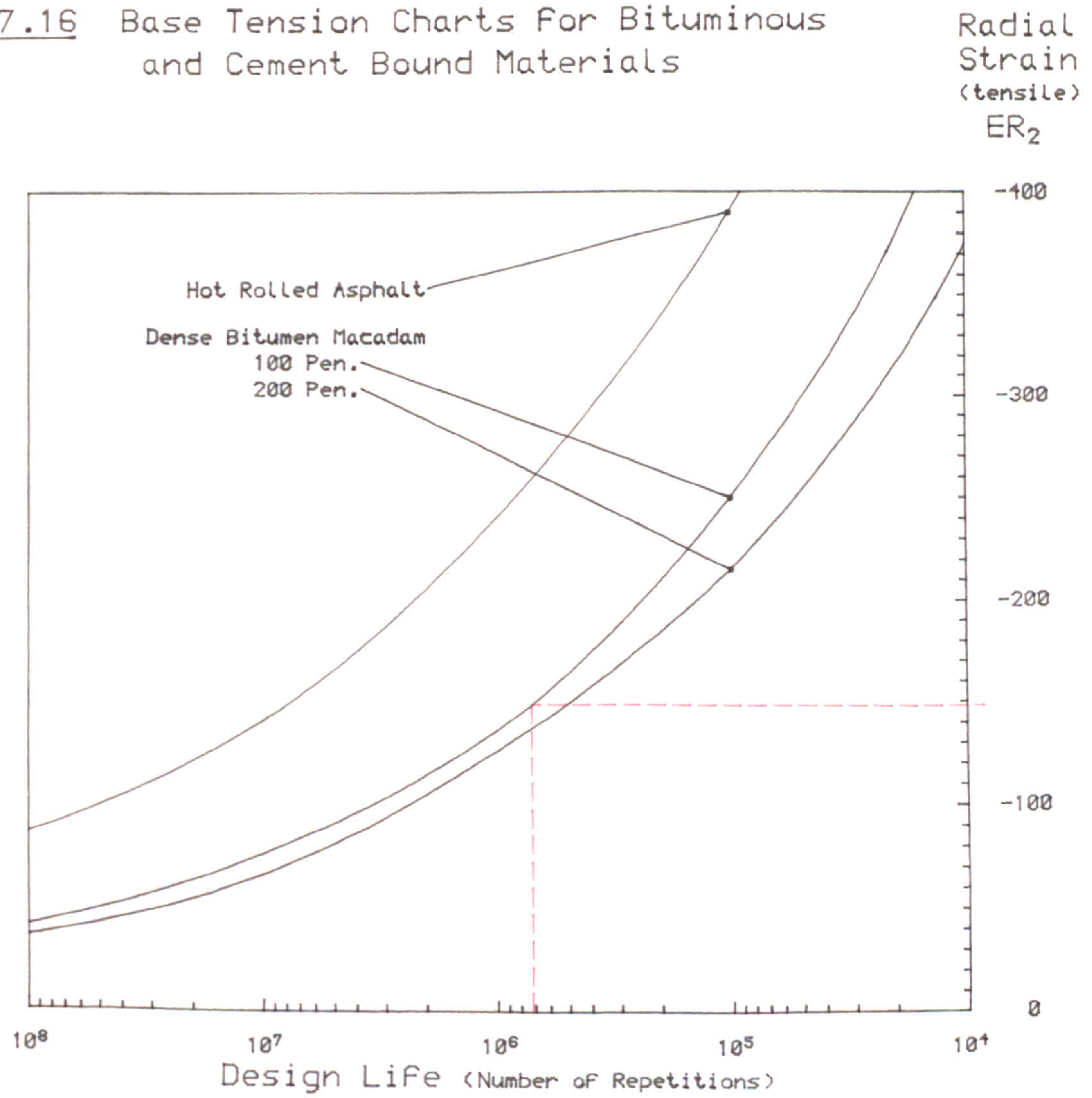
Figure 7.15 Subgrade Compression Chart and Heading Box



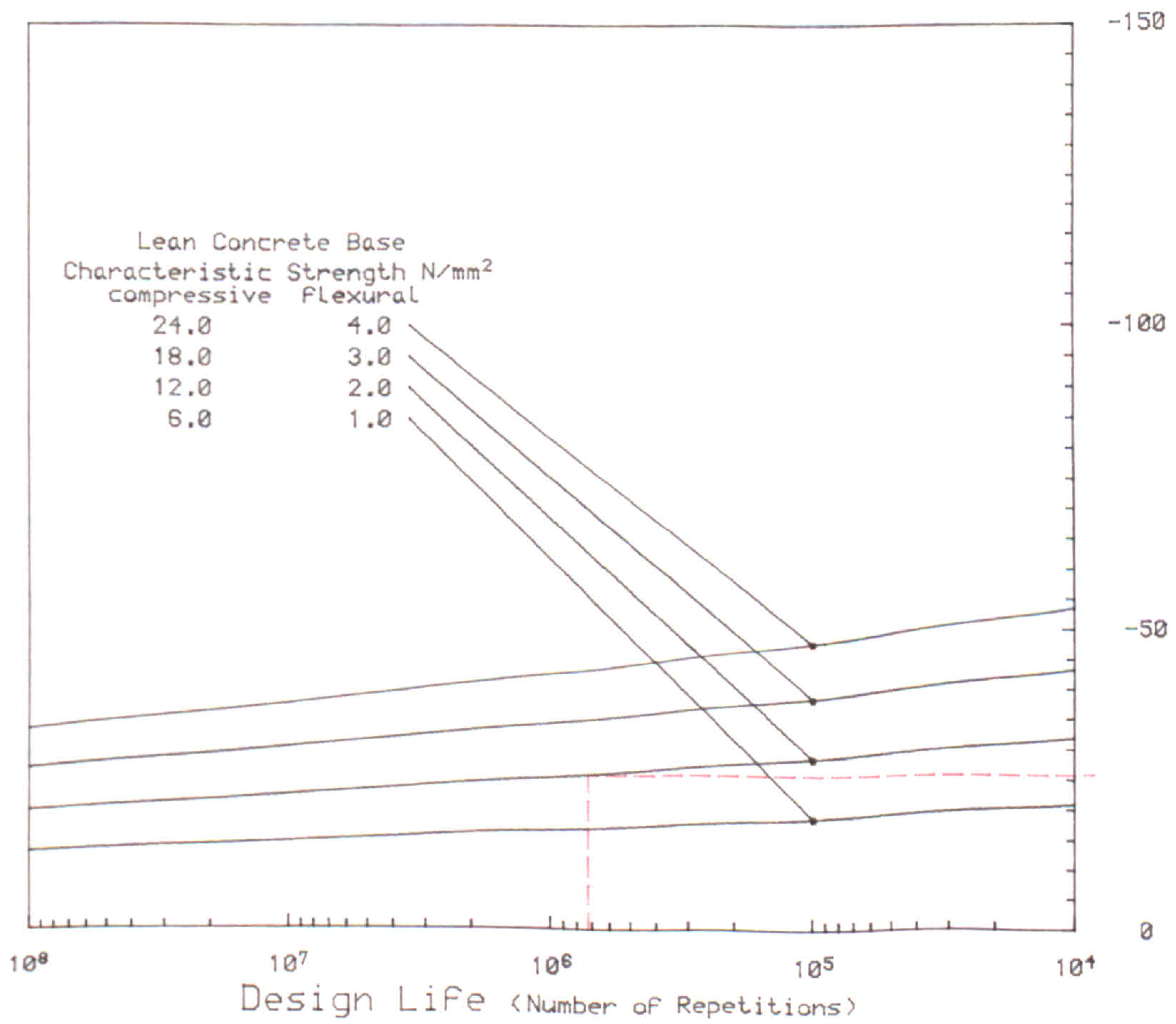
SURFACING	80mm Concrete Blocks Layer Thickness 130 mm Elastic Modulus 7500 N/mm^2
BASE	Lean Concrete Layer Thickness ? Elastic Modulus ?
SUBBASE	Granular Layer Thickness 250 mm
SUBGRADE	C.B.R. 5 %

Figure 7.16 Base Tension Charts for Bituminous and Cement Bound Materials

(a)



(b)



selected for the pavement is bituminous, then these are replaced by curves for three typical asphaltic mixes; hot rolled asphalt, 100 pen. and 200 pen. dense bitumen macadam. The derivation of the fatigue curves for both these materials was given above.

A horizontal line from the PAWL Chart to the respective fatigue curve gives estimated design life. This value is then compared to the estimated design life for the subgrade, and the pavement's design life is the lower of the two values. If, for a given magnitude of loading, this value is too low, then base thickness or base strength can be increased and the checking procedure repeated until a satisfactory structure is established. This iterative procedure can be avoided by using the chart in the reverse order; i.e. starting with design life and finishing with thickness of base required. This is outlined in Chapter 8 where a design example is given.

7.4. 'SETUP' - Data Entry.

The program 'CHART' requires a large number of results with the different variables incremented in a pre-determined step size. Manual data entry of this magnitude is not plausible and a third program is required to compile all values in the correct order for the analysis program. This program is called 'SETUP'.

Like 'PEPAP', this program is written with full prompts for the operator and all data entry is in free format. It offers the following options for materials in each layer :-

SURFACING	-	Concrete blocks (80 mm)
		Asphaltic
		Other
BASE	-	Asphaltic
		Lean concrete
		Cement bound granular material
		Soil cement
		Other
SUB-BASE	-	Soil cement
		Granular
		Other

The subgrade strength, CBR, is also required. For both surfacing and sub-base, thickness and elastic modulus are requested unless they are given specific values by the analysis program (see Section 7.2.4).

