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Innovative structural backfills to integral bridge abutments

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Executive Summary

Investigations have confirmed that most bridge deck expansion joints leak and contribute more than any other factor to deck and substructure corrosion from de-icing salts. For this reason joint-free integral bridges, with the abutments structurally connected to a continuous deck, are more durable and cheaper to maintain. However, thermal strains in an integral deck cause cyclic loading on the soil behind the abutments which may result in the development of passive soil pressures. Traditionally, thorough compaction of high quality granular backfill has been used behind bridge abutments to avoid problems associated with settlement of the backfill and carriageway. In an integral bridge situation however, better quality backfill accentuates the risk of high passive pressures developing.

Integral bridge design has therefore to either accommodate or avoid the high forces and moments in the structure that may be generated by the abutment moving into the backfill. One method of avoiding the development of high lateral pressures on the abutment is to use a low stiffness but compressible elastic cushioning layer. This, in turn, would allow a more economical design for the construction of new integral bridges. In addition the method may also provide an economical conversion of existing conventional bridges into integral structures as part of the need to reduce long-term maintenance costs.

This report has identified the loading requirements which will need to be met when using a stress absorbing layer behind an integral bridge abutment. Various compressible materials, for example polymeric and geocomposite materials, which may be suitable for use as innovative structural backfill behind integral bridge abutments have been investigated. The engineering properties of these materials are reviewed from available literature and their likely performance evaluated. The report will be of benefit to design engineers and enable economic and durable construction of integral bridges.

It is recommended that a performance specification is developed outlining simple test methods determining suitability for this application. These methods should place emphasis on long term cyclic loading effects, creep and durability of the selected material.

1 Introduction

1.1 Objectives

In an integral bridge, the abutments are structurally connected to a continuous deck, thus avoiding the bearings needed in more conventional road bridges. In turn, this obviates the need for deck joints and hence reduces the possibility of road de-icing salts causing long-term damage. However, seasonal thermal movements of the deck cause interactions between the bridge abutments and the retained soil. Card and Carder (1993); Springman, Norrish and Ng (1996); and England and Dunstan (1994) have identified that lateral earth pressures acting behind an integral abutment are likely to increase progressively with time because of thermal cyclic seasonal expansion and contraction of the bridge deck. Increase in earth pressures above conventional design values and up to the limiting passive pressure value, K_p, can occur and, together with foundation movement, result in design serviceability limits being exceeded and possible structural distress.

Traditionally, thorough compaction of high quality granular backfill has been used behind bridge abutments to avoid problems associated with settlement of the backfill and carriageway. In an integral bridge situation however, better quality backfill accentuates the risk of high passive pressures developing. Integral bridge design has therefore to either accommodate or avoid the high forces and moments in the structure that may be generated by the abutment moving into the backfill. One method of avoiding the development of high lateral pressures on the abutment is to use a low stiffness but compressible elastic cushioning layer. This, in turn, would allow a more economical design for the construction of new integral bridges. In addition the method may also provide an economical conversion of existing conventional bridges into integral structures as part of the need to reduce long-term maintenance costs.

The objective of this report is to identify the various compressible materials, such as polystyrene, which may be suitable for use as innovative structural backfill behind integral bridge abutments. The engineering properties of these materials are reviewed from available literature and their likely performance evaluated. The report will be of benefit to design engineers and enable economic and durable construction of integral bridges.

1.2 Background

Compressible materials are used in many ground engineering applications, such as void formers to ground beams and slabs to avoid high pressures from soil heave. These materials generally have a low density and this has also resulted in their use as light weight embankment fill materials (McElhinney and Sanders, 1992; Sanders and Seedhouse, 1994; Campton, 1995).

In North America there has been an increasing interest in the application of compressible polymeric products to reduce earth pressures on rigid retaining structures. Partos and Kazaniwsky (1987) describe the use of prefabricated drainage boards consisting of expanded polystyrene beads as a vertical compressible layer to reduce lateral earth pressures on a two level basement wall.

The general concept of a compressible layer was further developed by Horvath (1991) in studies to identify the effect of changes in stiffness and thickness of polystyrene backfill on the lateral earth pressures acting against rigid retaining walls and bridge abutments. Two possible concepts were identified:

1 *Reduced Earth Pressure (REP) wall*. Used alone, the compressible layer is inserted behind a rigid wall. This results in a reduction of lateral earth pressure exerted by the backfill on the wall due to mobilisation of the soil shear strength in the backfill to or below active conditions.

2 *Zero Earth Pressure (ZEP) wall*. This is similar to (1) but the retained backfill is also strengthened using soil reinforcement techniques. Mobilisation of tensile forces in the reinforcement, as the backfill yields and moves towards the compressible layer, results in near zero lateral pressures being exerted on the back of the rigid wall.

Use of the above techniques generally means more economical design as the retaining wall or abutment would otherwise have to accommodate high lateral earth pressures. In these applications the compressible layer is used to allow controlled yielding and mobilisation of shear strength in the backfill so that minimum earth pressure acts against the wall (Murray and Farrar, 1997). Figure 1 qualitatively illustrates the general concept of matching the stress-displacement behaviour of the backfill to that of the compressible material.

A similar concept might also be possible for integral bridge abutments with the aim of reducing earth pressures and achieving a more economical design. However, unlike a conventional retaining wall or bridge abutment, a compressible layer behind an integral abutment is subjected to cyclic loading from the deck and also lateral earth pressure from the backfill. In this situation the compressible layer has two main functions:

1 *On thermal contraction of the deck*. To expand near elastically allowing mobilisation of the shear strength in the backfill without creating a void into which debris and water can ingress.

2 *On thermal expansion of the deck*. To prevent the cyclic movements being transferred from the abutment into the backfill which would result in an increase in passive pressure.

A secondary function of the layer is to reduce wall friction between the abutment and the backfill. In this way the magnitude of passive earth pressure that can develop as the abutment moves towards the backfill during thermal expansion of the deck is also reduced.

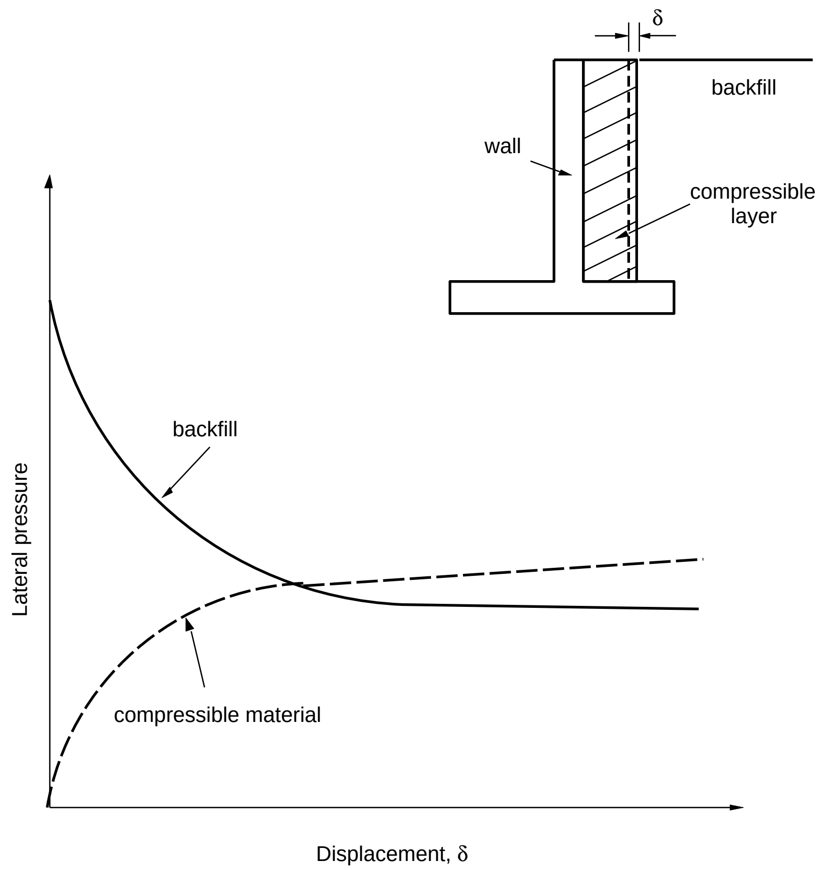

Figure 1 Design principle using a compressible layer (after Horvath, 1995)

2 Material requirements

Materials that may have the required engineering behaviour and can accommodate the likely movement behind an integral bridge abutment are:

- polymeric materials such as polystyrene and polyethylene products
- geocomposite drainage materials that might have the required characteristics because of an elastic polymer core or could be engineered to have the required properties
- rubber materials or composite products.

It must be noted that, with some exceptions such as natural rubber, the materials discussed in this report were only introduced from about 1930 onwards. Thus data regarding long term performance are therefore based on extrapolation techniques, rather than actual data.

Economic production of the material to meet the required specification may depend upon the size of the market. It is possible that a suitable grade of material for use as a compressible layer in this application will need to be specially manufactured. From the manufacturer's point of view the volume of material required per bridge will be small so the cost of an innovative product could be seen as relatively high. However, there are potential savings in costs by reducing the size of structural elements which, if offset against the production costs, could make manufacture of such a product attractive.

The compressible layer should satisfy the loading conditions imposed by the integral abutment and be durable throughout the design life of the structure, currently 120 years (Clause 4.1, BD 37, DMRB 1.3). In addition the method of construction or installation of a compressible layer behind the abutment will also influence the choice of material or product. For new construction the compressible layer can be installed relatively easily against the integral abutment prior to placement of the backfill. However, in the case of maintenance or strengthening of existing conventional bridges, where movement joints between the deck and the abutments are fixed or grouted to form an integral structure, installation of the compressible layer will require excavation behind the abutment.

The factors influencing material requirements are described below.

2.1 Loading conditions

The material properties required to meet the loading conditions for the compressible layer behind an integral abutment are as follows:

1 Under cyclic loading the mode of displacement of the abutment can be rotation about the base or translation or a combination of both. Typically for a 60m span integral bridge the maximum horizontal displacement at the top of each abutment is calculated as 16mm (ie. ±8mm) based on an effective deck temperature range of 46°C and a coefficient of thermal expansion of concrete of

12×10^{-6} per ^{o}C (Clause 5.4, BD 37, DMRB 1.3). The magnitude of this displacement is proportional to the bridge span and will therefore reduce for shorter spans. The compressible layer must be capable of deforming near elastically over the calculated range of horizontal movements produced by the thermal cyclic effects. In addition the behaviour of the compressible layer should ideally remain unaffected by repeated cyclic loading, ie. there should be no hysteresis effects causing stress/strain hardening that can lead to fatigue cracking and failure.

2 The compressible layer should be sufficiently stiff to transmit active earth pressures (K_a) from the backfill onto the abutment wall. These pressures are necessary to support the abutment wall and provide resistance to movement from longitudinal and vehicle braking loads. The magnitude of the active lateral earth pressures from free-draining granular fill acting at any depth (h) on the compressible layer can be calculated from $K_a\gamma h$. For a typical 6m high abutment and granular fill with ϕ' of 35^{o} (ie. K_a of 0.27), the active pressure distribution will increase linearly to a pressure of about 30kN/m^2 at 6m depth.

3 If the compressible layer is installed prior to backfilling behind the abutment, the layer must be able to resist compaction induced stresses in backfill material. This procedure can produce high transient stresses and residual lateral stresses against a rigid structure approaching the passive value (K_p) in the upper few metres of backfill. Guidance on the magnitude of compaction stresses and depth of penetration is given by Broms (1971), Ingold (1979) and Murray (1980). Experimental data reported by Carder et al (1977) indicate that residual lateral pressures due to compaction of granular backfill are of the order of 20kN/m^2 over the upper 2m of backfill. In an integral bridge situation, the compressible layer needs to be capable of absorbing the high stresses induced towards the top of the abutment by thermal deck expansion and, for this reason, the construction sequencing and method of backfilling needs to be such that the layer is not detrimentally preloaded by compaction.

4 The compressible layer should have adequate strength to resist superimposed vertical live loading. From Clause 5.8.2 of BD 37 (DMRB 1.3), the live load surcharge could be as high as 20kN/m^2. Use of a run-on slab may help to spread the vertical loading in some situations.

