CONVERTING RAW MATERIALS INTO THE PRODUCTS – ROAD BASE MATERIAL STABILIZED WITH SLAG-BASED BINDERS

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Abstract

A procedure is defined for commercializing slags arising as by-products of steelmaking, and this is used to develop certain products. The outcome of the present work comprised three products or groups of products: 1) slag-based binding agents, 2) a road structure improved by means of stabilization with such binding agents, and 3) a procedure for designing stabilized structures. The commercialization procedure was drawn up by examining the technical properties of the initial materials, excluding environmental acceptability and the mechanisms of their chemical reactions.

The research proceeded in stages, of which the first was a reconsideration of the results presented in the author's licentiate thesis concerning factors affecting the stabilization of road construction aggregates with blast-furnace slag-based binding agents and their significance. This was followed by a series of experiments designed to test the validity of these results. At the third stage the selection of slag-based binding agents was extended to include LD steel slag products, while the final stage consisted of the implementation of a set of test structures and associated preliminary experiments and monitoring measurements.

The binding properties of three blast-furnace slag products and three LD steel slag products, used separately or in various combinations, were examined in the course of this work, taking cement as a reference material. Materials were then eliminated as the research proceeded, either on account of their poor binding properties or on economic grounds. The final construction experiments were performed with three combinations: cement-activated granulated blast-furnace slag, LD steel slag-activated granulated blast-furnace slag and a mixture of ground granulated blast-furnace slag and cement.

The actual novel product among the slag-based binding agents to be introduced here is LD steel slag-activated granulated blast-furnace slag, which was found to be similar in its technical properties to cement-activated granulated blast-furnace slag. Structural layers stabilized with these two binding agents showed very similar bearing capacities, although the former did not reach the same compression strengths at an age of 91 days. The reasons for these similarities lay in a larger amount of binding agent used when activated with LD steel slag and the greater thickness of the stabilized layer, factors which both tended to compensate for the poorer compression strength.

The new information gained on the properties of cement-activated granulated blast-furnace slag and the mixture of ground granulated blast-furnace slag and cement opens up fresh opportunities for selecting binding agents and designing road structures. Formulae were developed here for predicting the uniaxial compression strength at 91 days, used as a stabilization criterion, from the value for a sample taken at 28 days, a technique which will speed up the assessment of stabilization results, especially when using slowly reacting slag-based binding agents.

Keywords: blast-furnace slag, by-product, commercialization, LD steel slag

….Suomen maa on pohjan perillä ajettunut eikä se itseksensä tuo ilmi lihottavia eli woimallisia kasvuja; ihmisen toimi ja uutteruus on sen parantavia. Ja kyllä se paraneekin, jonka näemme ilahuttavalla mielellä niistä kokeista, joita ajattelewaiset maanmiehet jo tähänki asti ovat tehneet.

Masuunin kuona eli konto, jota wiimeisinä aikoina on ruwettu pellon hyväksi käyttämään sanotaan olevan hywin uljasta ainetta täksi tarpeeksi.

Kiintiän saven se hellittää, höllän, newaisen, sammalisen ja turpeisen maan se tekee **lujemmaksi** ja onnellisemmaksi. Paitsi tätä on se myös vahvasti lihottava; sillä siinä on aina osaksi kalkkia. Pieneksi kuin santa muokattuansa wiedään se kesäpellolle ja sekoitetaan tawallisesti.

Niin kuin mergeli ja kalkki häwittää se sammalen niituilta, jos sitä syksyllä ohuelta maalle kylwetään. Kuinka paljon sitä kullekin maalaille annetaan, on maan luonnosta arvattava. Sammaliseen ja turpeiseen newamaahan sitä tuskin saadaan liiaksi; parempi on toki panna sitä kunakin kesantowuonna wähemmän erältänsä, kuin yhtäpäätä peräti paljolta.

Tätä ainetta saadaan arwattavasti ainoastaan rautaruukien lähellä, mutta on niissä olewa oiwallinen aine maan parannukseksi….

- C.R.Lindberg (1861) Maalaadun parantamisesta. Toimellinen ja taitava Sonnan hankkia. I.W.Eilijan kirjapaino, Turku -

Acknowledgements

List of Symbols and Abbreviations

a	radius of loading plate, mm
AC	asphalt concrete
AC_th	thickness of AC, mm
AC_T	temperature of asphalt concrete, ^{o}C
BF	blast-furnace
BOF/BOS	basic oxygen furnace / basic oxygen steelmaking
C	CaO
CG	crushed gravel
CT	crushed till
CR	crushed rock
DS	desulphurization
D	maximum grain size, mm
d_r	deflection at distance r from the centre point of the plate, mm
d_0	deflection at the centre of the plate, mm
d_{40}	grain size with a pass rate of 40%, mm
d_{50}	grain size with a pass rate of 50%, mm
E_{STAB}	Modulus for the stabilized layer (=MN/m^2), obtained by back-calculation by fitting the calculated theoretical deflection profile as closely as possible to that measured with a falling weight deflectometer.
E_{TOT}	Modulus for the whole road structure (=MN/m^2), determined from the total deflection in the structure at the loading plate of the falling weight deflectometer.
$E_{substrate}$	substrate modulus at the centre of the plate, MPa (=MN/m^2)
E_r	surface modulus at distance r from the centre point of the loading plate, MPa
E_0	surface modulus at the centre of the loading plate, MPa
EAF	electric arc furnace
F	FeO
FWD	falling weight deflectometer
K_1	modulus
K_2	modulus exponent
k_dx	correction coefficient for bending d over the distance x