Thickness and elastic modulus of base materials have to be incremented to give sufficient points to plot the design curves. Up to eight values for each variable can be specified, and these can be entered by the operator or the following default values adopted :-

material type	lean concrete
base thickness, mm	150, 200, 250, 300, 350, 400, 450, 500
elastic modulus, N/mm2	750, 1000, 2000, 5000, 10000, 30000, 50000

The only other variable on the design chart is the magnitude of wheel loading. This is quantified in terms of critical PAWL rating and again up to eight different values can be specified for each chart. These can be entered by the operator, together with twin tyre spacing, or, if specific values are not required, default PAWL ratings can be used; these increase from 0.25 to 32.0 PAWLs in multiples of 2.0, all are assumed to have a single contact area.

7.5. The Complete Program Suite.

These three programs are run in sequence from a single command call. They use two data files where results are stored temporarily until called by the next program. The complete routine is shown in Figure 7.17. The final design chart can be drawn on a VDU or copied to a plotter, where it measures 560 x 297 mm (triple A4 foldout), although this may be scaled up or down to any required size. Six design charts are included here, Figures 7.18 to 7.23.

Figure 7.17 Routine for construction of design charts

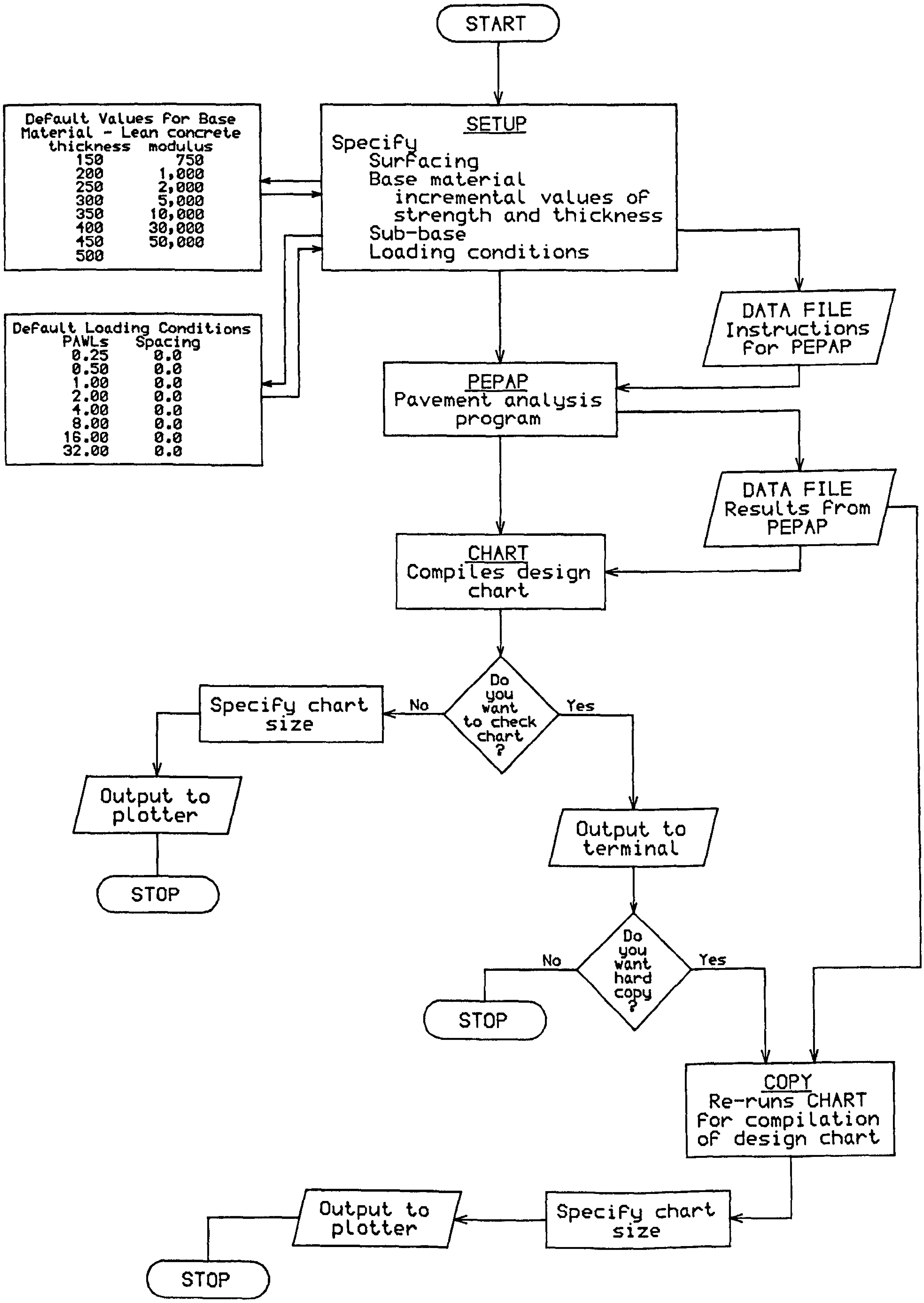
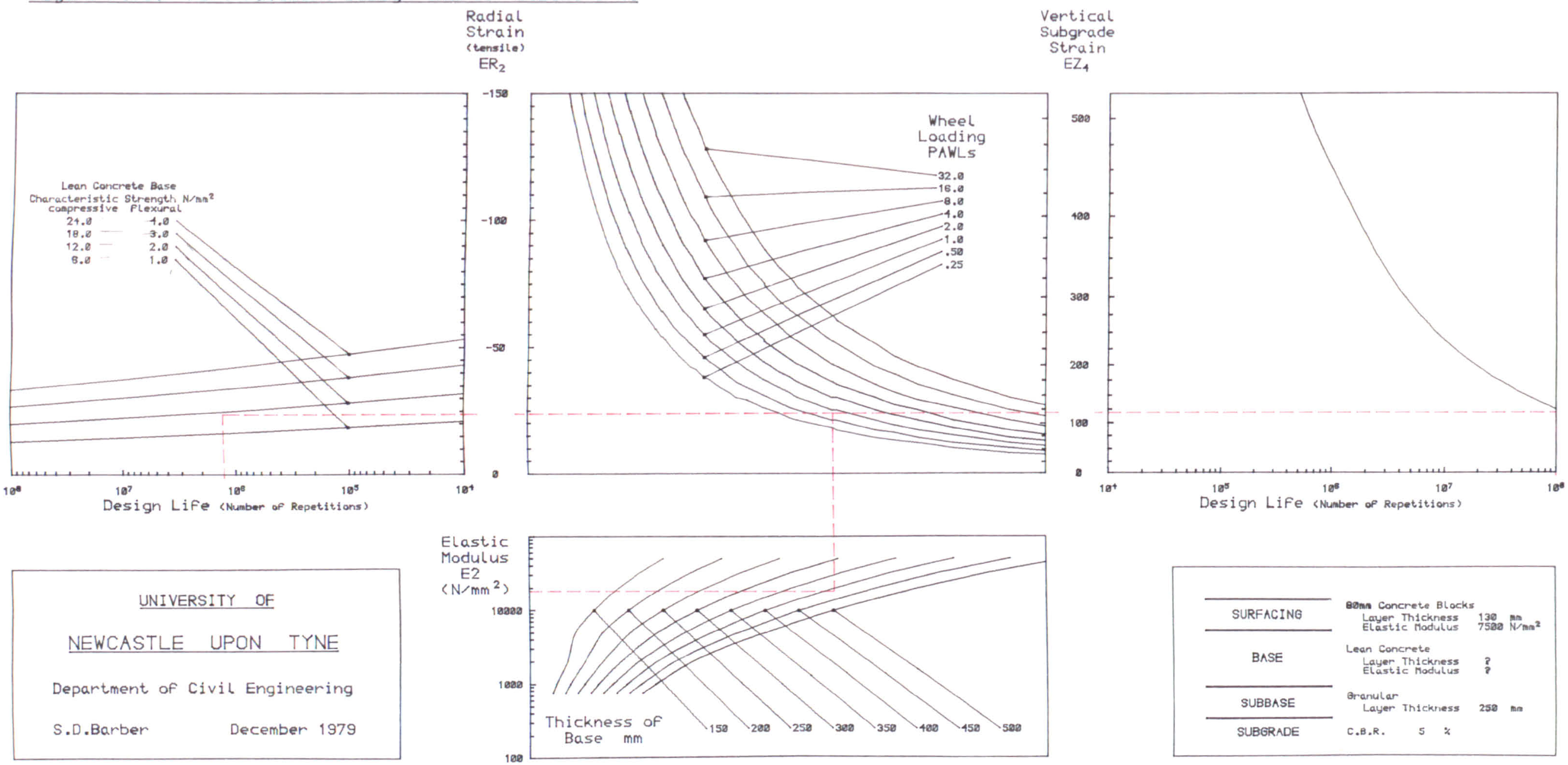


Figure 7.18. Port Pavement Design Chart. C.B.R. 5 %.



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S.D.Barber December 1979

Figure 7.19. Port Pavement Design Chart. C.B.R. 5 %.