5 The layer should have adequate shear strength in the vertical direction. A vertical shear force (down drag) on the layer will be created by differential settlement of the backfill. If ϕ' is fully mobilised on the interface between the backfill and the compressible layer, this shear force is likely to exceed that mobilised by friction at the wall/ compressible layer interface. Assuming that the stress absorbing layer is acting effectively and as a block, a crude estimate of the minimum shear force that the compressible layer must resist can be based on “at rest” earth pressure conditions (K_o) for free-draining granular fill as follows:

$$\text{Total shear load} = 0.5K_o\gamma h^2 \tan\phi'$$

Assuming the same parameters as in (2), this is equivalent to about 96kN per metre run of abutment and equates to a vertical shear stress of 16kN/m^2 assuming the compressible layer is placed over the full height of a 6m high abutment. However, in reality, the stress absorbing layer will not be totally efficient and a lateral stress increase will occur towards the top of the abutment as the deck expands. If a K value of 1 is assumed to occur over the top metre of backfill, similar calculations of shear stress in this area give about 6kN/m^2 although the stress will act in the opposite direction.

6 The layer should also have adequate shear strength in the horizontal direction. Horizontal shear across the compressible layer can develop as a result of a change in lateral stress with depth on the material. As the layer deforms at different values of strain, horizontal shear stresses can develop between elements. The magnitude of the shear stress will depend on the cross-sectional area of the compressible material. The maximum horizontal shear force will occur near the base of the wall stem and can be estimated for an element of 1m height from the mean earth pressure at rest ($K_o\gamma h$) assuming the layer is satisfactorily absorbing stress induced by deck expansion. Using the same parameters as in (2), the maximum shear force on the element is then about 40kN and the corresponding horizontal shear stress would be about 130kN/m^2 for a compressible layer thickness of 0.3m. In practice this high shear stress is unlikely to develop because a layer of this thickness will tend not to act as a discrete block but deform more locally. In this case, the maximum shear stress will probably not exceed the lateral stress $K_o\gamma h$, ie. about 40kN/m^2. The use of granulated or discrete element compressible materials would assist in reducing the magnitude of horizontal shear stress that could develop at any particular depth behind the abutment wall.

From the above loading considerations and assuming the stress absorbing layer is acting reasonably efficiently, the change in horizontal pressure on the compressible material from granular backfill is likely to be of the order of magnitude of 50kN/m^2. If a maximum abutment movement of between 8mm and 16mm occurs and assuming a 0.3m thick compressible layer the required horizontal elastic stiffness ranges between 1.9 and 0.9MN/m^2.

The vertical settlement of the compressible layer should be similar to that of the retained soil in order to minimise differential settlement. However because of the significant difference in density between granular backfill and the materials considered in this report, the vertical loadings due to their self-weight will be different. For this reason equal settlement in both backfill and the compressible material (ie. strain compatibility) can be achieved under the different vertical loads with a lower vertical stiffness for the compressible layer. Typical values of drained stiffness for a well compacted granular backfill are in the range 10 to 30MN/m^2 and calculations taking into account the surcharge and self weight loads indicate the vertical stiffness of the lightest compressible material under consideration may need to be only about one third of the vertical stiffness of the granular backfill for strain compatibility, ie. a minimum of 3MN/m^2.

The typical requirements of a compressible layer are summarised in Table 1. For guidance the requirements for 0.3m and 1.0m thick layers are shown in order to provide an indication of the range in parameter values. The ideal horizontal stiffness is much smaller than the vertical stiffness for a compressible layer. This requirement could impose limitations on the choice of materials or products that would form a satisfactory compressible layer. However, adjustment of the thickness of the compressible layer enables the range of materials to be expanded. Alternatively, this problem could be overcome if vertical loading onto the compressible material is prevented by the use of a run-on slab to the integral bridge.

Table 1 Typical requirements for a compressible layer

Material property	*Layer thickness (m)*	*Typical requirement*
Horizontal elastic stiffness, MN/m^2	0.3	0.9 - 1.9 [Strain 2.7-5.3%]
	1.0	3.1 - 6.2 [Strain 0.8-1.6%]
Vertical stiffness, MN/m^2	All	3 - 30 [1]
Minimum horizontal shear strength, kN/m^2	0.3	130 [2]
	1.0	40
Minimum vertical shear strength, kN/m^2	All	16

1 Engineering of the vertical stiffness will depend on material density and may not be critical if a run-on slab is employed.

2 This theoretical value is unlikely to exceed the lateral stress of about 40kN/m^2 developed by the backfill.

It should be noted that the compressible layer will be subject to both normal and shear forces as a result of the loading conditions described above. For some materials or products it may therefore be a requirement to carry out performance testing under combined normal and shear forces.

2.2 Material durability

The engineering behaviour of the compressible layer will also depend on material durability and its resistance to degradation during the design life. The key factors are discussed below:

1 The material should be resistant to chemical and biological attack from agents in the soil and groundwater. This will include resistance to de-icing salts, acids, alkalis, sulphates and chlorides, etc. as well as petroleum hydrocarbons and other organic compounds that can soften or erode polymeric materials such as polystyrene

2 The material should be non-combustible or be protected from naked flames

3 The material should be non-absorbent to moisture or not retain moisture by capillary action. Moisture retention could allow chemical or biochemical attack as well as physical breakdown from frost action and water freezing within the material. Moisture would also increase the bulk weight of the material and this might not be desirable

4 The material should not become embrittled with age or the effects of cyclic loading.

2.3 Construction implications

The method of construction or installation of a compressible layer behind the abutment will influence the choice of material or product. There are two situations where a compressible layer may be considered. These are:

1 new construction where the compressible layer is installed as part of the design of the integral bridge

2 renewal or repair of an existing integral bridge or a conventional bridge where movement joints between the deck and the abutments are fixed or grouted to form an integral structure.

For new construction the compressible layer can be installed relatively easily against the integral abutment prior to placement of the backfill. In these circumstances there is little restriction on the product form that can be used, for example blanket roll, granules or beads, or solid blocks. However, compaction of the soil backfill against the compressible layer needs to be controlled to ensure that the layer is not detrimentally preloaded by compaction. This may not be a problem if the backfill is placed in hot weather as deck contraction will immediately relieve any high compaction stresses.

The installation of a compressible layer behind an existing structure is more difficult because of limitations on access and placement. Unless the backfill is excavated and removed, it will be necessary to excavate a trench to the required depth through the backfill and install the compressible layer from ground surface. In these circumstances a layer comprising a rigid blanket or granules or beads would be easier to install than a flexible blanket or large solid blocks of material.

The depth of the compressible layer needs careful consideration. Springman, Norrish and Ng (1996) advise that high lateral stresses will be induced over at least the top half of the retained height of a spread-base integral abutment and over at least the top two thirds of the retained height of an embedded wall integral abutment.

3 Polystyrene products

Polystyrene products are produced in two forms as follows:

1 expanded polystyrene (EPS)

2 extruded polystyrene.

To produce polystyrene foam, expandable polystyrene beads, containing an agent such as pentane, are expanded by dry steam up to 40 times their original volume to form a bead like material. The beads are then allowed to *mature* and then placed in a mould and further steam is added to induce final expansion and fusion to form a material of the required shape and mechanical properties. The macrofabric of the material comprises spherical polyhedra. The amount of expansion directly affects the packing of the polyhedra. The final density of the EPS block is primarily determined by the initial expansion of the beads.

Extruded polystyrene is formed, as the name suggests, by continuous extrusion of the foamed material through a mould, a process which builds-in a certain amount of compression into the material due to the cell structure shape on the outer skin.

In general, organic substances can undergo degradation when exposed to sunlight and ultraviolet radiation. Because of the inert styrene molecule, however, EPS is regarded as a stable polymer provided that temperatures remain below 50°C. Data for EPS aged over a 7 year period under no strain conditions indicate only a slight reduction in some mechanical properties as shown in Table 2. Further information is required on aging under cyclic loading.

Table 2 Ageing of EPS (after Vegsund, 1987)

Properties	*Initial EPS*	*Aged EPS after 7 years*
Density, kg/m^3	25	23.3
Compressive strength at 10% strain, kN/m^2	176.6	176.6
Modulus of elasticity, MN/m^2	7.5	6.9
Tensile strength, kN/m^2	343.4	402.2

** EPS aged at 24°C under no strain*

In terms of its chemical resistance, there are some substances that can attack polystyrene to a significant degree generally dissolving the material (Engineered Materials Handbook, 1988). These include:

- most hydrocarbons including methane, petrol and tar oils
- chlorinated hydrocarbons and chlorofluorocarbons (CFCs)
- other organic solvents such as ketones, ethers and esters
- concentrated acids.

Many paints, adhesives, cleaning fluids and construction products (eg. curing compounds, bituminous sprays or paints) contain organic solvents.

Although the above chemical products will damage polystyrene they will need to be present in large quantities or in high concentrations to have any significant effect on the engineering performance of polystyrene. If good quality clean granular backfill (Class 6N or 6P) is used behind an integral abutment in accordance with draft BA42 (DMRB 1.3) and the Specification for Highway Works (MCHW 1) the risks of chemical attack due to deleterious substances will be minimal. Nevertheless deleterious substances might be present in significant quantities in contaminated ground, or introduced by accidental spillage or during construction operations. With a compressible layer behind an integral abutment it would be possible to provide protective membranes such as high or low density polyethylene (HDPE or LDPE) over the exposed surfaces.

Polystyrene is stated to yellow slightly in sunlight (Harper, 1975) and non-stabilised polystyrene is stated to have ‘poor’ resistance to weathering of 0.5 years (Furness, 1995): no data for stabilised polystyrene are given. Polystyrene foam boards have been in use as a frost protection measure beneath roads in Sweden successfully for over 20 years (Gandahl, 1987).

Polystyrene materials have a low potential for water absorption, despite their low density. The individual beads that make up the materials have a closed cellular structure and therefore will absorb little water. The void space available for water uptake is, therefore, limited to the interconnected pore spaces between the beads, which form a small proportion of the total volume. Long-term water absorption tests on EPS indicated values less than 2% after 15 years (Brydson, 1989). Tests performed on EPS used as lightweight embankment fill all showed water contents below 1% by volume after 20 years in the ground (Frydenlund and Aaboe, 1996). Extruded polystyrene absorbs water to a much lesser extent, with water absorption by volume being generally in the range of 0.05% to 0.2% (Sanders and Seedhouse, 1994). Frados (1976) gives values of between 0 and 1% by volume for extruded polystyrene. Brydson (1989) indicates a water absorption value for extruded polystyrene foam 70mm thick to be less than 4% after 15 years.

Both expanded and extruded polystyrene materials have a very low Poisson’s ratio with a typical value of 0.1 (Duskov, 1994). They will, therefore, impose low lateral forces on adjoining structures. Tests on polystyrene fills placed in contact with a bridge abutment (Frydenlund and Aaboe, 1989) indicate that, for restrained fills, the lateral load applied to the adjoining wall is between 0.1 and 0.2 times the vertical surcharge load to the polystyrene. These loads are insignificant in relation to other loads likely to be imposed on the structure. In Norway it is normal design practice to assume a uniform horizontal pressure of 10kN/m^2 is exerted by an EPS backfill against adjoining abutments and walls (Aaboe, 1987).

Because of the different production processes, there are some inherent differences in the physical and mechanical properties of expanded and extruded polystyrene, as described below. It should be noted that polystyrene products do not develop their full strength until approximately 2 days after moulding.

3.1 Expanded polystyrene

Expanded polystyrene (EPS) has a variety of uses in the construction industry, typically fill material, void formers, ground beam shutters, insulating layers etc. Horvath (1995) describes the use of EPS as a compressible inclusion to reduce lateral earth pressures acting on retaining walls.

For many of its industrial applications the most important characteristic of expanded polystyrene is its very low density. Densities that can be achieved range from 50kg/m^3 down to 8kg/m^3 but it is usually produced in four different grades in accordance with BS3837:Part 1:1986 with densities as shown in Table 3.