LCA	life cycle assessment
LCCA	life cycle cost analysis
LD	Linz-Donawitz (= BOF/BOS process)
MHC	Modified humus class. The fourth class in a traditional four-category NaOH test is divided into two parts, with brown liquid placed in class IV and distinctly black liquid in a new class V.
M_R	resilient modulus, MPa. The relation of the loading pulse in a dynamic triaxial test to the reversible part of the deflection parallel to the direction of loading.
OM	old road material, taken from an old structural layer of thickness approx. 200 mm
pH	acidity
r	geophone distance, mm
S	SiO_2
SMP	secondary steelmaking process
SSC	steel slag cement
UCS	unconfined compression strength
UCS_{28}	unconfined compression strength after 28 days of curing
UCS_{91}	unconfined compression strength after 91 days of curing
w_{opt}	optimum moisture content, %
α	hexagonal crystal system for compound Ca_2SiO_4
γ	orthorhombic crystal system for compound Ca_2SiO_4
γ_{dmax}	maximum dry density, kN/m^3
ε_1 or $\Delta\varepsilon$	reversible radial relative deformation
ε_3	reversible axial relative deformation
σ_0	contact pressure, MPa
σ_3	principal stress
μ	Poisson ratio
Δq	range of variation in deviatoric stress
θ	sum of principle stressess (= $3\sigma_3 + \Delta q$)
θ_0	reference stress

Binding agent abbreviations

GBS	**g**ranulated **b**last-furnace **s**lag
PGBS	**p**artially ground **g**ranulated **b**last-furnace **s**lag
GGBS	**g**round **g**ranulated **b**last-furnace **s**lag
LD1	crushed LD steel slag 0-3 mm
LD2	partially ground LD1
LD3	ground LD1
CEM	Cement (CEM I 52.5 or CEM II A 42.5 or CEM II A 42.5R depending on the phase of the research)

Binding agent formula abbreviations (examples):

GBS-LD1 (90/10)	binder containing 90 w-% GBS and 10 w-% LD1.
GGBS-CEM 3(70/30)	binder comprises 3 w-% of dry weight of the stabilized material and consists of 70 w-% GGBS and 30 w-% cement
GBS5/LD1 10	dry mixture to be stabilized contains 5 w-% GBS, 10 w-% LD1 and 85 w-% road material

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1 Introduction

The increasing attention paid to environmental protection in recent decades has meant that international organizations and communities have drawn up innumerable programmes and declarations setting out general principles for forms of industrial production that take environmental considerations into account (ICC 1991, IISI 1992). It has become customary in this context to speak of employing the "best available techniques" (BAT) in each branch of industry, and a BAT Reference Document has been drawn up for the steel industry, for example (European Union 2000). This is an industry in which, alongside the actual production of iron and steel, vast quantities of waste and by-products are generated, the most significant by-products in terms of volume, at least, being slags of various kinds. Each species of slag has its own inherent properties, which have to be determined in order to optimize their exploitation in both a technical and an economic sense.

Research into slags and their utilization has to a large extent been dependent on the type of slag concerned. As a general rule, it may be said that the later the stage in steel production at which a slag is generated, the more difficult it is to find an economically and technically rational means of utilizing it. It has proved possible to make fairly efficient use of blast-furnace slag, which has been employed in the manufacture of building materials and cement for some time, whereas research into steel slags, particularly those arising in the oxygen blow converter process, and their utilization is still feeling its way somewhat.

The following generalizations may be made regarding research into the various types of slag and their utilization: that the principal use for blast-furnace slag is in the cement industry, that for LD steel slag is in road construction, mostly as a pavement aggregate, although it is likely to be used in increasing amounts in the cement industry in future, that of electric arc steel slag as a pavement aggregate in road making, and that of other slags, such as desulphurization slag and secondary steel slag, mostly in massive embankment and landscaping work.

Refined slag products can be utilized in all forms of civil engineering and road building, for purposes ranging from subgrade strengthening to surfacing aggregates, the most rational use being deducible in each case from the particular properties of the slag as a raw material.

Strict requirements in terms of bearing capacity are laid down for materials to be used in civil engineering and road construction, as the structures concerned have to be able to

withstand the loads imposed on them for periods of several decades and retain their target service properties, often under highly variable temperature and humidity conditions. In addition, there are few opportunities to influence or control the conditions to which they are likely to be subjected. It is often difficult to obtain sufficiently good materials for such purposes, and the choice is increasingly often coming to be governed by environmental questions.

Slags are usually used in the massive structural layers of roads in an attempt to improve both the bearing capacity of the resulting road and its thermal insulation under conditions of seasonal ground frost. Considerable financial savings have been achieved with the use of blast-furnace slag for stabilizing the structural layers of roads by comparison with the traditional binding agents, cement and bitumen.

Research into the use of slag for road construction and civil engineering purposes has involved scarcely any actual large-scale programmes of more than a year's duration aimed at the development of new commercial products. In other words, the work has been mostly a matter of one steel mill defining the technical properties of one of its species of slag and comparing these with the properties of traditional road construction materials, i.e. natural stone products. From a research point of view, and with regard to raising the value added in the re-use of such materials, the best solution would be to favour the most highly developed uses, e.g. in the cement industry. The use of unprocessed slags directly for landfill purposes, sometimes in combination with other materials, does not usually represent a very well-developed way of taking advantage of their material properties, whereas their use as binding agents conforms to a higher degree of processing.

The present research was carried out as an integral part of the "Slags in geotechnology" project at the University of Oulu in the years 1995-2000. The target was to develop a procedure for commercializing slags from the steel industry as a binding agent for use in road construction. The work took the form of a case study of five test structures produced at a single experimental site.

2 Approach, objectives and hypothesis

The actual industrial processes involved in the production of iron and steel give rise to a considerable number of other materials of various kinds that were at one time simply referred to as waste. Nowadays, however, it is somewhat more difficult to find the correct term to cover these materials, as work has progressed on converting them into marketable products and they have become objects of separate business activities. Terms such as waste, product, by-product, co-product, recycled product and raw material are problematical to define, as different terms may be used to describe precisely the same material according to the situation. It is nevertheless important to be able to select the correct term in each instance, for many reasons. The greatest impact of the incorrect choice of term may be felt in cases where some law or regulation may prevent a material from being used at all if classified in that way. The choice of the appropriate term as far as the marketing of the resulting products is concerned can have a considerable effect on their image and the extent to which they are accepted by the general public.

The terms applied by the steel industry to its products and the materials generated alongside its actual production processes have become fairly firmly established over the years, and their use continues to be dictated largely by the viewpoint of the steel producer. Consequently, the terms used for the materials generated alongside the actual steelmaking processes do not in general promote their marketing in a situation in which attempts are being made to create a high-quality "by-product" image. A good example of this is the term "slag", which is still popularly equated with waste.