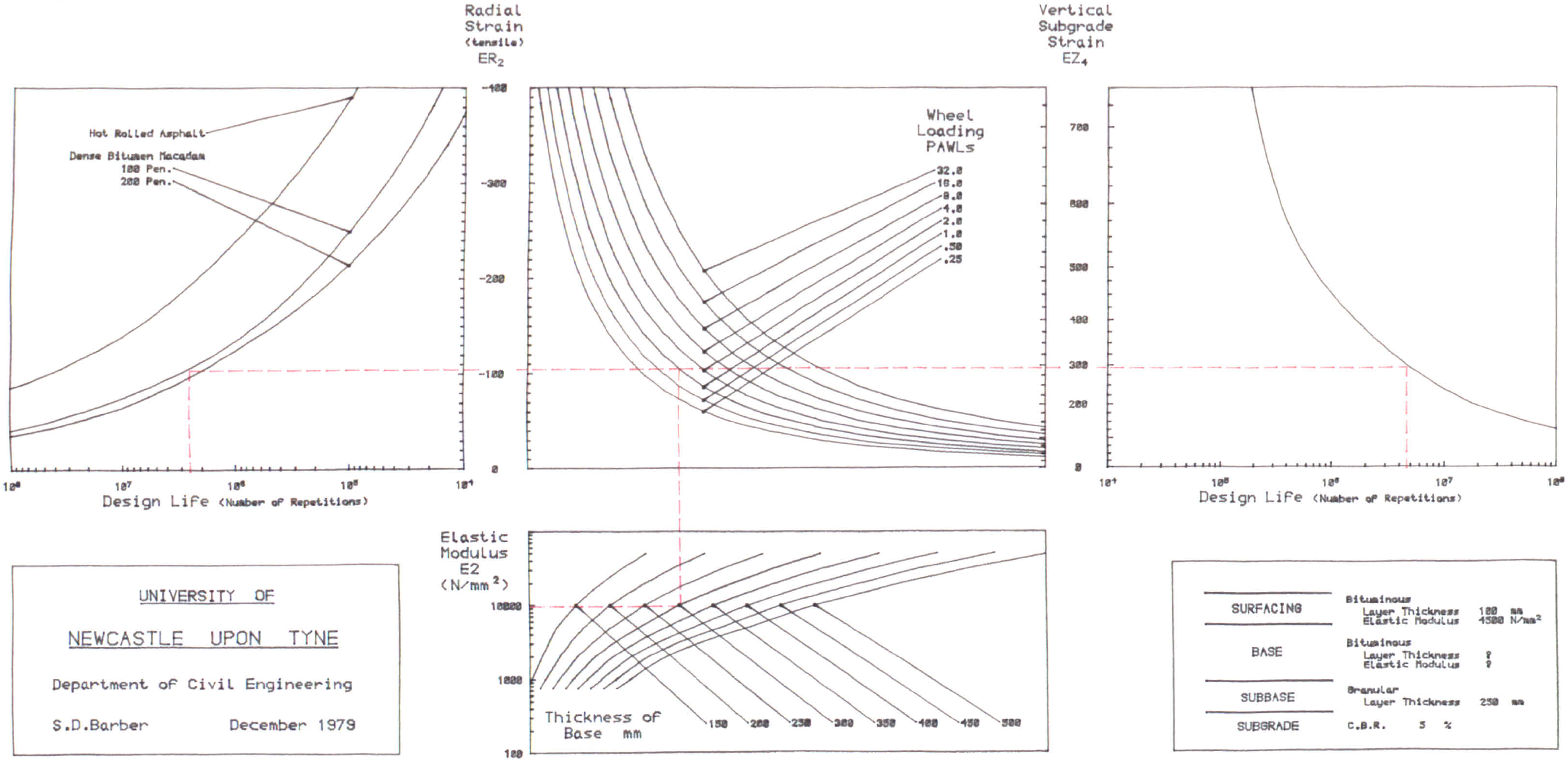


Figure 7.20. Port Pavement Design Chart. C.B.R. 7 %.

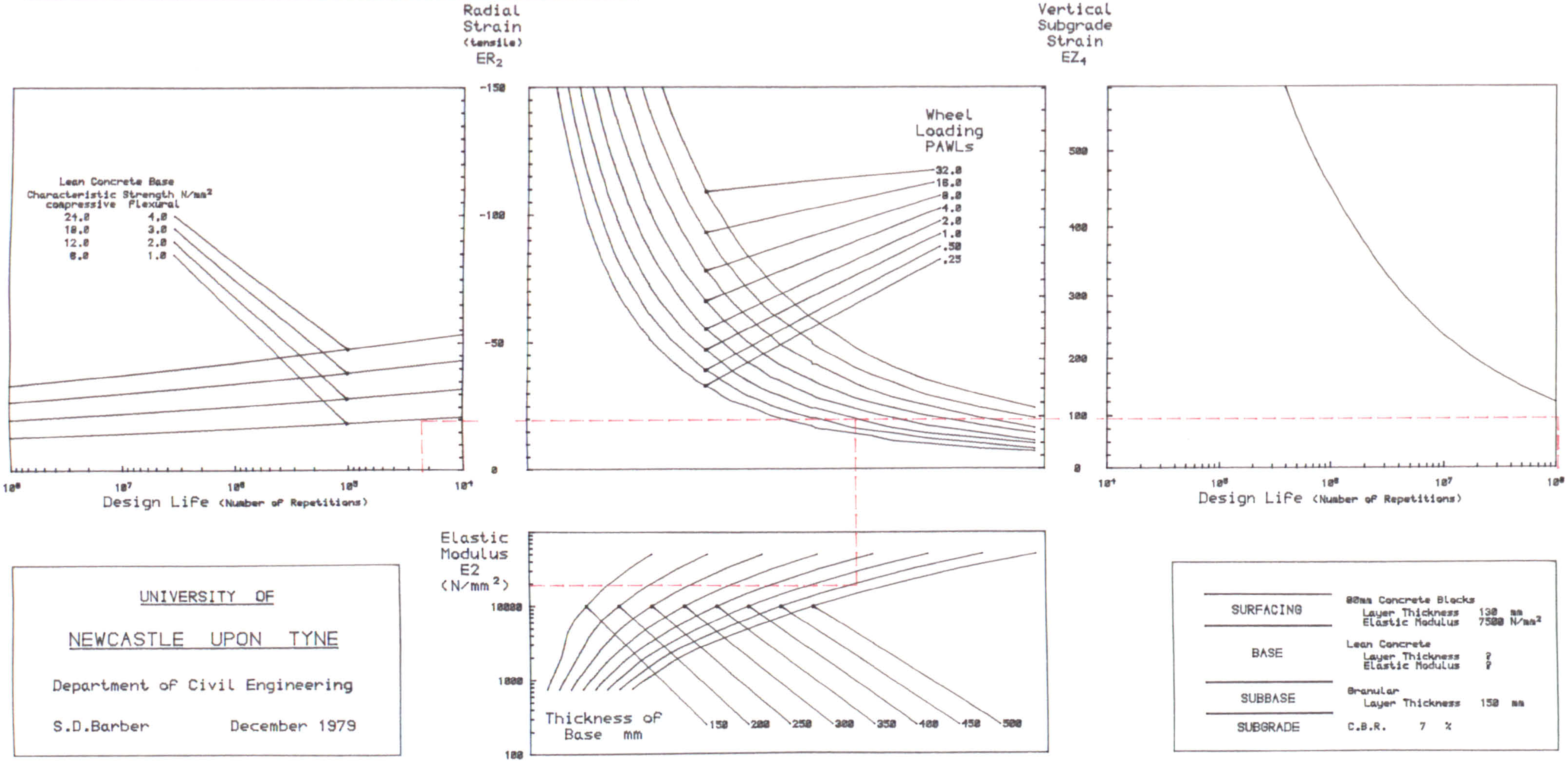


Figure 7.21. Port Pavement Design Chart. C.B.R. 7 %.

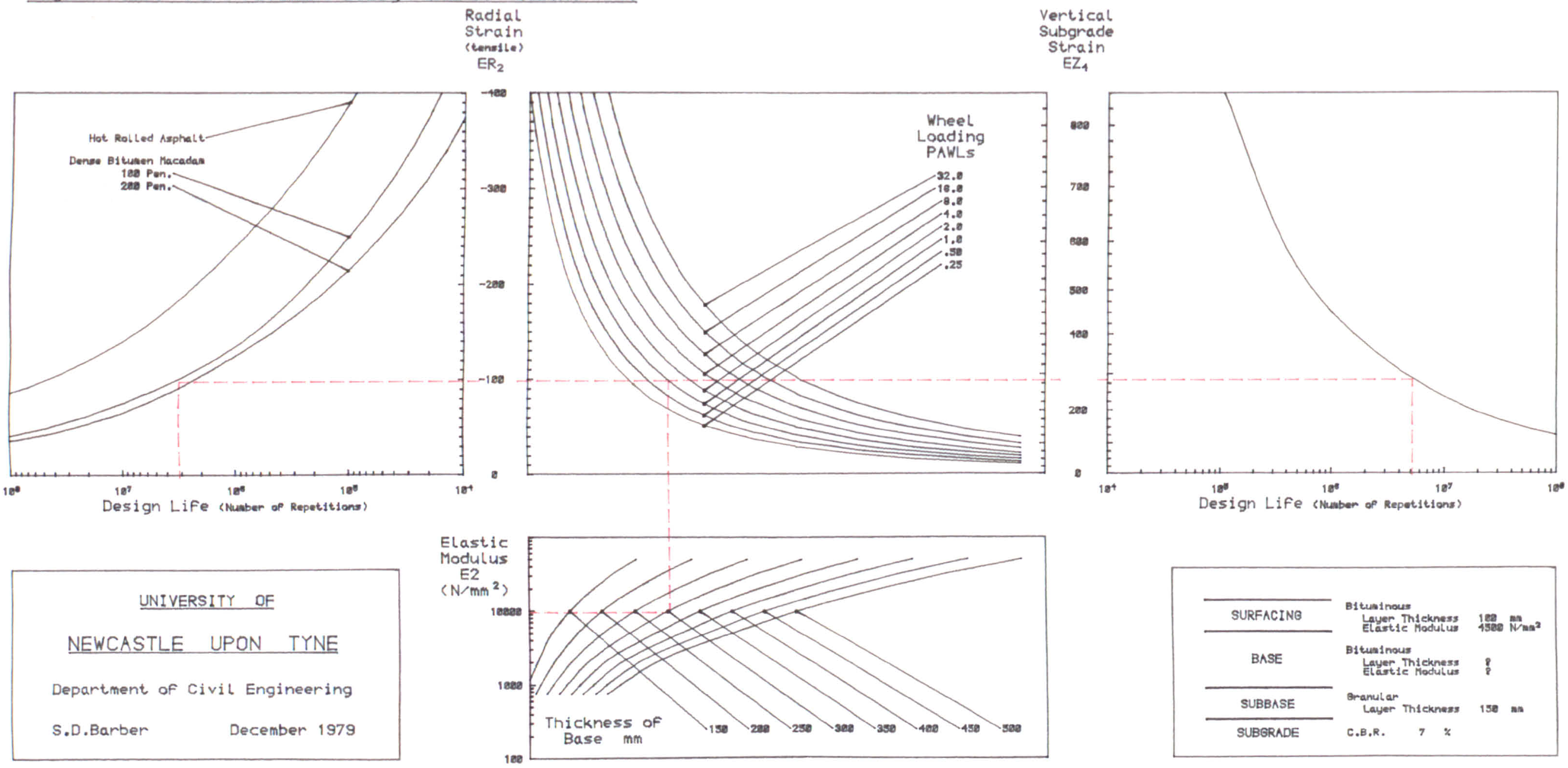


Figure 7.22. Port Pavement Design Chart. C.B.R. 10 %.