Figure 2 shows the typical stress strain behaviour of EPS in unconfined compression. It behaves as a linear elastic material up to strains of around 1% to 2% where yield occurs. Thereafter the material shows typical strain hardening behaviour in strain-controlled tests with the compressive stress continuing to increase, but at a decreasing rate, with increasing strain up to about 50% strain. Horvath (1995) has shown that at strains typically

Table 3 Densities for standard EPS grades (after Sanders and Seedhouse,1994)

Grade (BS3837:Part 1: 1986)	*Moulded density (kg/m³)*
Standard duty (SD)	15
High duty (HD)	20
Extra high duty (EHD)	25
Ultra high duty (UHD)	30

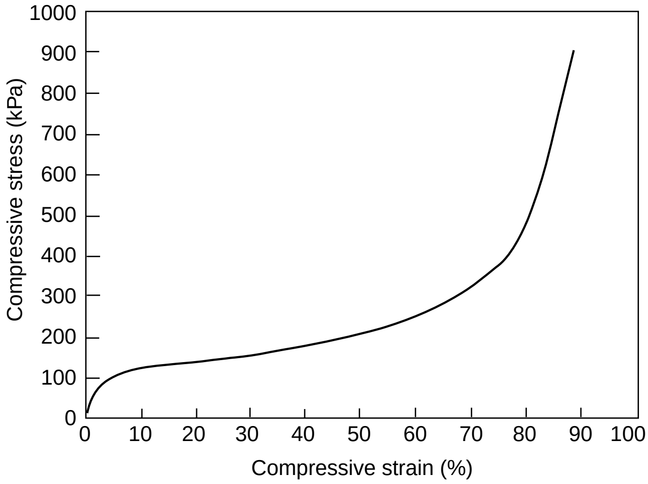

Figure 2 Typical stress-strain behaviour of EPS under uniaxial compression (Horvath, 1995)

greater than 50% the compressive strength of the material increases exponentially with strain. This occurs as the EPS structure collapses and the material becomes *elasticised*, see Section 3.2. Tests undertaken at TRL show similar stress-strain behaviour for various grades of EPS available in the UK, as shown in Figure 3.

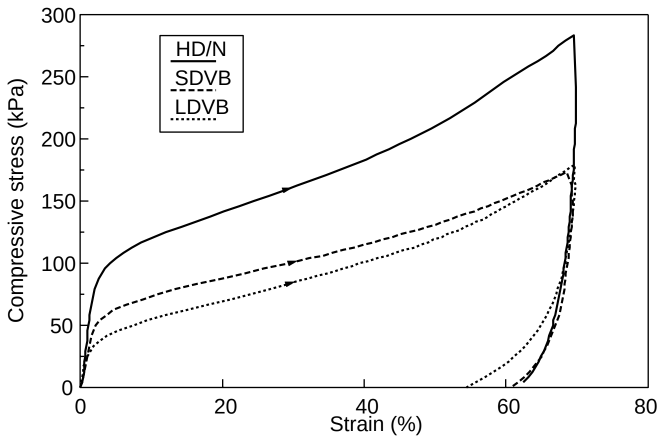

Figure 3 Stress-strain behaviour of different grades of EPS

The rate of strain is also an important factor in determining the strength of the material. Horvath (1995) provides idealised stress strain curves for EPS under rapid loading and a load duration of one year as shown in Figure 4. The latter conditions are more consistent with the rate of

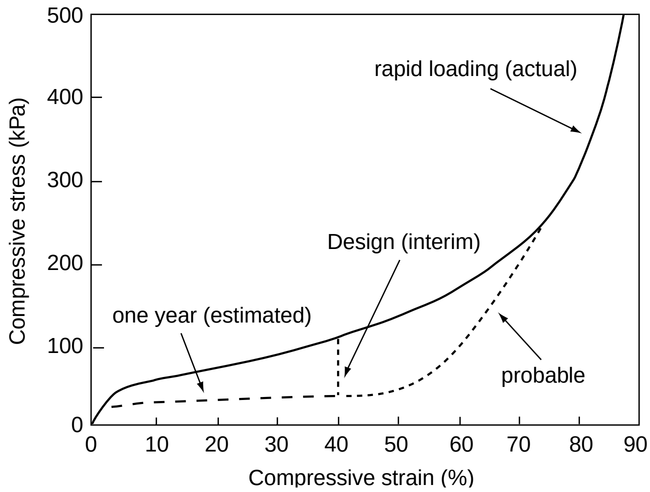

Figure 4 Stress-strain behaviour of EPS at various strain rates (Horvath, 1995)

loading that would be experienced by a compressible layer behind an integral bridge abutment.

Although polystyrene materials are graded according to the maximum compressive strength, this parameter has little practical significance. For engineering purposes the stress at which yield occurs is more significant as, if this is exceeded, non-recoverable plastic strains will accumulate under repeated loading. In general the yield stress may be safely taken to be the compressive stress at 1% strain. The unconfined compressive strength developed at 1% and 10% strain for various grades of EPS is given in Table 4.

Table 4 Typical strength properties of EPS (after Sanders and Seedhouse, 1994)

	Standard duty	*High duty*	*Extra high duty*	*Ultra high duty*
Density (kg/m³)	15	20	25	30
Compressive stress (kN/m² @ 1% strain)	21	45	70	100
Compressive stress (kN/m² @ 10% strain)	70	110	150	190
Shear strength (kN/m²)	90-120	120-150	150-190	190-220

Tests undertaken by Frydenlund and Aaboe (1989) indicate that the compressive strength and elastic behaviour of polystyrene are maintained even after repeated loadings of up to 80% of its yield strength, ie. at low strains of up to 1% before yield occurs.

Creep tests at constant loads indicate that the tendency for long-term creep deformations increases with the applied stress. Where the stress is such that the initial compressive strain is around 1% the rate of creep is small. Creep appears to be negligible if the initial strain does not exceed 0.5%. Tests undertaken at 20°C indicate a rate of creep of around 0.2% per year for this condition. The effect of increased temperature does, however, appear to cause an increase in the rate of creep. For the same relative stress level, low density EPS (typically less than 16 kg/m³) tends to creep somewhat more than higher density EPS (Horvath, 1995).

Magnan and Serratrice (1989) undertook laboratory tests to determine creep under cyclic loading. Their findings indicate that as long as cyclic loads are below the yield stress no creep deformation occurs. This is supported by the results from long-term monitoring (Aaboe, 1987) which show little apparent increase in compressive strain with time within polystyrene fill used for highway embankments. Horvath (1995) published data indicating that plastic deformations can occur even though stresses are within the elastic limit (ie. less than about 1% strain) although the reliability of these data is uncertain.

In an attempt to simulate vehicular loading on an EPS sub-base Duskov (1994) carried out dynamic load tests in a triaxial cell (axial strain up to 0.51%) with some 270,000 cycles of load (frequency 3Hz). Permanent deformations of the EPS samples were recorded although, in every case, these were not considered to be significant. The elastic modulus was found to range between 6.1 and 8.3MN/m^2 and Poisson's ratio varied between 0.07 and 0.11 on completion of the tests.

EPS has a high shear strength in relation to its compressive strength. Typical shear strengths for various grades are shown in Table 4: data from some manufacturers suggest that these values may be on the low side.

For use as a compressible layer behind an integral abutment subject to cyclic loading it is important that the compressive strength and elastic behaviour of EPS remain constant throughout the design life of the bridge structure. The potential for creep must also be negligible. To ensure this a safe design assumption would be to limit the lateral strain within the material to less than 0.5%. Thus under serviceability design limit conditions provided the abutment backfill loading onto the polystyrene is low compared to its yield strength the potential for creep is very low.

3.2 Elasticised expanded polystyrene

It is possible to produce elasticised EPS which is a spongy material formed by uniaxial compression of EPS blocks by 50% or more and then unloading. The reason for using this term is that the elastic limit is extended significantly beyond the 1% strain level for normal EPS. Elasticised EPS is currently mainly manufactured in Germany and is as yet only made on a limited basis in the UK.

The change in properties is caused by the EPS being permanently transformed into a mechanically anisotropic material due to permanent distortion of the cellular polyhedra from spherical to ellipsoidal and rupture of a proportion of individual cell walls within the polyhedra. Stiffness is permanently reduced in the direction parallel to the short axis of the ellipsoids. The extent to which EPS becomes elasticised depends on its original density and the magnitude of the strain to which the material is subjected before unloading. The optimum strain level for creating elasticised EPS appears to be between 60% and 70%, at least for EPS with a density of 12kg/m^3. Elasticised EPS exhibits linear elastic stress-strain behaviour up to strains of about 30%. In addition the initial tangent Young's modulus is reduced by a factor of about 10 to about 0.2MN/m^2 to 3MN/m^2, as shown in Figure 5.

Another unusual property of elasticised EPS when compressed in one axis is that it tends to *waist* in the other axis instead of bulging as might be expected. It therefore exhibits a negative Poisson's ratio in compression which may also be related to cellular collapse.

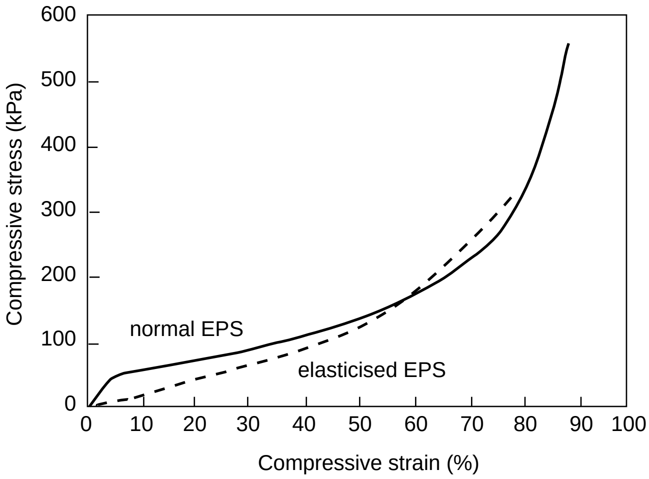

Figure 5 Comparison of stress-strain behaviour in uniaxial compression of EPS and elasticised EPS (Horvath, 1995)

Elasticised EPS is anisotropic in that its stiffness is much reduced along the compressed axis compared with its stiffness along the other two axes. This could be an advantage if the material is used as a compressible layer behind a bridge abutment as it would give more vertical support to a run-on slab whilst remaining more compliant in the direction of abutment movement.

Figure 6 shows the stress-strain behaviour of elasticised EPS and the effect of three consecutive loading cycles carried out at TRL. The material was initially elasticised by compressing a block of standard duty EPS by 70% of its original thickness. With the elasticised material each increment in load showed a similar near linear increase in strain with an approximate elastic stiffness of 0.25MN/m^2. However, in these consecutive tests there was an increasing amount of non-recovered deformation on unloading, although over a longer period of about a day the material recovered to near its original thickness.

3.3 Extruded polystyrene

Extruded polystyrene is generally denser than EPS and, as a consequence, is more expensive to manufacture. The current British Standard applicable to boards made from extruded polystyrene is BS3837:Part 2:1990. They are produced in various grades labelled E1 to E7 although only grades E3, E4, E5 and E7 have adequate strength and are suitable for civil engineering purposes. Strength values for the various grades are shown in Table 5.

As shown in Figure 7 extruded polystyrene shows linear elastic behaviour over a similar range of strain to EPS. After yield occurs the material shows strain softening behaviour in marked contrast to EPS. In strain controlled tests a peak compressive strength is reached at a strain of about 5% and this value is generally quoted as the compressive strength. At greater strain the compressive stress reduces approaching a constant stress at a strain of about 10%.