A product can be defined in a number of ways. It is a commodity that has been produced, in everyday speech usually by some form of agriculture or industry, and is intended to be sold as such. A commodity is understood in economics as something that satisfies human needs (Halinen *et al.* 1997b). Products may be defined in a broader sense, however, as combined effects of goods and services, so that they can even consist exclusively of services (Kärkkäinen *et al.* 1995, Lipponen 1993). A product is everything that the customer pays for (Kärkkäinen *et al.* 1995, Jaakkola & Tunkelo 1987). A product can be a single object or service which satisfies the needs of a certain customer or group of customers, or it may be a functional entity or system, and its value will be affected to an essential degree by auxiliary factors such as its image, guarantee, maintenance and other services (Jaakkola & Tunkelo 1987). When defining a product

one is usually thinking of an object placed on a shelf in a shop for sale rather than something that is manufactured *in situ* such as a road structure, and the same is true of product development, for such projects are usually thought of as involving prototypes and models. We could imagine, of course, that an experimental stretch of road might be a prototype of a kind. A further problem is that in the case of slag the raw material is supplied whether it is needed or not, and in some cases steps have to be taken to create the need. The marketing of slag products departs from the conventional pattern in this sense, as it is normal to perceive a need and then consider a suitable product to meet that need, although a greater similarity emerges when we are obliged to consider how a slag can be processed in order to meet a certain customer's requirements.

A by-product is the result of further processing of a material that arose as waste in the actual production process, or an extraneous part separated off from the principal product (Halinen *et al.* 1997a). It may also be defined as "a material obtained during the production of another, which is considered to be the main product" (Kamon 1998, OECD 1997, Mroueh *et al.* 2000), or as "an additional result, sometimes unexpected or unintended" (Kamon 1998). The important thing is that it is not an end product, and therefore it can be considered an industrial product when it is used in our lives or waste when it is not used. The waste disposal legislation does not recognize the concept of by-product, but defines all such materials as waste.

Increasing emphasis has been placed in recent years on the structural behaviour of roads and the definition of objectives that apply to the whole service life of a road. In other words, attention has shifted from the choice of materials to the choice of solutions and the advantages attached to these. This development has made it possible to put forward new solutions, frequently ones based on the use of industrial by-products, for consideration alongside the "traditional" construction materials, often giving rise to fairly competitive alternatives.

The aim of this research was to develop products under the following boundary conditions:

1. The eventual outcome should be a number of products, depending on how a final product is defined. These could be 1) saleable *binding agents*, 2) an improved *road structure* stabilized by means of a binding agent, or 3) a *dimensioning procedure* for structures to be stabilized.

2. The commercialization process should be specified by examining the technical properties of the starting material independently of environmental considerations. The main reason for this is that the procedures for assessing environmental suitability are not sufficiently clearly defined and generally accepted as to be examined reliably in the present work with a view to future utilization.

3. Starting materials of an indeterminate character, i.e. with no clear definition as to whether the material to be commercialized is initially a form of waste, a by-product, a raw material or some other material.

4. No direct numerical evaluation of economic aspects was to be attempted, although some materials were rejected on economic grounds.

5. The analysis of the results should not go into reaction mechanisms but be confined to technical aspects.

It may be noted from the above five points that the research is angled towards the technical aspects of the commercialization of materials. The experimental part of the

research took the form of a case study. The overall commercialization procedure is presented in the diagram in Figure 1.

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┌─────────────────────────────────────────────┐
│        ┌──────────────────────────────┐      │
│        │   BY-PRODUCT - RAW MATERIAL   │      │
│        └──────────────┬───────────────┘      │
│                       ▼                       │
│   1.   ┌──────────────────────────────┐      │
│        │   PROPERTIES OF THE MATERIAL  │      │
│        └──────────────┬───────────────┘      │
│                       ▼                       │
│   2.   ┌──────────────────────────────┐      │
│        │      BINDING PROPERTIES       │      │
│        └──────────────┬───────────────┘      │
│                       ▼                       │
│   3.   ┌──────────────────────────────┐      │
│        │   BINDING AGENT - AGGREGATE   │      │
│        │         INTERACTION           │      │
│        └──────────────┬───────────────┘      │
│                       ▼                       │
│   4.   ┌──────────────────────────────┐      │
│        │   CONSTRUCTION EXPERIMENT     │      │
│        └──────────────┬───────────────┘      │
│                       ▼                       │
│        ┌──────────────────────────────┐      │
│        │           PRODUCTS            │      │
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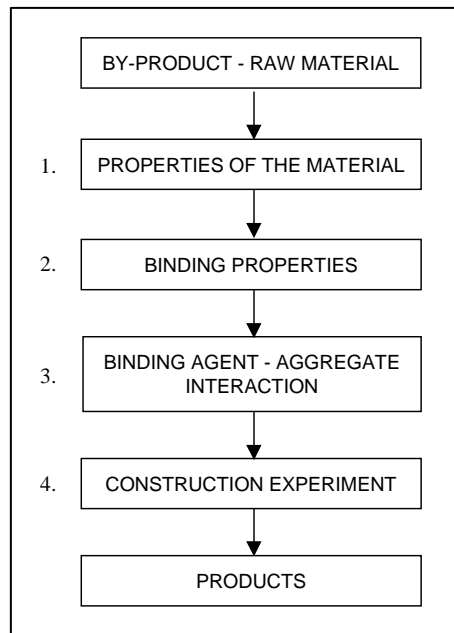
Fig. 1. The commercialization procedure followed in this research.

It was also important that the research should be carried out with a view to practical application of its results, i.e. attempts were to be made to simplify the methods. This was done in practice by restricting the laboratory work to mainly uniaxial compression tests, the results of which provided five experimental road structures for the analysis of bearing capacity behaviour. The research was carried out in phases, starting out from the results achieved by the author in his licentiate thesis.