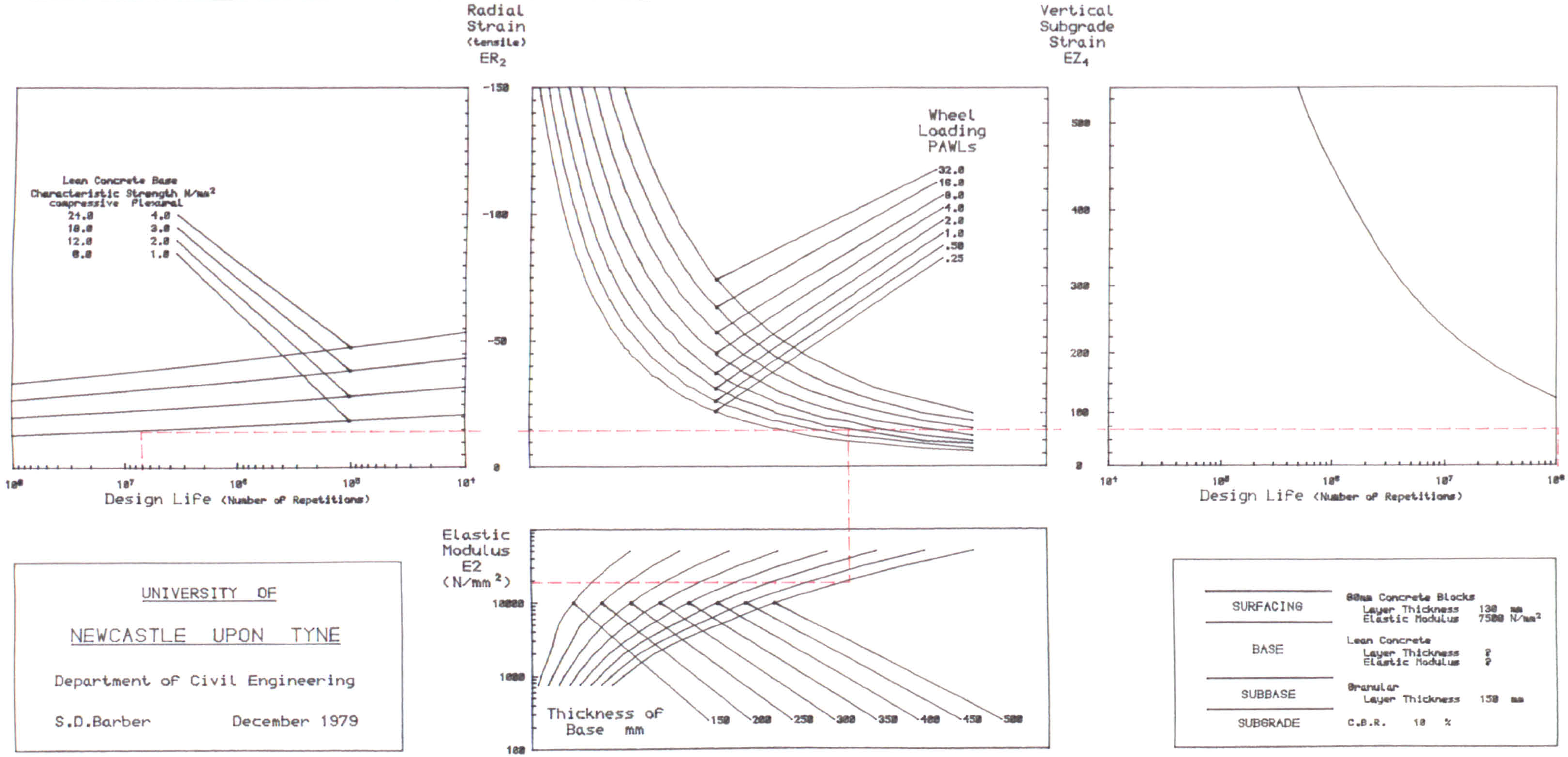
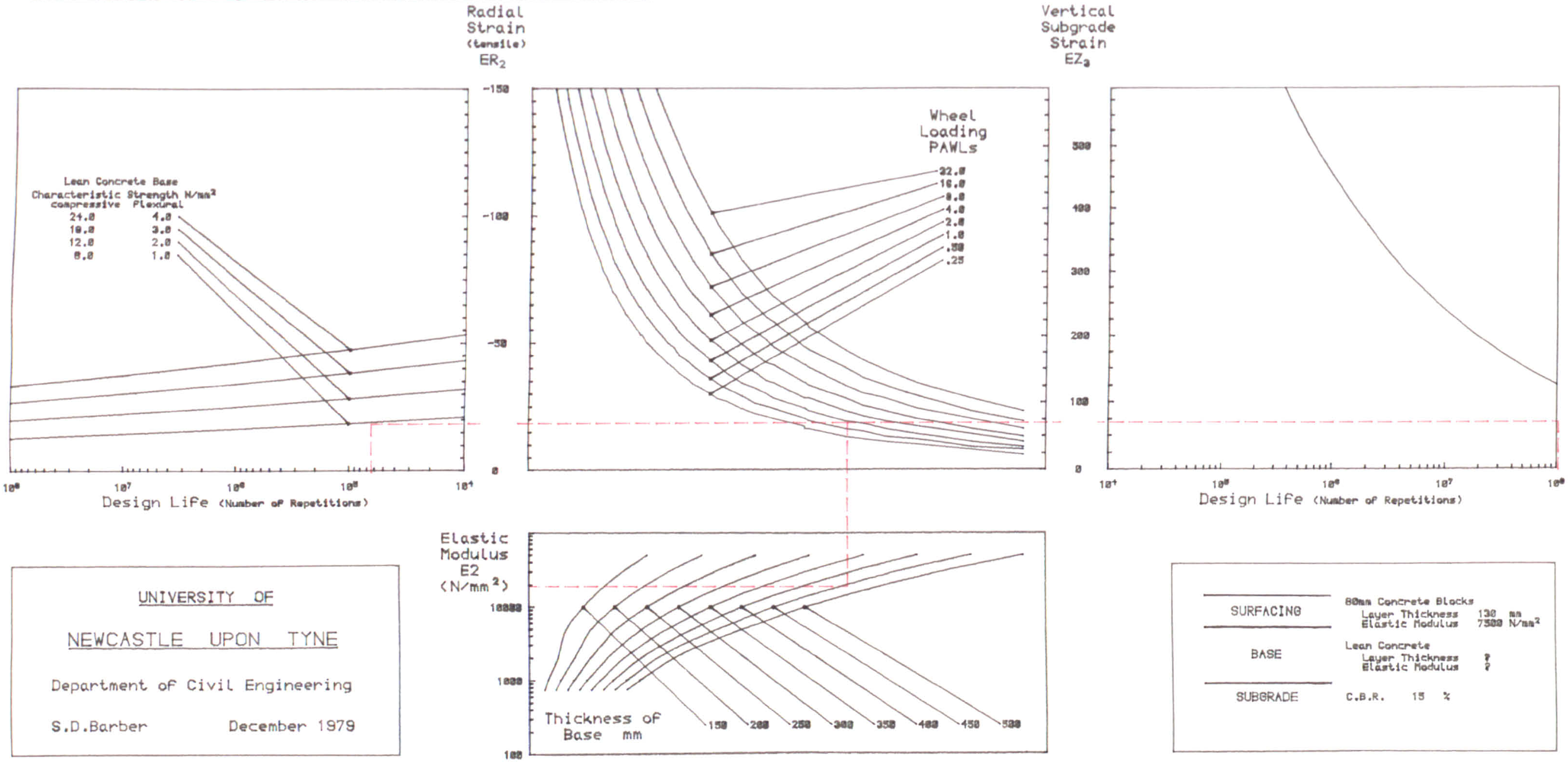


Figure 7.23. Port Pavement Design Chart. C.B.R. 15 %.



7.6. References.

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CHAPTER EIGHT

APPLICATION OF DESIGN METHOD

Synopsis

This Chapter illustrates the application of the design method for both existing pavements and new structures. Some of the pavement structures described in Chapter 2 are analysed and the results correlated to pavement performance. Two design examples are presented, one using the analysis program and the second using a design chart. Both examples are based on pavements in container handling areas. The final section of this Chapter discusses future extensions and refinements to the method.