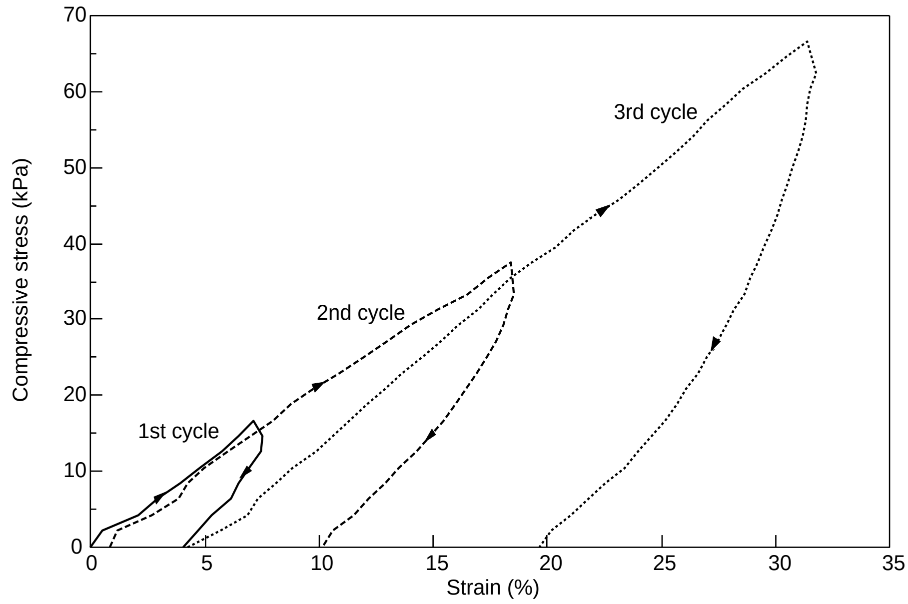

Figure 6 Effect of three consecutive loading cycles on elasticised expanded polystyrene (standard grade)

Table 5 Typical compressive stress of extruded polystyrene (after Sanders and Seedhouse, 1994)

	Grade E3	*Grade E7*
Density (kg/m^3)	28	55
Compressive stress (kN/m^2) at 1% strain	135	370
Compressive stress (kN/m^2) at 10% strain	200 (200)	700 (250)

Values in parentheses are BS3837:Part 2:1990 property requirements

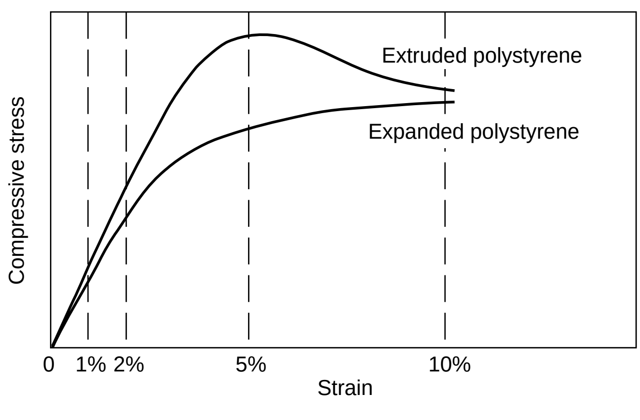

Figure 7 Typical stress-strain curves for expanded and extruded polystyrene (Sanders and Seedhouse, 1994)

Under cyclic loading and creep the behaviour of extruded polystyrene is similar to EPS, as discussed in Section 3.1, provided the initial compressive strains are low and not greater than about 1%. The material also has a high shear strength in relation to its compressive strength similar to EPS.

It must be noted that the above strength and stiffness values are for compressive loading in the direction perpendicular to the extrusion direction. The extrusion manufacturing process is such that the product exhibits significant mechanical anisotropy. The strength for loading parallel to the extrusion direction is generally about 25% of the value in the perpendicular direction, whilst the initial tangent stiffness may be only about 10% of the stiffness in the normally reported perpendicular direction.

Like EPS, extruded polystyrene could be suitable for use as a compressible layer behind an integral abutment subjected to cyclic loading although the direction of the mechanical anisotropy is of no particular advantage in this application. It is important that the compressive strength and elastic behaviour of the material remain constant throughout the design life of the bridge structure. The potential for creep must also be negligible. To ensure this a safe design assumption would be to limit the lateral strain within the material to less than 0.5%.

4 Polyethylene foam

Polyethylene can be supplied as a foam. It is more expensive than polystyrene but its properties are such that it would allow a much thinner layer of material for a given expansion and contraction range. Both materials can be produced at different densities and both suffer with production difficulties at very low densities. Polystyrene tends not to bond individual expanded beads together whilst polyethylene is difficult to foam reliably. This may cause a problem using either material as a compressible layer because the volumes involved are large.

As described by Beadle (1970), low density polyethylene is made by the high-pressure process of polymerising ethylene gas. High density polyethylene is made at low pressure using a catalyst. Polyethylene foam is a non cross-linked polymer material manufactured in planks to variable compressive strengths. It has high deformation characteristics due to a flexible closed cell structure and has primarily been used as a joint filler. It is recyclable and does not contain fully halogenated chlorofluorocarbons (CFCs). Typical mechanical properties of polyethylene foam are given in Table 6.

Table 6 Typical properties of various grades of Aerofil polyethylene foam (from Grace Construction Products Ltd)

Property	*Grade 1*	*Grade 2*	*Grade 3*	*Grade 4*
Density (kg/m^3)	35	60	105	145
Compressive strength (kN/m^2) at 10% strain	40	85	180	250
Compressive strength (kN/m^2) at 50% strain	110	170	340	450
Tensile strength (kN/m^2)	350	500	650	800
Tear resistance (N/mm)	1.2	2.5	4.0	4.5
Water absorption after 24hr immersion (% vol)	2	1	1	1

Other useful properties are: good processability, translucency, toughness, impact strength, flexibility, near-zero moisture absorption and low coefficient of friction. Originally polyethylene was used in radar equipment because of its excellent electrical insulation properties at all frequencies.

Polyethylene has good chemical resistance to a range of substances including most acids and alkalis, inorganic salts, organic acids, some hydrocarbons and sea water (Engineered Materials Handbook, 1988). However, it has lower resistance to some chemicals:

- poor resistance to aliphatic, aromatic and halogenated hydrocarbons and variable resistance to oils and greases (Snook, 1994);
- poor resistance to oxidising acids, concentrated nitric acid, benzene, carbon tetrafluoride, trichloroethylene, toluene, xylene (Engineered Materials Handbook, 1988).

Without stabilisation polyethylene is stated to have ‘poor’ resistance to weathering over one year (Furness, 1995): however, stabilised polyethylene is rated with ‘good’ resistance to weathering of over one year. The durability of polyethylene pipes is reported by Mruk (1990) who suggests that the only types of ageing mechanism that may limit this are photo-degradation, oxidative degradation and slow crack growth under tension; however, anti-oxidant and anti-ultra violet agents can be incorporated. Mruk reports many pipes have performed satisfactorily for 25 years in severely corrosive soils without significant strength loss.

5 Geocomposite materials

Geocomposite materials are used widely in civil engineering applications including drainage (fin drains), soil reinforcement and impermeable barriers: they may also be suitable for use as a compressible layer behind an integral bridge abutment. Generally geocomposite materials consist of a relatively rigid polymer core with an attached geotextile on one or both faces. The polymer core elements vary in size, shape and polymer type but all are used for their in-plane drainage capability. Non-woven geotextiles that cover the drainage core surface serve a dual role of filtration and separation. These are generally either of the needle punched or heat bonded type.

Geocomposite materials have been employed as fin drains in highway earthworks for many years and have to satisfy the test requirements given in the Specification for Highway Works, Clause 514 (MCHW 1) and have a current British Board of Agrement (BBA) Roads and Bridges Certificate. Under long-term lateral loading the geocomposite drain can reduce in thickness as the polymer core deforms. This will reduce the flow capacity of the drain. For this reason a composite fin drain must have adequate strength and low creep behaviour under long-term loading in order to function. The Specification for Highway Works, Clause 514 and Table 5/8 provides test criteria to ensure adequate drainage characteristics. Due to ingress of road debris and clogging, the normal replacement time interval for conventional roadside filter drains is around 10 years. However, geocomposite drains are not normally designed to accommodate surface water flows and clogging arises by fines migration from pavement foundation material and surrounding soils. The rate of deterioration of geocomposite drains will therefore depend on the grading of the fill material, but maintenance-free life is expected to be in the range 10-20 years. Given the 120 year design life for structures it is unlikely that geocomposite materials could be used as a combined compressible layer and back of structure drain without further development and performance evaluation.

Clause 6.3.10 of BD 30 (DMRB 2.1) states that proprietary drainage materials can be used as permeable backing behind bridge abutments and retaining walls provided they have a British Board of Agrement Roads and Bridges Certificate. The Specification for Highway Works (MCHW1) does not cover such materials, other than that Clause 513 requires details to be included in Appendix 5/1. The use of geocomposites as permeable backing on UK trunk roads has been limited. Requirements for use as a stress absorbing layer behind integral bridge abutments would be similar to that of a permeable backing in so far as the geocomposite would be subjected to lateral earth pressures exerted by the backfill. In both cases the geocomposite compressible layer would need to resist the applied pressure without failure over a 120 year design life.

Typical stress/strain characteristics of some geocomposite drainage materials are shown in Figure 8 and values of lateral strength and stiffness are given in Table 7 (after Geosynthetic Research Institute, 1995).

Table 7 Typical properties of geocomposite drainage materials

Property	*Typical range*
Horizontal stiffness (MN/m^2)	0.3 to 5
Ultimate compressive strength (kN/m^2)	800 to >1200
Deformation range at ultimate compressive strength (mm)	2 to >9

From Figure 8 it can be seen that there is a wide range in stress/strain characteristics reflecting the product assembly of the geocomposite. However, certain geocomposites may have suitable strength and stiffness characteristics to act as a compressible layer. Indeed it is likely to be possible to synthesize a geocomposite material with the desired stress/ strain characteristics for use as a compressible layer. In the UK BS6906 states that the compressive strength of all products should be taken at 10% deformation. This is normally well within the linear elastic range of the product. Given the low strain range requirements for a compressible layer it is likely that a product, of suitable thickness and stiffness, could be designed to comply with this requirement.

Some materials might be limited in terms of their peak strength and low stiffness characteristics and limited elastic recovery under long-term cyclic loading. These materials tend to have a non-structured resilient polymer core that undergoes large deformation when compressed by lateral earth pressures from backfill. They also have low shear strength characteristics (vertical plane through core) and are unlikely to resist the vertical down drag pressures exerted on the material by settlement/consolidation of the backfill resulting in excessive deformation and collapse of the inclusion.

Most geocomposite materials are less than 25mm in thickness and their deformation capacity is inadequate to accommodate the relatively large abutment displacements due to thermal cyclic movement, ie. ±8mm. In theory this could be overcome by increasing the thickness of the polymer core (or using multiple cores) to accommodate sufficient displacement. This is likely to be relatively expensive but clearly cost would be influenced by supply and demand.

There is limited information available on the long-term durability and performance of geocomposite materials. Brady et al (1994) undertook long-term studies, up to 30 years, on proprietary plastic products stored in various environments. The study identified that the performance of plastics was variable but there tended to be an increase in tensile strength and a reduction in ductility with increasing age. PVC incorporating certain organic plasticisers tended to become brittle with age because of loss of the plasticiser. In contrast high and low density polyethylene showed little change in stress-strain characteristics after 30 year ageing. For all plastics the initial elastic stiffness of the materials (for strains up to about 2% to 5%) was relatively unaffected. Durability considerations for some geocomposite products may therefore limit their use as a compressible layer in an integral bridge situation where typically a life of 120 years would be required.