3 Commercialization of products in road construction

Certain difficulties are encountered in defining the terminology connected with the commercialization of products, and especially with the commercialization of materials arising as by-products of industrial processes. There may be more than one term in existence for the same thing, for instance, in addition to which the relevant legislation actually recognizes no other terms than waste, so that there is a good deal of room for interpretation in the definitions of the materials concerned.

If we set out to define materials arising alongside an actual industrial production process in accordance with the existing terminology and legislation, we arrive at a scheme like that depicted in Figure 2. These materials can be referred to as by-products, co-products, recycled products or waste. If the authorities order a material to be removed from the process, this becomes waste, but if there is no obligation to remove it, we then have to ask ourselves whether there are any chances of exploiting it profitably. If there are no chances of doing so, or if it is not exploited for some reason, it must also be regarded as waste. If it is decided to make use of it, we must then assess whether it is used as it is, without further processing, or whether it must be processed in some way at first. If no processing is required, the material may be regarded directly as a by-product, while in other cases it becomes a by-product only after the necessary processing. If the material proves unsuitable for use even after processing, it is still to be considered waste.

The criterion in the above definitions and in the scheme set out in Figure 2 is the general one of environmental acceptability, although it is always possible, at least in theory, for a material arising in this way to be environmentally entirely acceptable but incapable of being employed technically for any useful purpose. This would mean that an environmentally acceptable material was nevertheless classified as waste. It should also be noted that a by-product at the "raw material" stage does not necessarily require any processing to become a "product", but that "product development" is required. GBS, for example, is already processed to a form in which it can be used when it is at the "raw material" stage. The boundary conditions to be placed on the product, e.g. with regard to dimensioning, nevertheless have to be defined by means of "product development".

When one is setting out to commercialize slags arising in steel production processes as road construction materials the starting point is slightly different from that set out above, although a problem still arises on account of the fact that the waste disposal legislation

does not recognize industrial by-products. As Vaittinen (2000) puts it, slag producers have tried during the last decades to raise the image of slag from that of a form of waste to that of a by-product. From a practical point of view, i.e. the marketing of slag products, it is vital that slag should not be classified as waste.
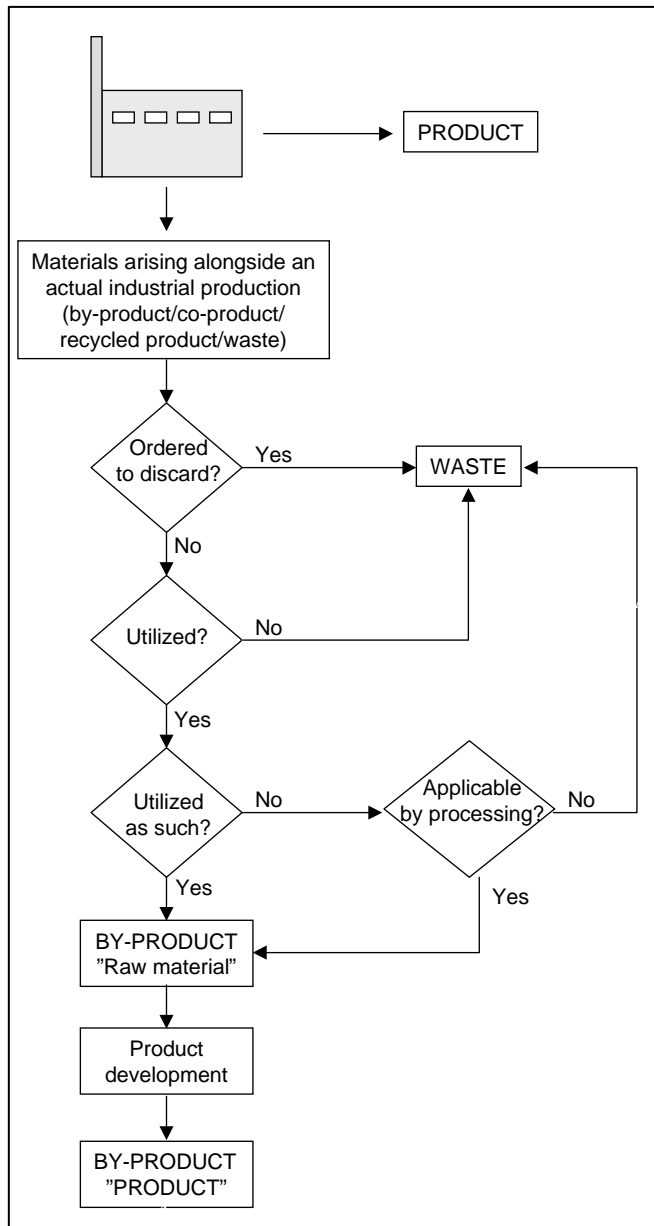
Fig. 2. Definitions of materials arising alongside industrial production processes in the light of the waste disposal legislation.

When a by-product (slag) is used for road construction purposes, it should be considered who is the user and who is the beneficiary. A company engaged in developing and marketing industrial by-products could be classified as the beneficiary, in which case the user must be the construction company that includes the slag products in its road structure. If we pursue the user-beneficiary chain somewhat further, however, we may say that the eventual user is the motorist who drives along the completed road, in which case the construction company becomes the beneficiary.

A further problem that has emerged for the commercialization of industrial by-products is that no clearly defined procedure exists for this commercialization, with the consequence that it is very difficult to define when a material subject to product development can begin to be called a product. The problem may lie in the fact that there is no clear official supervision or control system that declares at what point the term *product* can be deemed to apply. This may make it difficult from the authorities' point of view to launch an accepted new product on the market, especially if it is the result of recycling industrial by-products, possibly in spite of the fact that the recycled product may already be in extensive use. In the end, the commercialization of waste or by-products arising alongside an actual production process may, for the sake of simplification, be divided into two closely linked processes: a process connected with chemical properties and a process connected with technical (physical) properties, Figure 3.
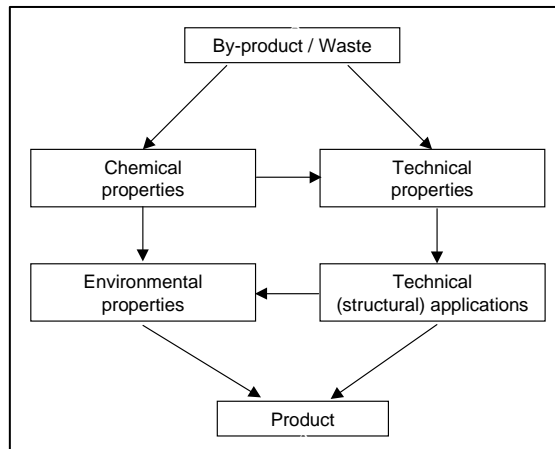
Fig. 3. Product development processes associated with materials arising alongside actual industrial production.