8.1. Introduction.

The development of pavement design theory and computer analysis has been described in the previous Chapters. This Chapter demonstrates the application of the design method to pavement structures in three ways; analysis of existing pavements, use of analysis program for design and use of the design charts. Also, design thicknesses from the analysis method developed in this thesis are compared to the results from other design methods.

In the final section of this Chapter, the comprehensiveness of the complete design method is discussed and future extensions and refinements proposed.

8.2. Analysis of Existing Structures.

The analysis of existing pavement structures can be used to provide explanations for the success or failure of particular pavements, but it can also provide a check on the results obtained from new design methods. Unfortunately, very little comprehensive data has been collected on the performance of industrial pavements, particularly in port areas. The only widespread classification being either "satisfactory" or "failed", and the broad definition of these classifications can vary widely between different port authorities. It is encouraging, however, that this distinction on performance is clearly reflected in the analysis results.

In Chapter 2 a wide range of pavement constructions were presented, and some of these are now considered in more detail. In all structures analysed, vehicle speeds are specified as 5 kph, or a pulse loading time of 0.2 s within the structure, and pavement temperature is 15 deg C. The magnitude of wheel loading and subgrade strength are representative of true values.

8.2.1. Asphaltic pavements.

For asphaltic constructions, the in-situ mix stiffness, or elastic modulus, must be estimated. The application of the equations in Chapter 7 gave the following two group distinction for asphaltic materials :-

(i) 50 or 60 pen. bitumen mixes with high stone contents (55 to 65 %) : mix stiffness 4,500 N/mm2.

(ii) 100 pen. bitumen mixes and rolled asphalt : mix stiffness 2,300 N/mm2.

These values are substantially lower than those used in highway design because of the much lower vehicle speeds.

Two ports with severe problems of rutting under trafficking from straddle carriers were Ipswich and Liverpool, Figures 2.4 and 2.2. The construction at Ipswich was 100 mm high stone content asphalt surfacing over 100 mm asphalt base, whilst that at Liverpool was 100 mm asphalt surfacing, 200 mm wet mix macadam base with 150 mm granular sub-base. The stresses and strains within these two structures are shown in Figures 8.1 and 8.2.

In both these structures the radial strain within the asphaltic layer and the vertical subgrade strain are very high. This would be expected to lead to excessive rutting and surface cracking in areas of concentrated straddle carrier operation. It was noted in Chapter 2 that oil pollution severly aggravated pavement damage at Liverpool, leading to severe surface disintegration.

The Port of Aarhus also has full depth asphaltic construction, Figure 2.3 (a) and (b). The first of these, like the pavements at Liverpool, adopted a wet mix macadam base. This is a relatively weak material having an elastic modulus of similar magnitude to granular material, its performance in these

Figure 8.1 Analysis pavement construction at Port of Ipswich.

```
       WLOAD= 144.4 kN
       PAWL = 2.00
       TPRES= 0.80  N/MM2
       RAD  =  240. MM

            VVVVVV
  -------------------------

 LAYER NO.1 ASPHALTIC

 THICKNESS =  100.  MM
 MODULUS   =  4500. N/MM2
 POISS RAT =  0.40
                          RADIAL STRAIN =   -22.
  -------------------------
                          VERT STRESS   = 0.7652 N/MM2
                          VERT STRAIN   =   109.
 LAYER NO.2 ASPHALTIC

 THICKNESS =  100.  MM
 MODULUS   =  4500. N/MM2
 POISS RAT =  0.40
                          RADIAL STRAIN =  -549.
  -------------------------
                          VERT STRESS   = 0.2703 N/MM2
                          VERT STRAIN   =  1363.
      SUBGRADE
 MODULUS   =   200. N/MM2
 POISS RAT =  0.28

                      SURFACE DELECTION   = 0.7634  MM
                      EFF SURFACE MODULUS=   464. N/MM2
                      EFF POISSONS RATIO =  0.28
```

Figure 8.2 Analysis pavement construction at Port of Liverpool.

```
      WLOAD= 144.4 kN
      PAWL = 2.00
      TPRES= 0.80  N/MM2
      RAD  =  240. MM

           VVVVVV
  -------------------------

 LAYER NO.1 ASPHALTIC

 THICKNESS =  100.  MM
 MODULUS   =  2300. N/MM2
 POISS RAT =  0.40
                          RADIAL STRAIN =  -211.
  -------------------------
                          VERT STRESS   = 0.6886 N/MM2
                          VERT STRAIN   =   710.
 LAYER NO.2 ASPHALTIC

 THICKNESS =  200.  MM
 MODULUS   =   750. N/MM2
 POISS RAT =  0.40
                          RADIAL STRAIN =  -435.
  -------------------------
                          VERT STRESS   = 0.3215 N/MM2
                          VERT STRAIN   =  1080.
 LAYER NO.3 GRANULAR

 THICKNESS =  150.  MM
 MODULUS   =   300. N/MM2
 POISS RAT =  0.24
                          RADIAL STRAIN =  -268.
  -------------------------
                          VERT STRESS   = 0.1494 N/MM2
                          VERT STRAIN   =  1017.
       SUBGRADE
 MODULUS   =   150. N/MM2
 POISS RAT =  0.31

                     SURFACE DELECTION  = 0.9845  MM
                     EFF SURFACE MODULUS=   353. N/MM2
                     EFF POISSONS RATIO =  0.31
```

structures does not appear to justify its higher cost.

Analysis of the two structures at Aarhus clearly shows that these pavements are much stronger constructions, Figure 8.3 (a) and (b). Although wheel loading is significantly greater, compressive vertical subgrade strains are significantly lower than above, giving a lower incidence of rutting. The two radial strains within the asphaltic materials are also lower, although in the original specification it is still relatively high. The prevention of rutting is probably helped by the more random operation of front lift trucks compared to straddle carriers. In the revised specification, Figure 8.3 (a), the road-base and macadam base were replaced by an asphaltic mix with 60 pen. bitumen. This considerably increased pavement stiffness, reducing vertical subgrade strain by 40 %, and radial asphalt strain by 50 %. Both these constructions have proved durable, the latter being marginally less expensive, considerably quicker to construct and having a substantially greater design life.

The Port of Felixstowe laid an area of asphaltic surfacing over 375 mm of various types of granular material, Figure 2.7 (b). This did not withstand heavy trafficking from straddle carriers and required complete replacement. The analysis, Figure 8.4, shows that whilst vertical subgrade strain is reasonable, tensile radial strain within the surfacing is much higher than

Figure 8.3 (a) Analysis pavement constructions at Port of Aarhus.

Original specification; 1969 - 1970.

```
WLOAD= 208.9 kN
PAWL = 8.00
TPRES= 0.80  N/MM2
RAD  =  204. MM
SPACE=  600. MM

    VVVV  VVVV
------------------------

LAYER NO.1 ASPHALTIC
                                                          OFFSET DISTANCE
THICKNESS =  240.  MM
MODULUS   =  2300. N/MM2                                            300.
POISS RAT =  0.40
                        RADIAL STRAIN =  -301.                      -298.
------------------------
                        VERT STRESS   = 0.1947 N/MM2              0.2579
                        VERT STRAIN   =   450.                      644.
LAYER NO.2 ASPHALTIC

THICKNESS =  230.  MM
MODULUS   =   375. N/MM2
POISS RAT =  0.40
                        RADIAL STRAIN =  -231.                      -216.
------------------------
                        VERT STRESS   = 0.1400 N/MM2              0.1354
                        VERT STRAIN   =   468.                      454.
LAYER NO.3 GRANULAR

THICKNESS =  150.  MM
MODULUS   =   300. N/MM2
POISS RAT =  0.24
                        RADIAL STRAIN =  -201.                      -177.
------------------------
                        VERT STRESS   = 0.0924 N/MM2              0.0840
                        VERT STRAIN   =   623.                      560.
      SUBGRADE
MODULUS   =   150. N/MM2
POISS RAT =  0.31

                   SURFACE DELECTION  = 0.8112  MM                0.8644
                   EFF SURFACE MODULUS=   515. N/MM2                484.
                   EFF POISSONS RATIO =  0.31
```

Figure 8.3 (b) Analysis pavement constructions at Port of Aarhus.

Revised specification 1973 - 1974.

```
     WLOAD= 208.9 kN
     PAWL = 8.00
     TPRES= 0.80  N/MM2
     RAD  =  204. MM
     SPACE=  600. MM

        VVVV  VVVV
 -------------------------

LAYER NO.1 ASPHALTIC
                                                      OFFSET DISTANCE
THICKNESS =  120.  MM
MODULUS   =  2300. N/MM2                                        300.
POISS RAT =  0.40
                          RADIAL STRAIN =   -30.                -30.
 -------------------------
                          VERT STRESS   = 0.2060 N/MM2        0.7674
                          VERT STRAIN   =    36.                113.
LAYER NO.2 ASPHALTIC

THICKNESS =  300.  MM
MODULUS   =  4500. N/MM2
POISS RAT =  0.40
                          RADIAL STRAIN =  -162.               -150.
 -------------------------
                          VERT STRESS   = 0.1036 N/MM2        0.0947
                          VERT STRAIN   =   351.                319.
LAYER NO.3 GRANULAR

THICKNESS =  150.  MM
MODULUS   =   300. N/MM2
POISS RAT =  0.24
                          RADIAL STRAIN =  -143.               -129.
 -------------------------
                          VERT STRESS   = 0.0674 N/MM2        0.0614
                          VERT STRAIN   =   459.                413.
      SUBGRADE
MODULUS   =   150. N/MM2
POISS RAT =  0.31

                      SURFACE DELECTION  = 0.6208  MM         0.6485
                      EFF SURFACE MODULUS=   673. N/MM2         645.
                      EFF POISSONS RATIO =  0.31
```

Figure 8.4 Analysis pavement construction at Port of Felixstowe.

```
        WLOAD= 144.4 kN
        PAWL = 2.00
        TPRES= 0.80  N/MM2
        RAD  =  240. MM

              VVVVVV
   -------------------------

  LAYER NO.1 ASPHALTIC

  THICKNESS =  100.  MM
  MODULUS   =  2300. N/MM2
  POISS RAT =  0.40
                           RADIAL STRAIN =  -587.
   -------------------------
                           VERT STRESS   = 0.6609 N/MM2
                           VERT STRAIN   =  1553.
  LAYER NO.2 GRANULAR

  THICKNESS =  375.  MM
  MODULUS   =   400. N/MM2
  POISS RAT =  0.21
                           RADIAL STRAIN =  -159.
   -------------------------
                           VERT STRESS   = 0.1887 N/MM2
                           VERT STRAIN   =   644.
       SUBGRADE
  MODULUS   =   300. N/MM2
  POISS RAT =  0.24

                       SURFACE DELECTION    = 0.7774  MM
                       EFF SURFACE MODULUS=    466. N/MM2
                       EFF POISSONS RATIO =   0.24
```

tensile strength.