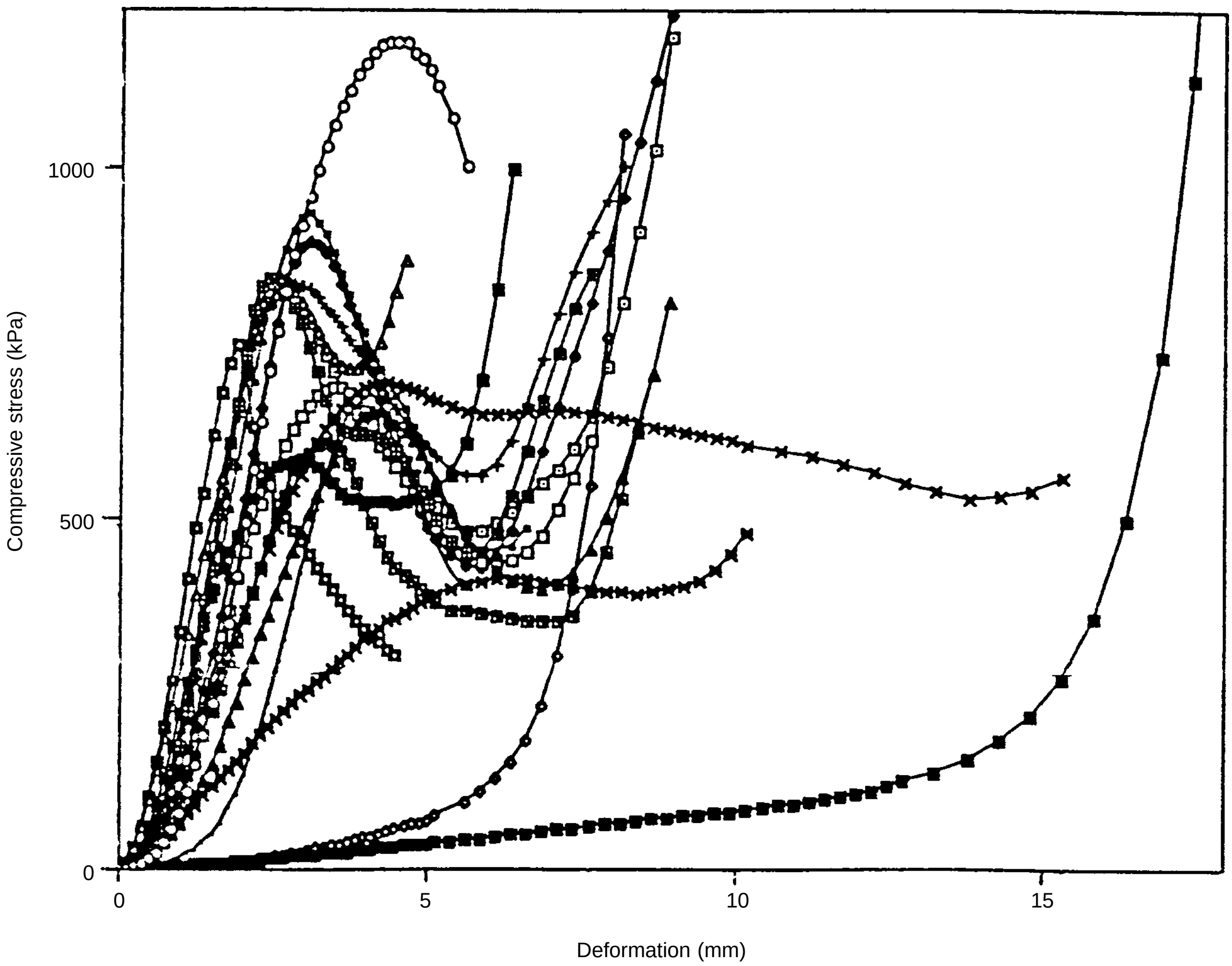

Figure 8 Stress-strain curves for a variety of geocomposite drainage core products (after Geosynthetic Research Institute, 1995)

6 Rubbers

Rubbers are defined as materials which may, under certain conditions, undergo large deformations and recover almost completely and instantaneously on release of the deforming forces (Freakley and Payne, 1978; Smith, 1993). Another definition is given in the Engineered Materials Handbook (1988): '*Cross-linked polymers having glass transition temperatures below room temperature that exhibit highly elastic deformation and have high elongation*'.

The elastic property is not a characteristic of a particular chemical substance, but derives from the molecular structure. The main requirements for a material to be a rubber are:

1 very long chain molecules (polymers) able to rotate freely about bonds joining neighbouring units
2 the molecules are cross-linked to form a three-dimensional network
3 apart from cross-linking the molecules should be free to move easily past each other.

The combination of 'polymer' and 'elastic' properties leads to the alternative name for rubber of elastomer. The term rubber was first attributed to 'natural rubber', but now applies in addition to an extensive range of synthetic rubbers. Products can be made with a very wide range of properties. This is achieved in three ways:

1 different compounding of the basic polymers
2 varying the vulcanisation or 'curing' process when the cross-links are formed
3 the addition of fillers and additives, such as carbon.

In general the properties of rubber vary with time and are affected by the environment in which it is used. For example, the stiffness modulus varies with temperature, rate of loading, stress level and stress history. Therefore any quoted property value has relevance only when the test method and conditions are known. Furthermore there are a large number of laboratory tests and standards applied in the rubber industry, and correlations between different test methods may not always hold. Smith (1993) suggests that laboratory test results in the rubber industry are generally used to establish satisfactory and reproducible results, rather than provide design information. They provide an indication only of the suitability of rubber products, but actual or simulated service tests are required to accurately predict suitability. Published property values do not always contain full information of testing conditions, nor does the scope of this report permit full reporting of this. Therefore, in this report, properties are provided in qualitative form or as indicative values. A selection of rubbers with typical properties and applications are shown in Table 8. Further information on different rubber products and general description of their properties is given in BS6716:1986 and Snook (1994).

The manufacture of rubber products is described by Smith (1993). Natural rubber originates from the *Hevea brasiliensis* tree and is extracted as an emulsion known as latex, which is coagulated and dried to produce crepe. Synthetic rubber is prepared by reacting monomers to form polymers as water emulsion or as a suspension in water or solvents. Elastomeric properties are developed by further compounding which is generally carried out in large mills or mixers. Fillers such as carbon black and silica may be added to stiffen, and oils, waxes and fatty acids added to improve processability but soften the product. Other additives are used to improve chemical resistance and help in the curing process. Sulphur is the vulcanisation agent in many rubbers. The compound is removed from the mill in the form of a sheet and then moulded by compression, transfer moulding, or direct injection. Vulcanisation or 'curing' results from the temperature and pressure of the moulding process.

Hardness is more commonly reported for rubbers than Young's modulus and shear modulus, see Table 8. Hickman (1996) indicates there is a linear relationship between hardness and Young's modulus, also by compounding and use of fillers the shear modulus can be varied over the approximate range 0.3 to 2.5MN/m^2, corresponding to Young's modulus of approximately 1 to 10MN/m^2. More details of the properties of natural and carbon filled natural rubber are listed in Table 9. The range of values is thought to be representative of all natural rubbers.

Although rubber is elastic, it is almost incompressible. This property enables rubber to be used in bridge bearings. By limiting the freedom to bulge with the introduction of steel plates, vertical deformation is reduced whilst horizontal movement is accommodated in shear. As a compressible layer behind an integral bridge abutment the lateral and vertical movement of a rubber block is likely to be constrained on both abutment and backfill surfaces, horizontal stiffness is likely to be excessively high. Assuming a single solid block of rubber 16m × 8m × 0.3m width is used as the compressible layer the horizontal stiffness is estimated to be about 300MN/m^2, based on the method of calculation given in BS6716:1986. This value, which is sensitive to the size of the rubber block, greatly exceeds the required horizontal stiffness as presented in Table 1. This can be overcome, however, by the introduction of holes or slots in the rubber block to allow lateral and vertical movement.

Alternatively the use of rubber in particulate form, such as shredded tyres (Section 6.1) or rubber soils (Section 6.2) may be appropriate for the integral bridge application.

The typical stress-strain curve of rubber is shown in Figure 9, which illustrates non-linearity. Furthermore for rubber to deform, molecular attractions have to be overcome by vibrational energy, therefore stiffness increases with decreasing temperature or increasing rate of deformation (visco-elasticity). Data contained in Lee (1994) suggest that this effect is significant only at relatively high rates of strain or frequency (greater than 0.1Hz) and with decreasing temperature. At high rates of strain or low temperatures rubber acts as a rigid solid. At low temperatures this effect is a result of crystallisation, which is reversible. Under load, rubber exhibits creep with time due to a slow breakdown of the vulcanised cross-linked polymers. When the load is released, some of the

Table 8 Typical rubber properties and applications

Name	*Chemical name*[1,3]	*Hardness range*[4] *(Shore A)*	*Tensile strength*[4] *(MN/m^2)* *Gum*	*Filled*	*Elasticity*[1]	*Creep*[1]	*Comp -ression set*[5]	*Rebound-cold*[5]	*Hysteresis properties*[1]	*Oil and petroleum resistance*[1]	*Oxidation and ozone resistance*[1]	*Service temp.*[4] *(°C)*	*Applications*[1]
Natural rubber (NR)	cis-Polyisoprene	30-90	>20	>20	Good	Good	Good-Excellent	Excellent	Good	Poor	Poor, without additives	-65 to 70	Commercial vehicle tyres, vibration mountings, dock fenders
SBR	Polystyrene-butadiene	40-90	<7	>14	Fair	Fair	Good	Good	Poor-Fair	Poor	Poor, without additives	-50 to 70	Car tyres
Neoprene (CR)	Polychloroprene	40-95	>20	>20	Good	Good	Fair-Good	Good	Good	Good	Better than NR, needs protection	-40 to 100	Vibration mountings, diver's suits, reservoir linings
Nitrile (NBR)	Polybutadiene-acrylonitrile	40-95	<7	>14	Fair	Fair	Good-Excellent	Good	Fair	Good	Better than NR, needs protection	-20 to 115[5]	Seals
Butyl (IIR)	Polyisobutylene-co-isoprene	40-80	>10	>14	Fair	Fair	Fair-Good	Poor	Poor	Poor	Better than NR, needs protection	-50 to 100	Tyre inner tubes, due to low permeability to gas
CSM	Chloro-sulphonated polyethylene	40-95	>17	>20	Fair	Fair	Poor-Fair	Fair-Good	Poor-Fair	Good[5]	Excellent	-40 to 135[5]	Sparking plug covers, reservoir lining for water and corrosive effluent
Butadiene (BR)	Polybutadiene	40-80	>10	>14	Good[6]	Fair	Good	Excellent	Excellent	Poor	Poor without additives	-60 to 100[5]	Long life tyre treads (in combination with natural rubber and SBR)
EPM, EPDM	Polyethylene-co-propylene-codiene	40-90	<7	>20	Good	Good	Good	Good	Good	Poor	Excellent	-40 to 150[5]	Cable sheathing, domestic appliances

1 Freakley and Payne (1978)
2 Smith (1993)
3 Morton (1987)
4 BS6716:1986
5 Snook (1994)
6 Hickman (1996).

Table 9 Selected physical constants of vulcanised natural rubber

Property	*Vulcanised natural rubber*	*Vulcanised natural rubber with 50phr[3] carbon black*
Density[1]	0.95 Mg/m^3	1.12 Mg/m^3
Young's modulus[1]	2 MN/m^2	6 MN/m^2
Bulk modulus[1]	2000 MN/m^2	2200 MN/m^2
Shear modulus[2]	0.38-1.42 MN/m^2	-
Shear stiffness[4]	1-2 MN/m at 25°C	-
Poisson's ratio[1]	0.4998	0.4995

1 MRPRA (1984)
2 Lee (1994)
3 Parts per hundred of gum rubber
4 Eyre and Stevenson (1991)

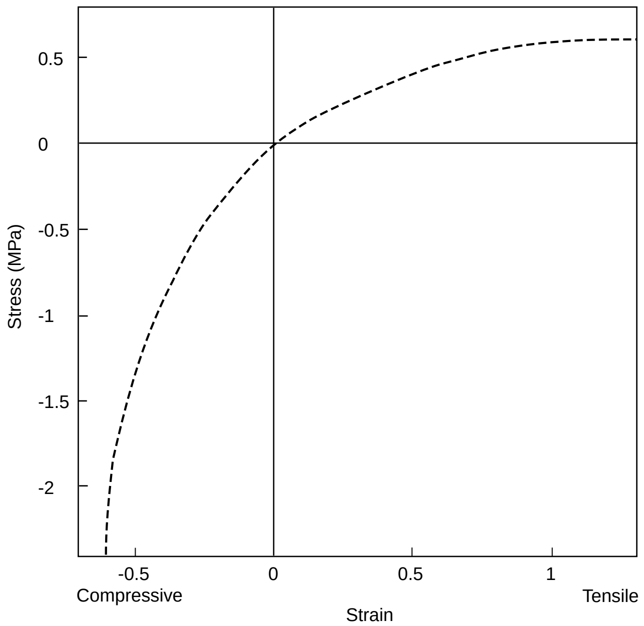

Figure 9 Uniaxial extension and compression curve for rubber (after Freakley and Payne, 1978)

deformation, known as 'compression set', is not recovered and the load-unload stress-strain curve exhibits hysteresis.