The commercialization processes presented in Figure 3 imply in the case of steel slags that the chemical properties of these materials, chiefly the bonding reactions associated with slags in general and the volume instability associated specifically with steel slags, will have a considerable influence on their physical properties. It is by virtue of these binding reactions that a structure built up of slag can be transformed within a few months from an unbound one to a semi-rigid or even rigid one. Where slags are concerned, there is little that one can do to regulate the chemical properties that influence this behaviour.

One of the basic requirements for the initiation of a new product development project is that there should exist a need for the product and a vision as to how it may be implemented. A need alone is not enough (Jokinen 1999). The technique of stabilization with blast-furnace slag set out from a need experienced by road constructors for alternative technologies and materials, chiefly on account of the poor quality of the natural materials available. There were thus good prospects for developing a useful product, since the product in itself had already appeared. The project could then be focused on elaborating the product further in order to conform better to customers' requirements and to widen the range of choices based on this technology that are open to customers, e.g. by extending the range of binding agents available, as in this work.

Problems may arise in the development of industrial by-products for use in road construction on account of various practical obstacles, which may be of a technical, administrative, juridical or attitudinal nature, see Table 1.

Table 1. Typical practical obstacles to the use of recycled products, from the viewpoints of the various parties involved, with examples (Mäkelä 1997).

Viewpoint	Technical obstacle	Administrative obstacle	Juridical obstacle
Manufacturer or supplier of recycled product	Has not defined all its physical properties, environmental impacts, quality fluctuations, or risks. Deficient instructions for use.	Has not gained official approval for construction materials in time.	Does not accept full responsibility for the product.
Instance commissioning, owning or using the structure	Looks on it as a risk, prefers old, well-tried solutions such as the use of gravel, requires more of the recycled product than is necessary for its purpose, shifts responsibility for demonstrating the length of its service life onto the supplier.	Does not take responsibility for anything, major projects are too large and important for making individualized decisions and small ones too insignificant and develop too rapidly - best results to be obtained with routine solutions.	Does not take responsibility for the product acquired, places conditions on it.
Designer	Unable to design, unfamiliar with the material, design requires more work and learning effort with a recycled material, design work cheap and effortless with old, familiar materials.	Unfamiliar with permit procedures, so that bureaucratic obstacles seem insurmountable.	Avoids risks and will not suggest using alternative recycled products.
Construction company	Unfamiliar with the material, does not have the proper equipment, or the equipment cannot easily be adapted, unable, wholly or in part, to rectify environmental damage occurring during construction work, dubious work safety measures.	Demands additional payment for use of an unfamiliar construction material, especially if the recycled material is an alternative.	Unwilling to take responsibility for use and handling of an unfamiliar material, lays down additional conditions and demands exemptions from warranty periods.

Table 1. (Continuing) Typical practical obstacles to the use of recycled products, from

the viewpoints of the various parties involved, with examples (Mäkelä 1997).

Viewpoint	Technical obstacle	Administrative obstacle	Juridical obstacle
Supervising authorities	Try to obstruct use of the material because no experience exists regarding its service life, i.e. no experience of its use over periods of 20-50 years, regard environmental assessments as inadequate, unable to decide anything without a statement and formal approval from the State Technical Research Centre (VTT)	Authorities at different levels disagree, information on ministerial aims not realized at the practical level, all action subject to permission, responsibility for demonstrating viability of the techniques placed on the constructor, parallel authorities make conflicting decisions, differences between local government districts, lack of expertise in civil engineering, failure to recognise the importance of given factors or to appreciate the whole situation, environmental authorities still searching for the limits of their powers.	Attitude that if you do not do anything you cannot be blamed for anything, reliance on laws that can be interpreted in different ways.
Owners of related equipment and structures, neighbours	Suspect the material of causing corrosion, preventing servicing of equipment or digging up in spite of the precautions taken	Make own work easier by refusing to do anything that has not been tried before and found suitable, reluctant to take responsibility in matters not related to their own interests such as protection of the environment or promotion of sustainable development in civil engineering.	Exercise the right to refuse to use unfamiliar materials on the grounds that these are aggressive or incompatible with existing ones.

4 Steel industry slags and their use in civil engineering and road construction

4.1 Slag generation and slag properties

The process of steel production can be considered to consist of three stages (Figure 4): reduction stage, steelmaking stage and secondary metallurgy stage.
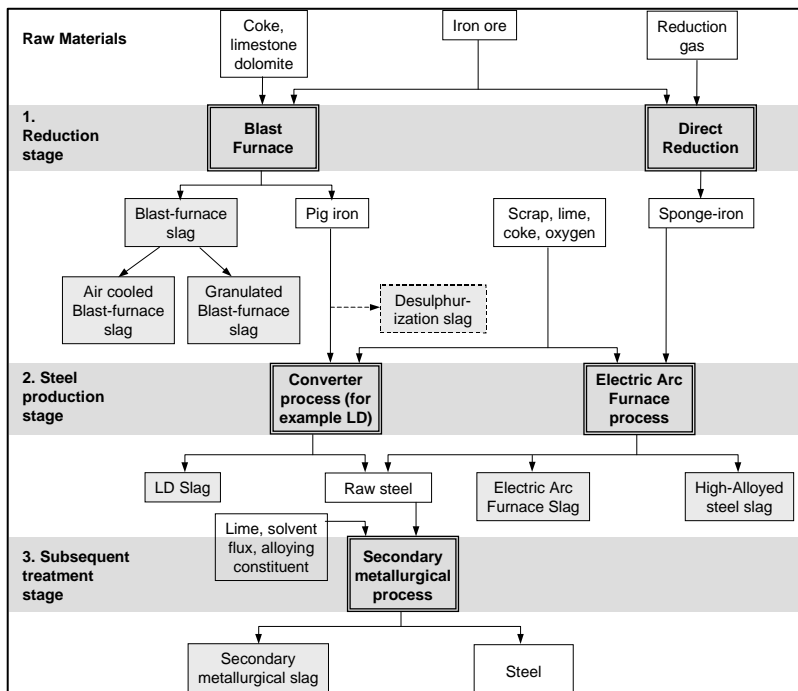
Fig. 4. Description of the processes involved in the production of steel and slags (Balcázar 1998, Geiseler 1998, 2000). The desulphurization slag aspect was added by the author.