8.2.2. Cementitious base construction.

Pavements with a lean concrete base have been widely used and have proved the most durable form of construction. The analysis of a typical structure is shown in Figure 8.5. Here critical tensile fatigue strain is at the bottom of the cement bound base. Tensile strain within the surfacing is very small, even compressive for pavements with a surfacing of lower relative stiffness. Also, compressive subgrade strain is low, thus design is relatively independant of subgrade strength.

Since the stiffness of cement bound material is very high, the strength of the surfacing has a much lower effect on tensile strain within the base. A doubling of an asphaltic surfacing from 75 to 150 mm for the structure illustrated reduces tensile strain by 16 %, whilst the same reduction is obtained by increasing base thickness by 25 mm to 275 mm. The type of surfacing, therefore, for a pavement with a cementitious base is selected for surface durability and not overall pavement strength.

Figure 8.5 Analysis of a typical pavement construction with a cement bound base.

```
        WLOAD= 144.4 kN
        PAWL = 2.00
        TPRES= 0.80  N/MM2
        RAD  =  240. MM

             VVVVVV
     -------------------------

    LAYER NO.1 ASPHALTIC

    THICKNESS =   75.  MM
    MODULUS   =  4500. N/MM2
    POISS RAT =  0.40
                            RADIAL STRAIN =    -6.
     -------------------------
                            VERT STRESS   = 0.7976 N/MM2
                            VERT STRAIN   =    19.
    LAYER NO.2 LEAN CONCRETE

    THICKNESS =  225.  MM
    MODULUS   = 33000. N/MM2
    POISS RAT =  0.20
                            RADIAL STRAIN =   -60.
     -------------------------
                            VERT STRESS   = 0.0439 N/MM2
                            VERT STRAIN   =   228.
           SUBGRADE
    MODULUS   =   200. N/MM2
    POISS RAT =  0.28

                        SURFACE DELECTION  = 0.2984  MM
                        EFF SURFACE MODULUS=  1186. N/MM2
                        EFF POISSONS RATIO =  0.28
```

8.3. Design Examples.

Two design examples are presented, one using the analysis program directly, and the second using a design chart. In both cases the resulting designs are compared to design recommendations given by the Cement and Concrete Assoc., C&CA, in the two publications "Concrete block roads for heavily trafficked roads and paved areas", based on Road Note 29, and ADS/39, Refs. 8.1, 8.2 and 8.3.

8.3.1 Design using "PEPAP".

A pavement in a Ro-Ro trailer park is to be designed to the following design criteria :-

Subgrade	dredged chalk fill material, CBR 16 to 40 %, average value 26 %.
Design life	15 years.
Trafficking	(all highway trailer units) 75 % towed by terminal tractors. 25 % towed by highway tractors. total of 800 vehicles per day in both directions (including future expansion).

Of the 800 vehicles/day, 75 % would pass through the parking area. It is assumed that trafficking in access lanes of general parking area is 50 % of this proportion.

No. movements per day = 800 x 0.75 x 0.50

= 300

Design vehicle is a highway trailer towed by a terminal tractor; axle loads when fully laden are 4, 10, 9 and 9 tonnes. The most damaging combination of frequency and load is under a 20 tonne load, thus for the vehicle :-

Average PAWL rating = 0.021 PAWLs

Critical PAWL rating = 0.0165 PAWLs

Thus, the design unit, corresponding to the critical PAWL rating, is a 4.7 tonne wheel load at 0.5 N/mm2 contact stress.

Total No. PAWLs = 300 x 365 x 15 x 0.021

= 34,500

No. repetitions = 34,500 / 0.0165

= 2.1 million

Four designs are considered :-

(i) lean concrete with block surfacing

(ii) high stone content asphalt and base

(iii) granular sub-base with block surfacing

(iv) 140 mm thick precast rafts (2m by 2m)

Design criteria were established from the PEPAP program, as follows :-

(i) allowable vertical subgrade strain 367 microstrain

(ii) lean concrete

compressive strength 15 N/mm2

elastic modulus 33,000 N/mm2

allowable tensile strain -29 microstrain

(iii) asphaltic mixes

100 mm surfacing, elastic modulus 5,000 N/mm2

base - 60 pen. bitumen with 65 % stone content, elastic modulus 4,500 N/mm2

allowable tensile strain -164 microstrain

By use of the editing facility within PEPAP, layer thicknesses were continually modified until all design criteria had been satisfied. The three final designs are shown in Figure 8.6, (a) to (c). (It should be noted that in the third design, block surfacing over a granular sub-base, the stiffness of the first layer was reduced considerably. This is because a segmental surfacing cannot accommodate high tensile strains, the equivalent stiffness was reduced until tensile radial strain at the base of the first layer was less than that in the granular sub-base.)

Figure 8.6 (a) Trial pavement designs. Design One.
Lean concrete base with block surfacing.

```
      WLOAD=   47.0 kN
      PAWL = 0.0165
      TPRES= 0.50  N/MM2
      RAD  =  173. MM

           VVVVVV
 -------------------------

LAYER NO.1 80 MM CONCRETE BLOCKS

THICKNESS =  130.  MM
MODULUS   =  7500. N/MM2
POISS RAT =  0.20
                         RADIAL STRAIN =      0.
 -------------------------
                         VERT STRESS   = 0.4722 N/MM2
                         VERT STRAIN   =    13.
LAYER NO.2 LEAN CONCRETE

THICKNESS =  150.  MM
MODULUS   = 33000. N/MM2
POISS RAT =  0.20
                         RADIAL STRAIN =    -28.
 -------------------------
                         VERT STRESS   = 0.0268 N/MM2
                         VERT STRAIN   =   107.
       SUBGRADE
MODULUS   =   260. N/MM2
POISS RAT =  0.25

                    SURFACE DELECTION  = 0.1075  MM
                    EFF SURFACE MODULUS=  1509. N/MM2
                    EFF POISSONS RATIO =  0.25
```

Figure 8.6 (b) Trial pavement designs. Design Two.

All asphaltic construction.

```
        WLOAD=   47.0 kN
        PAWL = 0.0165
        TPRES= 0.50  N/MM2
        RAD  =  173. MM

             VVVVVV
     -------------------------

   LAYER NO.1 ASPHALTIC

   THICKNESS =  100.  MM
   MODULUS   =  5000. N/MM2
   POISS RAT =  0.40
                              RADIAL STRAIN =   -20.
     -------------------------
                              VERT STRESS   = 0.4466 N/MM2
                              VERT STRAIN   =    74.
   LAYER NO.2 ASPHALTIC

   THICKNESS =  100.  MM
   MODULUS   =  4500. N/MM2
   POISS RAT =  0.40
                              RADIAL STRAIN =  -169.
     -------------------------
                              VERT STRESS   = 0.1213 N/MM2
                              VERT STRAIN   =   414.
   LAYER NO.3 GRANULAR

   THICKNESS =  150.  MM
   MODULUS   =   300. N/MM2
   POISS RAT =  0.24
                              RADIAL STRAIN =   -76.
     -------------------------
                              VERT STRESS   = 0.0694 N/MM2
                              VERT STRAIN   =   275.
        SUBGRADE
   MODULUS   =   260. N/MM2
   POISS RAT =  0.25

                         SURFACE DELECTION  = 0.2204  MM
                         EFF SURFACE MODULUS=   736. N/MM2
                         EFF POISSONS RATIO =  0.25
```

Figure 8.6 (c) Trial pavement designs. Design Three.
Block surfacing over granular sub-base.

```
        WLOAD=  47.0 kN
        PAWL = 0.0165
        TPRES= 0.50  N/MM2
        RAD  =  173. MM

             VVVVVV
  -------------------------

 LAYER NO.1 80 MM CONCRETE BLOCKS

 THICKNESS =  130.  MM
 MODULUS   =   300. N/MM2
 POISS RAT =  0.20
                         RADIAL STRAIN =   -69.
  -------------------------
                         VERT STRESS   = 0.4121 N/MM2
                         VERT STRAIN   =  1270.
 LAYER NO.2 GRANULAR

 THICKNESS =  375.  MM
 MODULUS   =   300. N/MM2
 POISS RAT =  0.24
                         RADIAL STRAIN =   -93.
  -------------------------
                         VERT STRESS   = 0.0855 N/MM2
                         VERT STRAIN   =   338.
       SUBGRADE
 MODULUS   =   260. N/MM2
 POISS RAT =  0.25

                    SURFACE DELECTION  = 0.5974  MM
                    EFF SURFACE MODULUS=   271. N/MM2
                    EFF POISSONS RATIO =  0.25
```

The four designs are then compared in terms of cost of construction. In this example the relative costs adopted are those given earlier in Chapter 2, they are representative of a typical site in southern Britain.