Creep is dependent on the type of rubber and temperature, as illustrated in Figure 10, showing that compression set may be less than 2% in some rubbers or more than 10% in many rubbers. A typical value of compression set for a low-creep natural rubber product is 5% after 3 days at 23°C (MRPRA, 1984). Smith (1993) suggests, however, that standard tests call for a recovery time of 30min, whereas compression set after 24 hours recovery can be as little as one-sixth of that after 30min recovery. A typical value of stress relaxation (loss of stress at constant strain) for the same material is about 2% over 10 years. These values of creep and set are thought to be acceptable in the application as a compressible layer to integral bridge abutments but behaviour over long periods requires further investigation.

Natural rubber in particular is susceptible to ageing effects. Oxidation aggravated by light results in loss of strength and elasticity (Freakley and Payne, 1978). Ozone causes surface cracking of rubber when under slight tensile strain. These effects can be reduced in the formulation of the rubber. The NRPRA (1969) suggests that ozone attack in natural rubber is a serious problem only when rubber is in tension, is small in section, and contains no protective anti-ozonant. The use of rubbers in bridge bearings provides indication of the durability of rubbers. Price and Fenn (1983) investigated the properties of elastomeric bridge bearings removed from two bridges in the UK and concluded that the bearings had suffered little, if any, deterioration in service and were likely to withstand a service life comparable to the 120 year design life of the bridge structure. Lee (1994) suggests that elastomeric bridge bearings can have a life comparable with that of the bridge and refers to an observed degradation of the surface of plain rubber pads in Australia limited to about 1mm after nearly 100 years service.

Natural rubber is commonly used in the UK but polychloroprene (neoprene) and chlorobutyl rubbers are also used. Polychloroprene has poorer performance than natural rubber which is preferable where temperatures drop below -10°C. Eyre and Stevenson (1991) studied the effect of low temperature stiffening on natural rubber and polychloroprene rubber bridge bearings and concluded that large increases in shear modulus can occur due to low temperature crystallisation of the material. The stiffening was dependent on the duration of exposure as shown in Figure 11. Furthermore stiffening of the rubber samples was accelerated when they were exposed to freeze/thaw cycles (between -25°C and +25°C). Such behaviour would not be desirable in a compressible layer since stiffening of the material would allow transfer of high stresses onto the integral abutment from the backfill. With regard to elastomeric bridge bearings, BS5400 Part 9 (1990) gives a performance specification for their use.

Some rubbers may absorb hydrocarbon liquids such as petrol and lubricating oils. In the case of natural rubber this absorption may amount to several times the volume of the rubber resulting in the reduction of mechanical strength. Attack by contact with oils is usually restricted to a thin surface layer due to slow diffusion rates. Lighter solvents will attack the rubber more rapidly (Hickman, 1996). Chloroprene is superior where oil or grease contamination is likely and temperature may exceed 60°C. Nitrile is used in corrosive crude oil conditions. Other properties of rubber are shown in Table 10.

In conclusion, it is thought that some rubber products would be suitable in the application in backfill to integral bridge abutments. However, low temperature stiffening is undesirable and thus the compressible layer will need to be well insulated against freezing climatic conditions, particularly against the effects of freeze/thaw cycles. The properties of creep, deformation and resistance to oxidation, ozone and petroleum products also require evaluation for each product. Rubber in particulate form will overcome the problem of high horizontal stiffness due to the incompressible nature of the material. Thus the use of shredded tyres and rubber soils may be appropriate as described below.

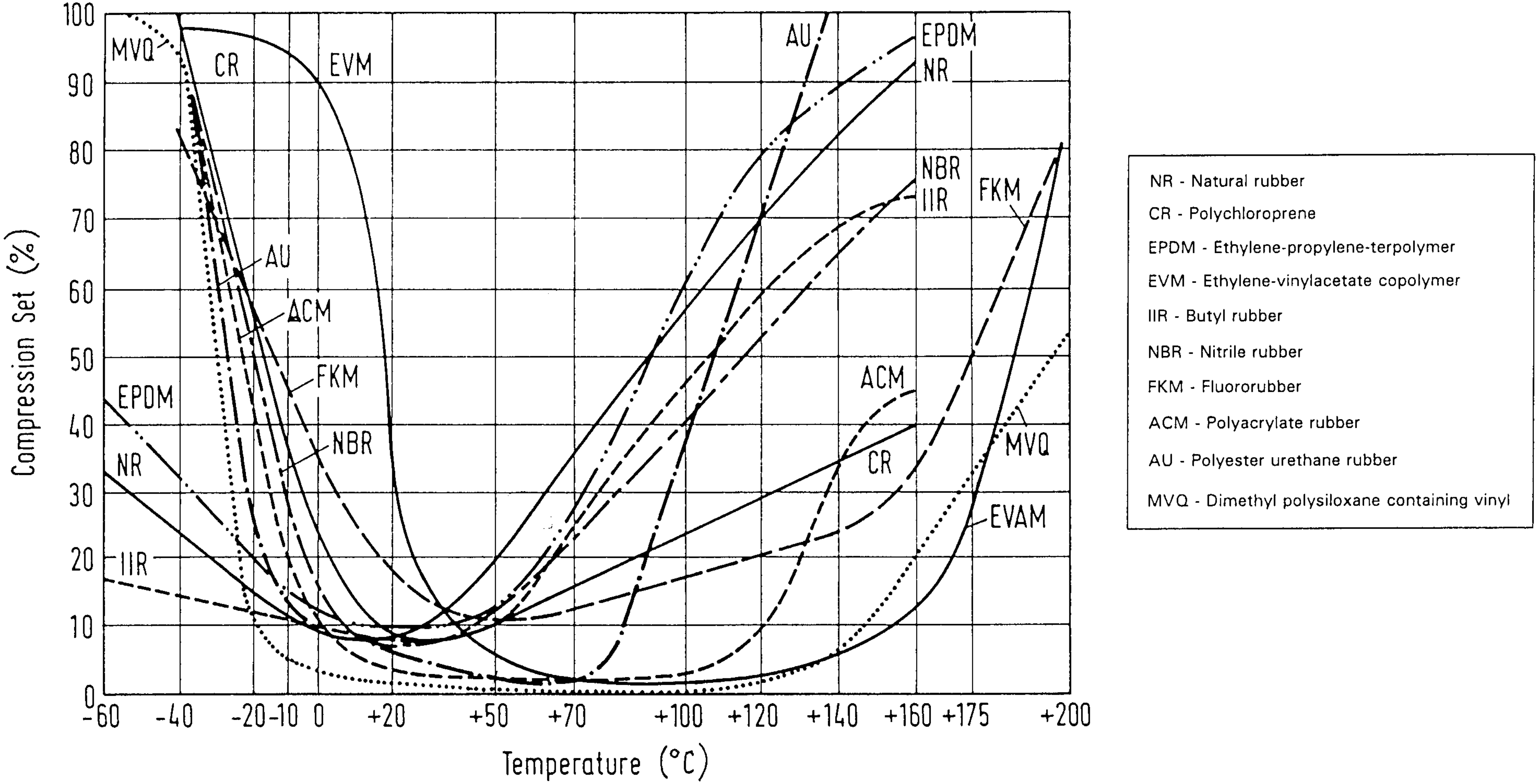

Figure 10 Compression set of some rubbers versus temperature (Hoffman, 1980)

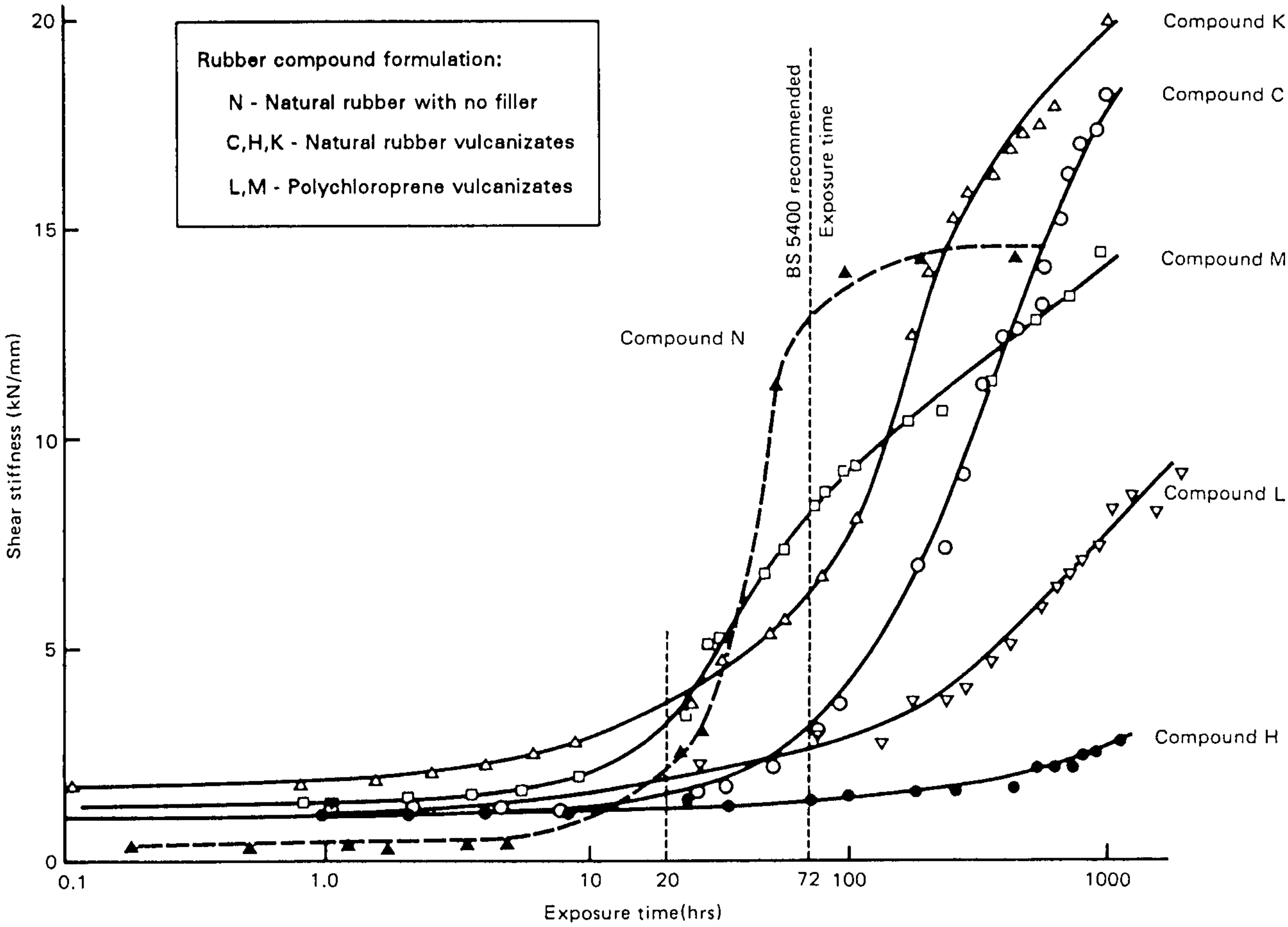

Figure 11 Increase in stiffness of full-size rubber bearings at -25ºC (Eyre and Stevenson, 1991)

Table 10 Miscellaneous properties of rubbers

Property	*Behaviour/typical value*
Heat conduction	Poor, typically 2.2 - 6.28W/m²K for 25mm thickness
Thermal expansion	0.0002/ºK (about 20 times that of steel)
Coefficient of friction	High but reduced on contact with oil or water
Flammability	Will continue to burn once ignited
Flexibility	Will undergo fatigue cracking under repeated flexing, particularly if the deformation passes through zero strain in the cycle - antioxidants can reduce this effect.

6.1 Shredded tyres

Discarded vehicle tyres are traditionally disposed of as waste materials, largely into landfills or tyre stockpile sites where they can pose potential safety and health problems as well as being unsightly. In recent years, however, interest has been shown in the use of scrap tyres as a feedstock for re-cycled or re-used materials. This has included highway engineering applications including pavements, soil reinforcement and drainage materials.

In Europe, tyres have been used to form soil reinforced slopes and retaining structures. Lareal et al (1993) describe the use of soil reinforcement with tyres behind retaining structures to reduce earth pressures onto the wall stem. Dalton and Hoban (1982) and Johnson (1985) describe remedial works on embankments on the M62 and A45 involving the construction of retaining walls at the embankment toes using scrap tyres tied together in layers.