The stages presented in Figure 4 above yield:

− blast-furnace slag
− basic oxygen steel (BOS) slag and electric arc furnace (EAF) slag
− secondary metallurgy slag

In addition to these, the integrated blast-furnace - BOF route can involve a separate desulphurization stage, generating desulphurization slag.

The liquid slag may undergo different kinds of heat treatment, so that blast-furnace slags can lead to a variety of products, as listed in Table 2.

Table 2. Different kinds of slags from iron- and steelmaking (Geiseler 1998).

lump slag	by slow cooling with air to form crystalline material.
foamed or expanded slag	by moderate cooling with a little water which leads to a partly vitreous and partly crystalline structure.
slag wool	which is generated by using air or steam to form a vitreous, woolly solidified product.
granulated slag	which is generated by rapid cooling by employing copious amounts of water in complex granulation facilities. The slag solidifies as a vitreous product which has latent hydraulic properties.
pelletised slag	by rapid or moderate cooling. The coarser pellets have properties like foamed slag and the finer pellets properties like granulated slag.

The usual chemical compositions of slags from the blast-furnace (BF), basic oxygen furnace (BOF), electric arc furnace (EAF) and secondary metallurgy processes (SMP) are shown in Table 3 (Geiseler 1998, 1999, 2000), together with a typical analysis of desulphurization slag (DS) from the Raahe Steel Works of the Rautaruukki Company in Finland. These slags always result from lime silicatic melts (Geiseler 1998). Blast-furnace slags additionally contain about 10% MgO and Al_2O_3, while their iron content is usually below 0.5% since they result from a reduction process. By contrast, BOF and EAF slags are generated in oxidizing processes, so that their Fe_{tot} is significantly higher (Geiseler 1998, 2000).

One significant factor as far as the utilization of slags is concerned is that scarcely any attempt is made to alter the chemical properties of the slag used as a raw material when considering its future use. Where the chemical properties are altered, this sets out primarily from the needs of the iron and steel processes themselves.

Table 2 above shows the various types of slag that arise as a result of differences in the means of cooling. The most common means employed with blast-furnace slag are air cooling and water cooling, i.e. granulation. In the air cooling process the molten slag is run off into a bed, from which it is later removed with an excavator and processed into products. Water cooling of blast-furnace slag is increasing in frequency all the time, largely in response to the needs of the cement industry.

Table 3. Usual chemical composition for steel industry slags in wt.-%.

	BF-slag[1]	BOF-slag[1]	EAF-slag[1]	SMP-slag[1]	DS-slag[2]
CaO	36-43	48-54	25-35	30-52	19-36
CaO$_{free}$	-	1-10	0-4	1-10	-
SiO$_2$	35-39	11-18	8-18	8-23	10-30
Al$_2$O$_3$	8-12	1-4	3-10	3-20	2.9-4.8
MgO	4-12	1-4	3-9	6-12	1.1-3.2
Fe$_{tot}$	< 0.5	14-19	20-30	5-12	17.9-39.3
Mn$_{tot}$	< 0.5	1-4	2-5	0.5-3	0.2-1.0
Na$_2$O	< 0.5	-	-	-	1.0-1.7
K$_2$O	< 0.7	-	-	-	0.05-0.70
S	1.2-1.6	-	-	-	1.5-2.1
CaO / SiO$_2$	1.0-1.2	2.8-4.4	1.7-4.0	1.4-5.5	0.6-2.5

[1] Geiseler 1998, 1999, 2000 [2] three analysis of Rautaruukki Raahe steel works slag (Hiltunen 1996)

Granulated blast-furnace slag, created by rapid chilling of molten slag using a large volume of water, consists of rough-textured grains of the size of coarse sand which are composed of 93-99% glass. This glass is inherently unstable and will form into stable crystals of calcium-alumina-silica hydrate in the presence of water. This reaction is extremely slow at normal temperatures, but when the granulated slag is finely ground and an activator or catalyst is present which will form an alkaline solution, the reaction is much more rapid and stable crystalline compounds very similar to the hydrate of Portland cement occur. In Portland-slag cement blends, lime released by the Portland cement hydration reaction serves as the catalyst for the blast-furnace slag cement reaction. (Heaton & Cao 1998)

According to Hiltunen and Putro (1993), silicic anhydride in a molten state occurs in a loose tetrahedral lattice, forming a triangular pattern with the Si atom surrounded by the oxygen atoms when viewed in two dimensions (Fig. 5). If the slag is allowed to crystallize, it will form a regular lattice with four oxygen atoms grouped around a Si atom, one of them always belonging simultaneously to two tetrahedra. If, on the other hand, the blast-furnace slag is cooled rapidly to form a vitreous material, the crystalline structure of the lattice remains more irregular, so that the calcium ions occupy the empty spaces in the CaO-SiO$_2$ system. This means that the oxygen introduced by the calcium oxide can interrupt the oxygen bridge between two tetrahedra and take the place of the ion that has been released. MgO behaves in a same manner to CaO, but an aluminium ion can act like either a calcium or a silicon ion. Briefly, this means that the more vitreous the material is and the more basic components it has, the more fractured its internal structure and the more reactive the resulting slag proves to be. This fact is decisive as far as the hardening of water-cooled slag is concerned. (Hiltunen & Putro 1993)