(i)	block surfacing	800
	150 mm lean concrete	375
	total	1175
(ii)	100 mm asphaltic surfacing	550
	100 mm asphaltic base	550
	Total	1100
(iii)	block surfacing	800
	375 mm type 1 material	280
	total	1080
(iv)	precast rafts, 140 mm	1950

It is clear that, with the exception of raft paving, there is very little difference in the cost of construction for the various designs. The structure with block surfacing and granular sub-base would be expected to deform and settle due to compaction over a period of time more than the other constructions. Whilst any relevelling that may be necessary would be straight-forward, it could prove inconvenient and may not justify the initial cost savings involved.

These designs are now compared to results given by the C&CA

recommendations for block pavements in industrial paved areas. This method assesses vehicle trafficking in terms of "standard axles".

Total No. vehicle repetitions = 300 x 365 x 15

= 1.64 million

The most damaging combination of load and frequency is the same as before, a 20 tonne load, thus the equivalent number of standard axles per vehicle is as follows :-

Axle	Load (t)	Standard Axles
1	4.1	0.07
2	9.4	2.0
3	8.3	8.3
4	8.3	1.3
Total	30.1	4.7

Total No. standard axles = 4.7 x 1.64

= 7.7 million

Road Note 29 gives the following designs :-

	Asphaltic	Cementitious
Surfacing (asphaltic)	100 mm	110 mm
Road-base	120 mm rolled asphalt or 200 mm D B M.	200 mm lean concrete
Sub-base (type 1)	150 mm	150 mm

The recommendations by the C&CA reduce the construction thickness for block construction by 225 mm, giving a lean concrete roadbase of 100 mm over 150 mm of type 1 granular material. The cost of this construction is the same as that of the structure given by the design program and the critical radial strain in the lean concrete is not exceeded (actual value is -21 microstrain). The designs can, therefore, be considered as equivalent.

8.3.2. Design using design charts.

It is proposed to construct a roadway between the ship's loading ramp and the container stacking area. It is anticipated that 600 20 ft units will be unloaded and driven along this roadway per ship, with two ships per week. These will be handled by 3 similar front lift trucks, Barton RoRo/20CH, the vehicles returning on the opposite side of the roadway with a container for export. In two years time there is expected to be a 50 % increase in throughput, when 40 ft containers will be introduced. It is planned to operate two larger FLTs, Knapper RoRo/40CH, to handle these and they will spend about 80 % of their time on the 40 ft units.

The subgrade is a reasonably well graded sand/gravel, no substantial settlement is anticipated, CBR 20 %. Design life 15 years.

The two front lift trucks used here were described in detail in the example on calculation of PAWL ratings in Chapter 6. The basic dimensions for the Knapper and the Barton, together with PAWL ratings, were given in Chapter 6, Table 6.2.

It is assumed imports equal exports, thus outward and return pavements are identical.

In the first two years :-

Vehicle movements, Barton 20 = 2 x 600 x 52 x 2

= 124,800

Throughput after 1982 will be 900 containers per ship divided into :- 60 % Barton 20

40 % Knapper 40

Vehicle movements, Barton 20 = 2 x 900 x 0.60 x 52 x 13

= 730,080

Knapper 40 = 2 x 900 x 0.40 x 52 x 13

= 486,720

Thus, total loading :- Barton 20 = 854,880 movements

Knapper 40 = 486,720 movements

To establish the total number of PAWL's the pavement has to carry, multiply by the respective average PAWL rating for the trucks.

Total No. PAWLs = 854,880 x 10.77 + 486,720 x 24.56

= 21.16 million

For the design, a single wheel load is adopted, this is assumed to be the most damaging wheel load for the whole load spectrum, 36.7 PAWLs in this case.

Design wheel load = 36.7 PAWLs

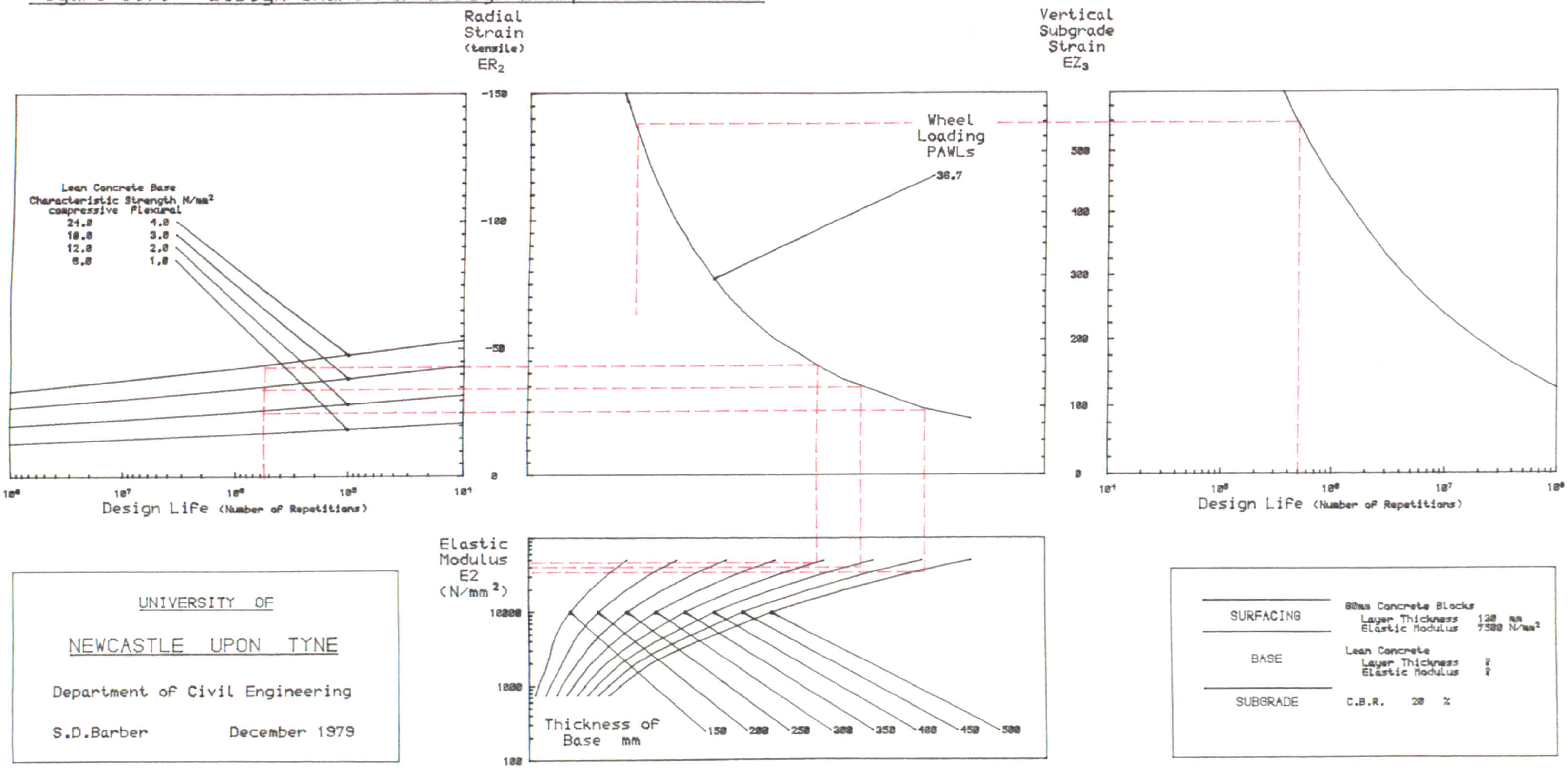
Design life required = 21.16 / 36.7 million

= 0.58 million repetitions

Referring to the design chart for a subgrade CBR of 20 %, Figure 8.7, it is clear that for this design vertical subgrade strain, on the right hand side, is not critical. On the left, a line is drawn vertically from required design life, 0.58 million, to intersect all curves. From each intersection, a line is drawn horizontally to the centre graph, the next intersection being taken at the design PAWL rating (36.7 PAWLs). From here drop vertically to the bottom graph.

Here the starting point is the abscissa, the elastic modulus of the base. This is taken from the typical elastic moduli given in Chapter 3, Table 3.1. The design thickness is found from the

Figure 8.7. Design Chart for Design Example. CBR 20 %.



intersection of the elastic modulus with the relevent line from above. Care must be taken to correlate the correct base modulus with the characteristic strength used in the first graph.

The following designs for the base course are obtained :-

Compressive strength N/mm2	24	18	12
Elastic modulus N/mm2	48000	42000	35000
Thickness required mm	350	400	500

From another design chart it is found that if a 150 mm granular sub-base, type 1 material, is used, then base thicknesses can be reduced to 300, 350 and 500 mm respectively. It can be seen that for a given operational situation, the charts can be used to produce a series of designs permitting the engineer to choose the one he favours.

The results from the design chart are now compared to the alternative method given in ADS/36. This method is based on the number of repetitions of the heaviest vehicle and the maximum wheel load.

For the Knapper :-

No. repetitions		490,000
axle load (31.5 t load)		77.5 tonnes
tyre spacing on axle		600 mm
tyre pressure		0.8 N/mm2
tyre contact area	=	775 / (4 x 800)
	=	0.24 m2

From the nomograph giving equivalence factors for dual wheels, Figure 8.8, the correction factor is 0.775. Hence, design wheel load is :-

= 0.5 x 77.50 x 0.775

= 30.1 tonnes

Revised contact area = 0.38 m2

From the design chart, Figure 8.9, base design thickness is 470 mm. This value is then reduced by 225 mm to allow for the block surfacing, thus the final design thickness is 250 mm of lean concrete.

It is clear from this comparison that the results from the ADS/36 method are similar to those given by the charts until the reduction of 225 mm is made. The basis of this reduction factor was outlined in Chapter 5, the blocks being equated to 225 mm of "asphaltic surfacing and lean concrete roadbase". Current U.K.