The use of bitumen bound shredded tyres in highway drainage has been investigated by TRL and has included pilot and full scale road trials (Carswell and Jenkins, 1996). In North America, field and laboratory experiments on the use of shredded tyres as lightweight embankment fill materials (Newcomb and Drescher, 1994) and lightweight subgrade for pavements (Rudd and Loney, 1991) have been undertaken. The elasticity of rubber is implicit and therefore the use of shredded tyres to provide a compressible fully recoverable inclusion layer behind an integral bridge abutment might be possible.

The shredding of tyres is usually undertaken as a pre-requisite to re-cycling or re-use of scrap tyres. The particle size of shredded tyres from the shredding process is a function of the number of passes of the material through the shredding device with a typical particle size being produced of about 30mm. However, it is possible to produce a graded material. In this connection a graded tyre material has been produced that conforms closely to Type B material in Table 5/5 of the Specification for Highway

Works (Carswell and Jenkins, 1996).

Exploratory field and laboratory tests to determine the basic engineering properties of shredded tyres for use in light weight fill applications have been undertaken by a number of researchers. The results are summarised in Table 11 and are discussed below.

The bulk density of shredded tyres is a function of the particle size. In general, large-size particles yield a lower bulk density (230kg/m^3 uncompacted - 350kg/m^3 compacted) than smaller particles (500kg/m^3 uncompacted). Similarly porosity and void ratio are also a function of the particle size with coarser graded material having higher porosity and void ratio than finer graded material. In this connection Brassette (1984) indicated that the coefficient of permeability of unbound shredded tyres with a particle size of 50mm is about 3.5 x 10^{-2}m/s, which is better than that of a typical granular backfill. The high permeability of shredded tyres is advantageous in their use as both a compressible layer as well as a drainage layer behind an abutment.

Figure 12 shows the typical relation between vertical stress and vertical strain of shredded tyres loaded through multiple cycles. The tests were undertaken on material placed in a cylindrical steel container 970mm high by 740mm in diameter. The material easily deforms at very low levels of vertical stress and becomes significantly stiffer at about 100kN/m^2, which corresponds to about 25% strain. Upon unloading and reloading, the stress-strain relation follows a path parallel to the steeper portion of the initial loading path. This latter behaviour would seem to reflect more accurately the characteristics of shredded tyres in the field after compaction and placement behind a bridge abutment. Typically at this higher stiffness level the strain changes by about 10% over a vertical stress range of 50kN/m^2 to 380kN/m^2.

The relation between the horizontal and vertical stress for shredded tyres derived from the same test is shown in Figure 13. A bilinear relation appears to exist but this may be a function of the compaction of the sample and the number of loading cycles. Humphrey et al (1993) investigated the shear strength, compressibility and stiffness of shredded tyres with cycles in vertical load. On the basis of these tests, the values of the coefficient at rest K_o, Young's modulus and Poisson's ratio for the material corresponding to one and three cycles of loading and unloading are shown in Figure 14 and are summarised in

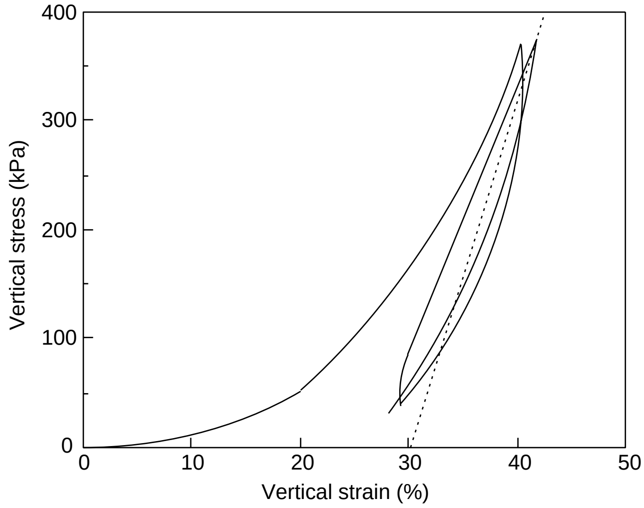

Figure 12 Vertical stress against vertical strain for shredded tyres (Newcomb and Drescher, 1994)

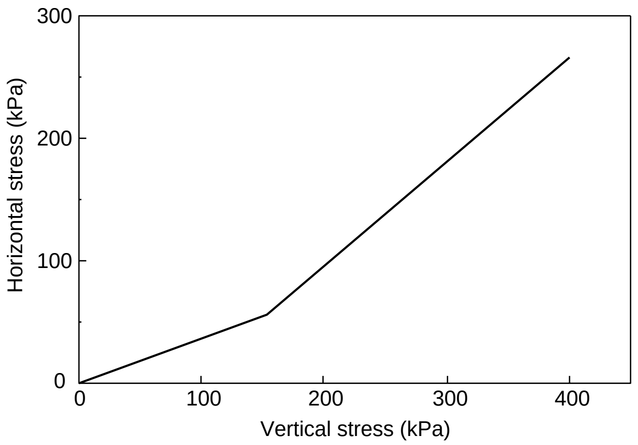

Figure 13 Horizontal stress against vertical stress for shredded tyres (Newcomb and Drescher, 1994)

Table 11 Typical engineering properties of shredded tyres

Ref.	*Mean particle size (mm)*	*Density (kg/m^3)*	*Porosity (%)*	*Void ratio*	*Young's modulus (MN/m^2)*	*Poisson's ratio ν*
1	No data	230-350	79	3.76	No data	No data
1	30	500	57	1.32	0.78	0.45
2	20-46	500-565	55-60	1.22-1.50	No data	No data
3	50	640-656	No data	No data	0.074 (0.22)	0.32
4	12-50	643	No data	No data	0.67	0.30

1 Newcomb and Drescher (1994).
2 Rudd and Loney (1991).
3 Humphrey and Manion (1992). Average initial tangent modulus and (secant modulus at 20% strain) given.
4 Ahmed (1993). Young's modulus estimated from compressibility test data and assuming μ = 0.3.

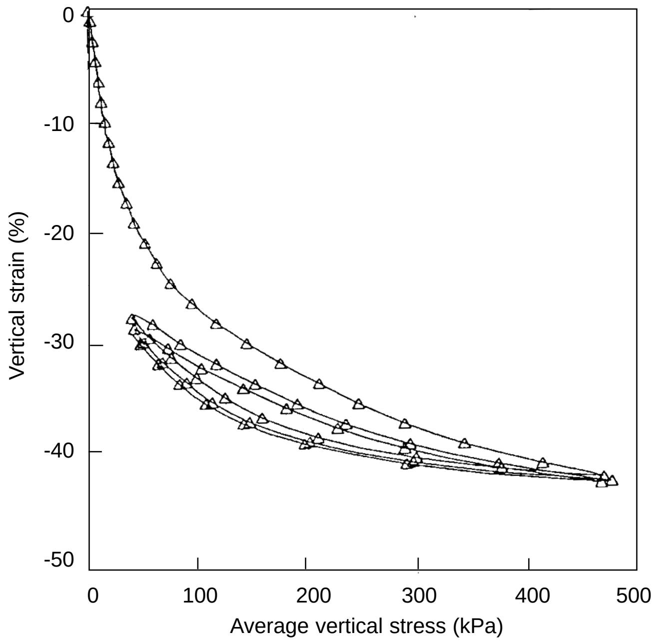

Figure 14 Deformation of shredded tyres under cyclic loading (Humphrey et al, 1993)

Table 12. The cohesion and angle of friction of shredded tyres was measured directly in a shear box apparatus (305mm box) and gave values ranging from 8kN/m^2 to 11kN/m^2 and 19^o to 25^o respectively. The apparent cohesion is thought to be due to the content of steel in the samples and was mobilised after large strain of the samples. This suggests that a low or zero cohesion intercept should be used for design because it appears that significant deformation is needed to develop cohesion. The relatively low angle of friction compared to conventional granular backfill is also advantageous as the magnitude of passive earth pressure will therefore be lower.

Table 12 Typical stress strain behaviour (Humphrey et al, 1993)

Stress-strain behaviour	*Coefficient of earth pressure at rest, K_o*	*Young's modulus, E (MN/m^2)*	*Poisson's ratio, ν*
Initial load cycle	0.26	0.17	0.25
Third load cycle	0.42	0.52	0.45

The values in Table 12 of stiffness and Poisson's ratio are based on the assumption that the material is isotropic. In practice shredded tyres are generally flat and will tend to arrange themselves mutually parallel when compacted or loaded. This creates a structure that is no longer isotropic but honeycombed with horizontally elongated cells. Accordingly the stiffness in the vertical direction may be higher than in the horizontal direction. In an integral bridge abutment situation, this is a favourable arrangement since vertical settlement of the compressible layer needs to be minimised in order to prevent cracking or movement at pavement level under highway loading. For granular soils K_o and υ range typically between 0.3 to 0.5 and 0.15 to 0.45 respectively. From Table 12 the corresponding values for shredded tyres are comparable suggesting that shredded rubber might behave similarly to granular soil if compacted to overcome the initial large degree of compression occurring during the first load cycle. The horizontal stiffness that can be achieved from placed and compacted shredded tyres is similar to the required value given in Table 1 for a compressible layer behind an integral bridge abutment.

Another advantage of tyre chips is their free draining characteristics and this may obviate the need for clean granular backfill or positive drainage provision behind the abutment wall. Humphrey et al (1997) have studied the effects on groundwater quality from run-off through embankments made from compacted shredded tyres. There was no evidence that shredded tyres increased the concentrations of any substances having an adverse effect on drinking water quality, although an increased concentration in iron was detected. This is to be expected since the shredded tyres also contained steel wire as a reinforcement material.

6.2 Rubber composite materials

In connection with studies into the re-use of waste tyres, a number of researchers have also investigated the use of rubber-asphalt and rubber-soil materials. Rubber-asphalts have been studied for their use as pavement or highway drainage materials as discussed in Section 6.1. In the USA rubber-asphalt materials have been studied for over 40 years for highway pavement and drainage applications. Generally asphalt paving products with crumb rubber (particle size range typically 1mm to 2mm) demonstrate better performance than asphalt alone. Reflective and thermal cracking is reduced whilst skid resistance and general wear is increased (Ahmed, 1993). Asphalts are susceptible to oxidation which will slowly cause their embrittlement and stiffening. The use of crumb rubber as an additive can overcome this. Many researchers have also investigated the use of asphalt cement modified with crumb rubber made from shredded tyres as a pavement material (Daly and Negulescu, 1997; Zaman et al, 1995).

There is little published information on the elastic stiffness and compressibility of rubber-asphalt products. This is because their stiffness is dependent on temperature and rate of shear of the material. The addition of rubber to asphalt or asphalt cement is reported to increase the shear viscosity, linear visco-elastic function, elasticity and creep resistance of the material (Zaman et al, 1995). Stress strain behaviour and total stiffness is best characterised in terms of the complex shear modulus G^*. Daly and Negulescu (1997) give typical values of G^* for rubber-asphalt cement blended materials ranging from about 400kN/m^2 to over 5500kN/m^2. For the integral bridge situation the rate of strain of the compressible layer will be relatively low such that visco-elastic effects might be small. On this assumption, and using basic elastic theory, a crude estimate of the elastic modulus is about 1MN/m^2 to 15MN/m^2 assuming a Poisson's ratio of about 0.3 for the material. Further research is required to investigate the stress-strain and creep behaviour under long-term cyclic loading to allow further evaluation of these materials for a compressible layer.

Rubber soils have been studied primarily for use as light weight fill material for highway embankments and as backfill behind bridge abutments. In France, whole tyres have been investigated in the use to reinforce soils and is patented under the name, pneusol (Audeoud et al, 1990). In a similar way the addition of shredded rubber tyres acts as soil reinforcement as well as reducing the bulk density of the rubber-soil mass. Ahmed (1993) found that rubber-sand with shredded tyre/soil ratios of 39% or less exhibited engineering properties that were suitable for use as embankment fill. The material was easy to compact with low compressibility and high strength properties as summarised in Table 13 (comparative strength data for tyre chips alone are also tabulated). The confining pressure used in the triaxial testing was found to be the most important factor affecting strength. The higher the confining pressure the higher was the strength. It was found that the rubber soil samples exhibited a strain hardening behaviour and continued to stiffen with increased axial strain. This behaviour is disadvantageous for a compressible layer, particularly if strain hardening occurs under cyclic loading since it will allow higher lateral pressures to be transmitted to the abutment.