In order to be able to harden hydraulically, the glass particles must first dissolve, which takes place by calcium ions separating out rapidly from the edges of the particles until a water-resistant surface layer has formed. In order to prevent this, an activator is required (Hiltunen & Putro 1993, Murphy *et al.* 1995, Roy 1989). One such activator which has been well known in the cement industry for a long time is the calcium hydroxide released by Portland cement. Pure granulated blast-furnace slag can also

harden without an added activator, one possible explanation for this being the sulphur contained in it, which is predominantly in the form of calcium sulphide, for when this reacts with water it likewise produces calcium hydroxide. Reactivity during the production of granulated blast-furnace slag can be affected not only by the composition of the slag itself but also by the temperature of the slag to be granulated. Generally speaking, the higher the temperature, the greater the reactivity that is achieved. After granulation, the reactivity can be adjusted by adding fine material and/or activators, and hardening reactions can also be enhanced by a rise in ambient temperature (Hiltunen & Putro 1993).
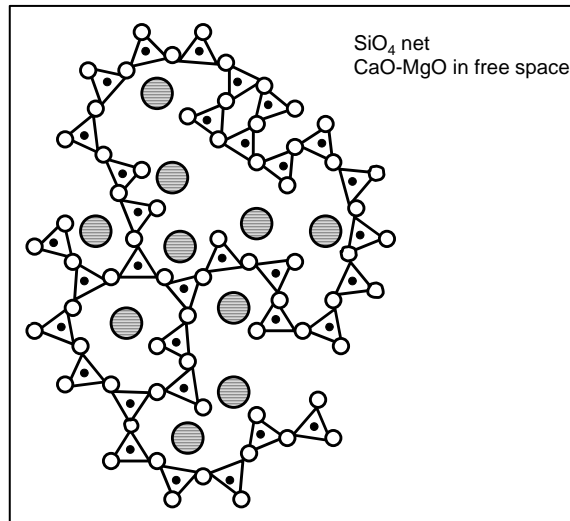
Fig. 5. Two-dimensional silicate lattice structure of granulated blast-furnace slag, with a Si atom surrounded by oxygen atoms and CaO or MgO in the free spaces (Hiltunen & Putro 1993).

Steel slags are normally divided into acid and basic types, the best-known example of the latter being Thomas slag, a variety no longer produced nowadays, as it is impossible to obtain phosphorus-containing raw materials economically.

One of the principal problems associated with the utilization of steel slags is expansion, or volume instability, a phenomenon which has been subjected to a great deal of research. Efforts have been made to determine its causes and how it can be minimized or controlled so that the material can be rendered as safe as possible to use.

Volume instability in steel slags arises from three mechanisms (Geiseler 1998, Viklund-White & Ye 1999):

− Hydration of free lime: $\quad CaO_{free} + H_2O \implies Ca(OH)_2$ (1)
− Hydration of free MgO: $\quad MgO_{free} + H_2O \implies Mg(OH)_2$ (2)
− Disintegration due to a $\alpha \rightarrow \gamma$ C_2S transformation

Free lime originates either from residual undissolved lime or from the dissociation of tricalcium silicate according to:

$3CaO \cdot SiO_2 \implies 2CaO \cdot SiO_2 + CaO_{free}$ (3)

The residual lime is normally dominant, and the total amount of free lime in BOF slag

can reach 20 wt-%. Free magnesium is either derived from undissolved slag former or is a result of MgO saturation in the slag. Free lime and free magnesia are hydrated in contact with air to form calcium and magnesium hydroxides according to formulae (1) and (2). This hydration results in a large expansion in the volume of the slag, with subsequent disintegration. Since the hydration of free magnesia is a much slower process than that of free lime, the presence of the former is a more serious problem, since it is impossible to predict when and to what extent the reaction will occur. (Viklund-White & Ye 1999)

Free lime and free MgO occur in slags both as precipitates from the melt and as unassimilated particles. The precipitated particles have no significant effect on volume stability, whereas the unassimilated particles are a major factor (Alexandre *et al.* 1993, Geiseler 1996). The species of free calcium and their structural forms are presented in Figure 6 (Geiseler *et al.* 1987). In the case of free lime, the important factor is the "spongy free lime", while in MgO-rich BOF slags it is the magnesio-wüstite, with MgO concentrations above 70% (Alexandre *et al.* 1993, Geiseler 1996).
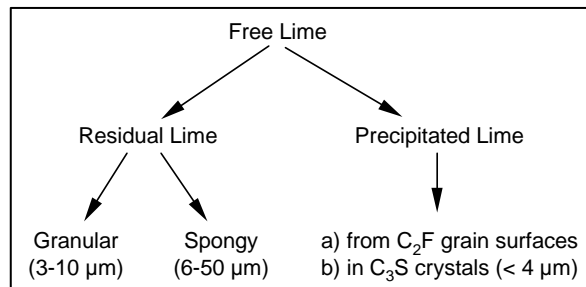
Fig. 6. Species of free calcium occurring in steel slag and their structural forms (Geiseler et al. 1987).

Efforts have also been made to determine the expansion of steel-making slag based on the effects of the periclase and lime phase and the textural form and distribution of the lime phase on stability, particularly the role of macroinclusions (Goldring & Juckes 1997). Also the effect of the free lime content and porosity of BOF slags on volume stability has been studied (Wachsmuth *et al.* 1981).

Efforts have been made to minimize volume instability in steel slags by various forms of processing, the simplest of which is weathering, in which the slag is stored in its eventual fraction for at least 6 months. The idea behind this is that any expansion that is going to take place will occur while it is in store, so that no instability damage will be experienced in the completed structure. If a steel slag is crushed after weathering, however, it will behave like unweathered material and will require a further period of weathering. The problem with this method is the time and storage space required, and criticism has also been expressed regarding its efficiency. In her examination of the mineralogy and chemical composition of LD steel slag from the Rautaruukki Company's Raahe Steel Works, Hiltunen (1996) noted that the free lime content of individual samples could be surprisingly high even after 1.5 years of weathering.

Water treatment in connection with cooling has also been shown to affect volume

instability (Hiltunen & Hiltunen 1996). There may be also differencies in the mean residual free lime level depending on the grain size of the slag (Thomas 1983).