Figure 8.8 Nomograph for equivalence factors for twin tyres, from ADS/36

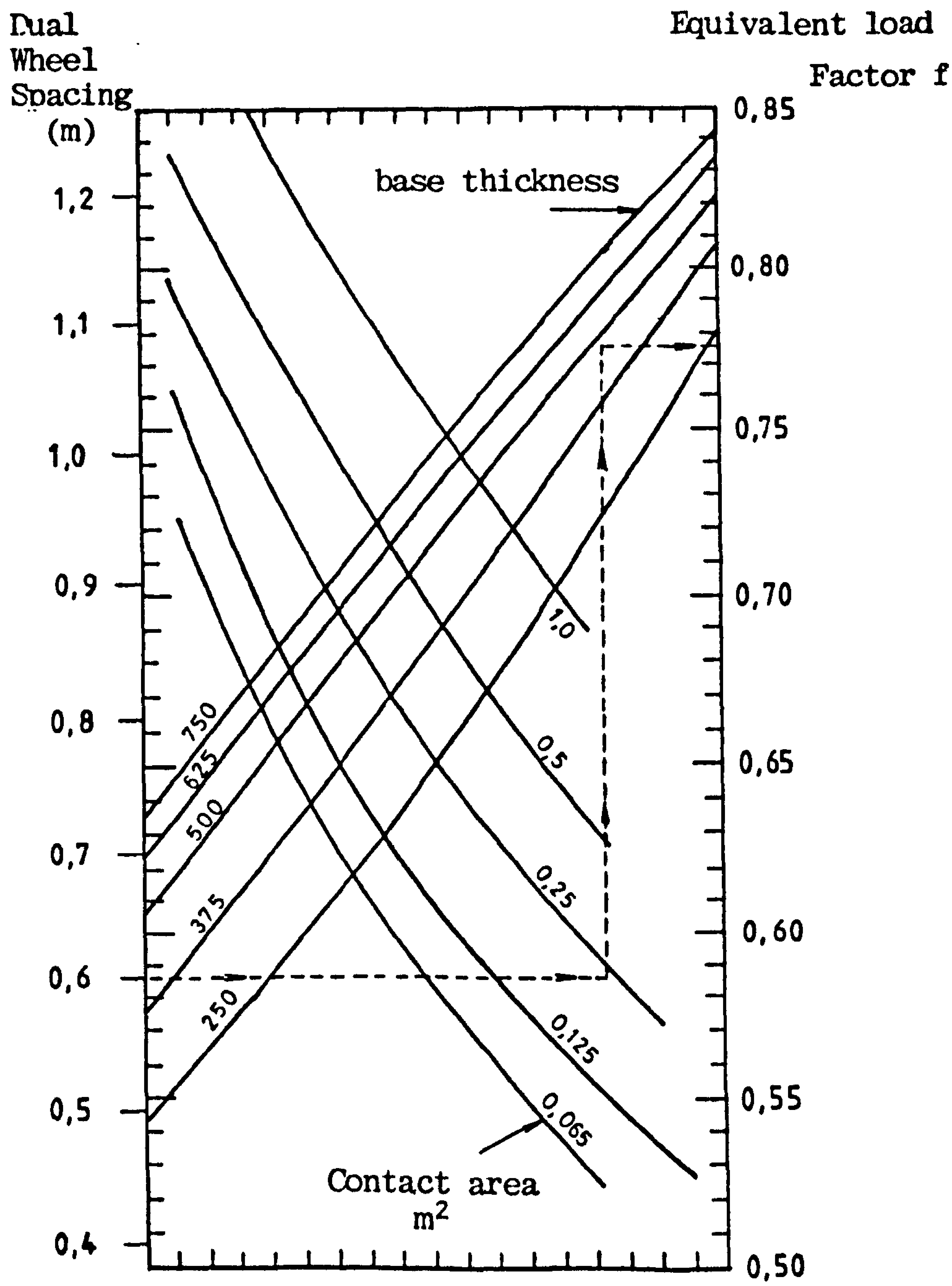
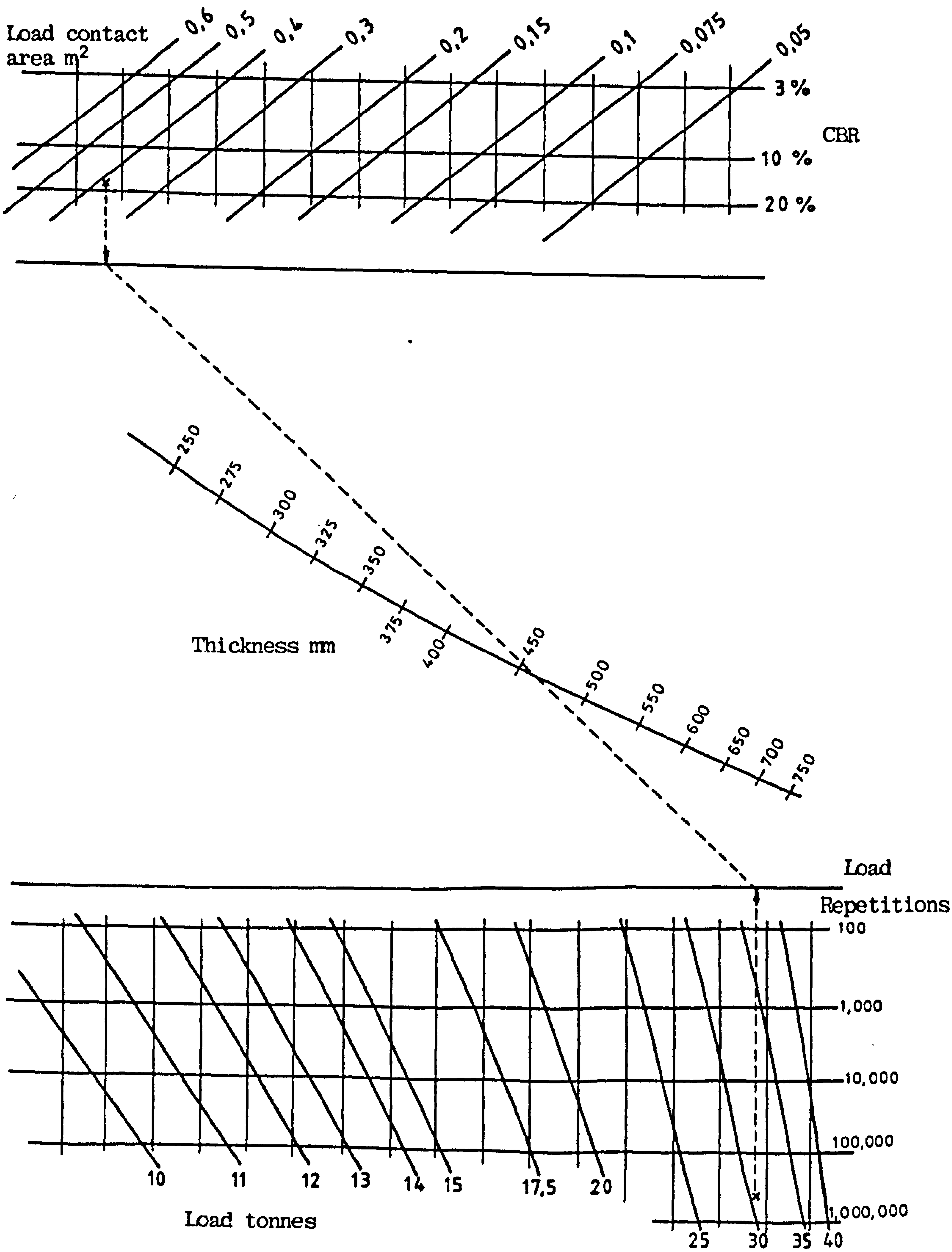


Figure 8.9 Design chart for cement bound base materials, from ADS/36



construction practice would require an asphaltic surfacing of at least 75 mm to prevent reflective cracking and under these conditions the original design method specified a maximum base reduction of 25 mm. This has been completely neglected in the revised design method and it is clear that this assumption severely reduces the overall strength of the pavement.

8.4. Conclusions.

The Chapters of this thesis have described in detail the development and application of a design method for flexible pavements in port areas. Whilst a basic framework for the method has been laid down, covering the principles of analysis and design, there are several areas in which the method can be developed further. One of the widest areas in which to extend this is in the characterisation of paving materials. Chapters 3 and 5, covering material characterisation and concrete blocks, presented data and information which has not been incorporated in the design method. Two particular topics referred to were non-linear stress/strain relationships, and the prediction of rut depths throughout the pavement's design life. Both these subjects require further research, but the simplified approaches described in these Chapters could be applied to this method.

Other areas which deserve further investigation are the effects of temperature (on both bituminous and cement bound materials), a coherent approach to characterisation and specification of cement bound base materials and a rationalisation of design for segmental block paving. Also, whilst the loading analysis presented here has made it possible to assess the damaging effect of container handling vehicles, further research is required on multiple gear loads and their relationship to overall pavement strength.

Finally, it is clear that this design method is not solely restricted to pavements in port areas, it can be readily extended to include other industrial or highway pavements. Further, since the mathematical calculations are relatively straight-forward, the method can be made available on desk top computers, giving a much more flexible design tool than current recommendations or design charts.

8.5. References.

8.1. LILLEY A.A. and WALKER B.J. Concrete block paving for heavily trafficked roads and paved areas. Cement and Concrete Association, Wexham Springs. Publ. No. 46.023. 1978.

8.2. ROAD RESEARCH LABORATORY A guide to the structural design of pavements for new roads. 3rd edn. London. HMSO. 1970. Road Note 29.

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