Table 13 Typical engineering properties for rubber soils

Material	*Dry density (kg/m^3)*	*Cohesion, c', at 10% strain (kN/m^2)*	*Angle of friction, ϕ', at 10% strain (degrees)*	*Estimated Young's modulus based on μ = 0.3 (MN/m^2)*
100% tyre chips[1]	336-496	4.3-11.5	19-25	0.93
39% 25mm tyre chips and 61% sand[2]	1421	43.4	34.6	4.88
40% 25mm tyre chips and 60% clay[2]	1299	56.5	21.1	1.91

1 Humphrey and Sandford (1993).
2 Ahmed (1993).

The values of Young's modulus shown in Table 13 are estimated from the results of one dimensional compressibility tests undertaken by Humphrey and Sandford (1993) and Ahmed (1993) assuming a Poisson's ratio of about 0.3. Comparing the values of elastic stiffness to those given in Table 1 suggests that the rubber sand or rubber clay soil mixes could meet the requirements for a compressible layer although layer thicknesses would need to be in excess of 1m. It must be noted however that mixes consisting of rubber and clay were generally found difficult to place and compact.

Little information is available on the durability of rubber soils, although it is reasonable to assume that the durability of the material would depend on the properties of the components, in particular rubber. From the discussions given in Section 6.1 shredded tyres would appear to have chemical inertness and durability and it is reasonable to suppose that rubber-soils would have similar properties.

7 Summary

Table 14 summarises the strength and stiffness properties of the materials investigated in this report and compares the various values with the requirements (extracted from Table 1) for satisfactory performance of a compressible layer behind an integral bridge abutment. In general there are limited data available on the performance of the materials and products investigated particularly their behaviour under long-term cyclic loading and combined normal and shear forces. It must be emphasised that, in many cases, further engineering of the polymeric materials may be possible to achieve the required mechanical properties. The simplest parameter which can usually be engineered is the thickness of the compressible layer and both the horizontal stiffness and shear strength requirement are dependent on this.

With regard to stiffness, the maximum likely movement of an integral bridge abutment due to thermal effects on a 60m deck is expected to be between 8mm and 16mm, as discussed in Section 2.1. For this range of movement, the induced strains in compressible layers with thicknesses of 0.3m and 1m will range between 2.7% to 5.3% and 0.8% to 1.6% respectively. Over the higher strain range most of the materials studied show some degree of non-linear elastic behaviour, which is only acceptable provided that the layer shows no plastic deformation and no significant permanent set when the cyclic load reduces. Once again this emphasises the importance of engineering the thickness of the compressible layer to achieve optimum performance.

The findings for the different materials can be summarised as follows.

(a) Expanded polystyrene (EPS)

High density EPS (typically high duty to ultra high duty) would appear to be suitable in terms of horizontal and vertical stiffness and shear strength for use as a compressible layer of 1m thickness. Properties are also broadly compatible with those required for a 0.3m thick layer. However, to ensure the potential for creep of the material is negligible, the lateral strain should be limited to the order of 1% and this will be exceeded when using a layer of this thickness. Horizontal stiffness for lower density EPS (such as standard duty) is on the low side for this application and as it has a greater tendency to creep, the preferred choice would be the higher density material. Generally EPS appears to adequately satisfy the shear strength requirements.

(b) Elasticised expanded polystyrene

The low horizontal stiffness of elasticised EPS means that a layer of thickness 0.3m or less will be adequate to absorb any high lateral stresses: however some concerns may exist on the effect of cyclic loading on stiffness and creep of the material. Further testing is required to provide more detailed information on this and on the shear strength of the material.

Table 14 Comparison of material properties with the ideal requirements for a compressible layer

Material property	*Layer-thick ness(m)*	*Require -ment*	*Expanded poly-styrene*	*Extruded poly-styrene*	*Elastised poly-styrene*	*Poly-ethylene foam*	*Geo-composites*	*Rubbers*	*Shredded tyres*	*Rubber-asphalt cement*	*Rubber-soils*
Horizontal elastic stiffness,	0.3	0.9 - 1.9 [Strain 2.7-5.3%]	1 - 4[2]	3 - 10[2]	0.2 - 0.5	1 - 3[4]	0.3 - 5[5]	1 - 5[6]	0.1 -0.8[8]	1 - 15	1 - 5[9] (1 - 2)
E_h, MN/m^2	1.0	3.1 - 6.2 [Strain 0.8-1.6%]	2 - 10	13.5 - 37	0.2 - 0.5[3]	3 - 10[3,4]	0.3 - 5[3,5]	1 - 5[6]	0.1 - 0.8[8]	1 - 15	1 - 5[9] (1 - 2)
Vertical stiffness, MN/m^2	All	3 - 30	2 - 10	3.4 - 9.3	> E_h	5 - 10	> E_h	> E_h	0.8 - 7[8]	> E_h	> E_h
Minimum horizontal	0.3	130[1]	90 - 220	No data	No data	No data	No data	> 3000[7]	No data	No data	See table 13[10]
shear strength, kN/m^2	1.0	40	90 - 220	No data	No data	No data	No data	> 3000[7]	No data	No data	See table 13[10]
Minimum vertical shear strength, kN/m^2	All	16	90 - 220	No data	No data	No data	No data	No data	No data	No data	See table 13[10]

1 See Section 2.1, item 6, this value is unlikely to exceed the lateral stress of about 40kN/m^2 developed by the backfill.
2. Secant moduli as stress-strain behaviour is non-linear; this may result in irrecoverable deformation on the unloading cycle.
3 May be difficulties in manufacture at this thickness.
4 Approximate range of values at these strain levels.
5 Most geocomposites show linear elastic response to about 10% strain before yield or brittle failure, see Figure 8.
6 Uniaxial test results. If confined, stiffness may be much higher because of the high Poisson's ratio.
7 Calculated from maximum values of shear stiffness given by Eyre and Stevenson (1991).
8 Stiffness increases with the compactive effort as the void ratio decreases.
9 Rubber-sand mixes: data in parentheses are for rubber-clay mixes.
10 Shear strength $\gamma = c' + \sigma_v \tan\phi'$ is a function of vertical stress.

(c) Extruded polystyrene
The horizontal stiffness of extruded polystyrene is compatible with the requirements for a compressible layer which is no more than 0.3m thick: however, this does mean that the material would be in use at strains of about 5% which are near its peak compressive strength and exceed the limit of its elastic performance. Even if it could be conveniently produced, a 1m thick layer of extruded polystyrene will be excessively stiff and transfer too much load. There would seem to be no significant advantage in using extruded as opposed to expanded polystyrene.

(d) Polyethylene foam
On the basis of limited mechanical data, the material appears to be compatible in terms of horizontal and vertical stiffness with the requirements for a compressible layer. Currently the material is manufactured in thinner layers and multi-layers may need to be engineered for use in an integral bridge application. Further information is required on the shear strength of the material and the effect of cyclic loading over the appropriate strain range.

(e) Geocomposite materials
Most geocomposites appear to have an almost linear elastic response up to a strain of about 10% and a layer thickness of 0.3m would meet the loading requirements in an integral bridge situation (Table 14). However, geocomposite layers are generally thinner than this because of their use for drainage purposes. A reduced thickness means that under normal deformation behind an integral abutment the induced strains could exceed 50% and hence be well beyond the yield point and near to failure. This problem could probably be overcome by using multiple layers or specifically engineering a new geocomposite product with the strength and stiffness characteristics to meet the requirements. One advantage of engineering a new product would be that drainage capability could be incorporated into the geocomposite design. Information on shear strength of the material and the effect of cyclic loading is not currently available.

(f) Rubbers
The elastic stiffness and shear strength properties of both natural and artificial rubbers appear to satisfy the requirements for a compressible layer. However, because of the incompressible nature of the material (Poisson's ratio is approximately 0.5), a solid layer of rubber confined behind an abutment will have a much higher stiffness than indicated in Table 14. This problem may be overcome by the introduction of holes or slots to reduce the Poisson's ratio effect or alternatively the rubber can be used in particulate form. Low temperature stiffening is also

undesirable although the abutment wall will provide some insulation against freezing climatic conditions and the effects of freeze/thaw cycles. Some rubbers also need to be protected against exposure to petroleum products which can cause swelling and/or softening. The long term durability of rubber is however generally well established through its use for bridge bearings.

(g) Shredded tyres and rubber composite materials

In terms of horizontal stiffness, shredded tyres have a marginally lower value than that recommended for a compressible layer but this should not preclude their usage. The vertical stiffness is determined by the degree of compaction of the shredded tyre particles and this could possibly be achieved by thorough compaction. The angle of friction of the material is comparable to a loose granular soil although it can possess a relatively high cohesion due to the steel content.

Shredded tyres mixed with an asphalt or asphalt cement binder might be made to conform with the requirements for a compressible layer. Certainly the material properties, at the lower end of the stiffness range, are compatible with the requirements for a compressible layer. One advantage with these materials is that the binding agent can be chosen to modify the stiffness of the material to suit the exact characteristics. However, their performance under long-term cyclic loading is uncertain because of the visco-elastic nature of the product. Further work is required to investigate this aspect before its suitability as a compressible layer can be assessed further. Like geocomposite materials, however, the requirements for drainage behind the abutment might be incorporated into the design formulation of the material thus providing economic benefits.

Similarly rubber-soils also have elastic properties that could satisfy the requirements for a compressible layer. In practice, however, it will be difficult to place these materials in a vertical layer against the abutment during construction. Their use might be more appropriate in a trench situation when installing a stress absorbing layer as a remedial measure behind an existing bridge. Rubber-sand mixes will compact more easily than rubber-clay mixes. However, rubber-sand soils can strain, harden and increase in stiffness, whereas low permeability rubber-clay soils may undergo strain softening and reduction in stiffness. Generally research is needed on the behaviour of rubber-soils under long term cyclic loading conditions to allow further evaluation of these materials.

8 Conclusions

A nominal value of earth pressure is required behind an integral bridge abutment to provide restraint to in service longitudinal live loads. This restraint can be provided by allowing in the abutment design for active or at rest earth pressure (K_a or K_o) to develop in the backfill. Higher earth pressures up to the limiting value of passive earth pressure K_p can develop due to seasonal thermal movement of the bridge deck causing interactions between the abutment and backfill. Although it is possible to design for these high pressures to avoid possible structural distress, a more economical design can be achieved by installing a compressible “stress-absorbing” layer between the backfill and the abutment. The compressible layer can reduce these high pressures in two ways:

1 Deformation of a compressible layer will accommodate the effects of cyclic loading due to deck expansion/ contraction and prevent high earth pressures developing in the backfill.

2 The compressible layer can also act to reduce wall friction between the abutment and the backfill. This secondary effect will assist in reducing the magnitude of the earth pressure that can develop.

This report has reviewed the engineering properties of the large material/product range which can potentially meet the requirements for use as a compressible layer behind an integral bridge abutment. Economic production of the material/product to meet the required specification, however, may depend upon the size of the market. Because of the ease with which polymeric materials, geocomposites and rubber composites can be engineered it should be possible to produce a compressible layer that will satisfy the design requirements as set out in Table 1. For this reason it is recommended that a performance specification is developed outlining test methods for determining suitability for this application in order to encourage development of these innovative products.

9 Acknowledgements

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Abstract

Investigations have confirmed that most bridge deck expansion joints leak and contribute more than any other factor to deck and substructure corrosion from de-icing salts. For this reason joint-free integral bridges, with the abutments structurally connected to a continuous deck, are more durable and cheaper to maintain. However, thermal strains in an integral deck cause cyclic loading on the soil behind the abutments which may result in the development of passive soil pressures. One method of avoiding the development of high lateral pressures is to use a low stiffness but compressible elastic backfill as a stress absorbing layer behind the abutment. This report identifies various compressible materials, for example polymeric and geocomposite materials, which may be suitable for use as innovative structural backfill behind integral bridge abutments.

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