Although weathering in storage heaps is by far the most common means of reducing the volume instability of steel slags, many other methods have been tried. The main problem with them, however, has been the high investment and handling costs. A method has been reported by Drissen & Kühn (1998, 1999) for improving the quality of steel slag products by adding sand to the liquid slag, while Featherstone & Holliday (1998) reported a treatment involving dry granulation, based on the atomization of molten slag using a cup or dish rotating at a variable speed. Ono *et al.* (1983) presented a blast granulation system for BOF slag in which the material was cooled rapidly in a stream of air, and Morishita *et al.* (1997) reported an open-yard steam ageing process for the weathering of steel slag which is in common use in Japan, in which slag expansion takes two days. In a new ageing process using high-pressure saturated steam, the slag is weathered in two hours. This means that the process can treat 12 000 tons of slag per month.

Literature searches suggest that three general types of test have been developed for estimating the volume instability of steel slag: chemical tests to establish the level of free CaO, and sometimes MgO as well and expansion tests to establish the degree of expansion. Expansion tests are often performed at an elevated temperature to reduce the time required to obtain results, and sometimes cycles of heating and cooling are included to add impetus to particle breakdown. There are also tests associated with the breakdown of individual particles, again sometimes performed at elevated temperatures (Heaton & Cao 1998). Juckes (1990) failed to see any significant differences on a microscopic scale between the reactions of slag components in the accelerated expansion test, a road panel or a weathering heap, which inspires confidence in the view that the accelerated test provides a good representation of behaviour in the long term under natural conditions, although the link between actual expansion percentages and the suitability of the slag for use as an aggregate has still to be defined.

Practically no research has been carried out into the use of desulphurization slag (mixer slag) and secondary steel slag (ladle treatment slag) for road construction purposes, apparently because of the relatively small amounts of these materials available and their inhomogeneous quality, especially in the case of secondary steel slag.

4.2 Utilization of slags

Hiltunen (1998) reports that blast-furnace slag is almost 100% reused in most countries and steel slag 75-80% reused. The principal uses for the latter are in civil engineering, road construction and the cement industry (Figure 7).

The most versatile in terms of its potential uses is blast-furnace slag, by virtue of its good technical properties and particularly its usefulness as a binding agent. The principal uses of blast-furnace slag are shown in Figure 8 (Hiltunen 1998).

The recycling of slags into the steelmaking process varies greatly from one mill to another. The main problem lies in the accumulation of substances that constitute

impurities as far as steelmaking is concerned. This is especially true of the recycling of BOF slag (Geiseler 1998, Drissen & Kühn 1999).
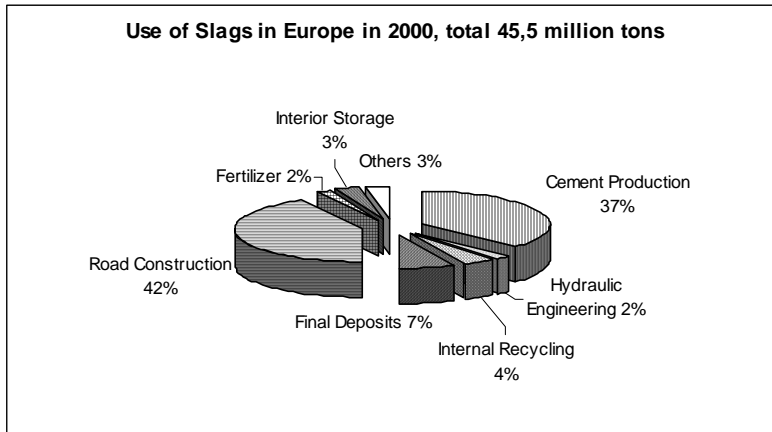
Fig. 7. Uses of slag products in Europe (Euroslag 2000).

Fig. 8. Principal uses of blast-furnace slag (Hiltunen 1998).

Slag has been used as both a binding agent and a basic constituent of concrete in the cement and building materials industry (Akinmusuru 1991), and also in breeze blocks, paving slabs and other bound structures, while Geiseler (1998) reports that slag wool and foamed slag were used in the past as insulating materials. Environmental considerations have been among the factors favouring the use of slags in the cement industry (Dikeou 1980, Lewis 1982a).

The basic model as far as use as a binding agent is concerned may be taken to be slag cement, which contains both blast-furnace slag and Portland clinker, in proportions that vary considerably, so that true slag cements may have as much as 90% slag in them. This allows advantage to be taken of the slower binding of slag relative to Portland cement alone and its lower reaction temperature, and is of particular benefit in massive structures. In addition, the resistance of the cement to chlorides can be improved by increasing the slag content.

A great deal of research has also been carried out into alkali-activated cement, but few attempts have been made to apply the results in practice. The reason for this may lie in the "complexity" of binder mixtures, which detracts from their practical usefulness. Alkali activation experiments have also been carried out with waterglass, Na_2SO_4 (Leivo 1995, Douglas & Brandstetr 1990) and with NaOH and Na_2CO_3 (Collins & Sanjayan 1998).

The cement industry makes use of blast-furnace slag as a component of cement mixtures, and some research has been done into the use of steel slag for this purpose, so that it may well prove to have a considerable potential in the cement sector. As Montgomery & Wang (1991) observe, it is well known that steel slag has a similar mineral composition to OPC clinker, and the favourable properties attributed to steel slag cement (SSC) in the literature include good abrasion resistance, high bending strength, high late strength, low heat of hydration, good impermeability, microexpansion and good frost and corrosion resistance (Sun *et al.* 1993, Guilin & Shushan 1999). According to Verma (1995), steel slag cement has the following advantages over OPC for the production of BF slag cement: less consumption of Portland cement clinker, lower investment costs, lower energy consumption and lower operating costs. Its low early strength limits its uses in some of applications, however.

There is a long tradition of using slags for agricultural purposes, and the form most commonly used for liming is steel slag, although blast-furnace slag is also used to a minor extent (Geiseler 1998, Geiseler & Hammer 1995, Pasanen 1997, Featherstone & Holliday 1998, Geiseler & Kühn 1999, Alexandre *et al.* 1993).

Slags have been used in a number of other applications, mostly for research purposes, e.g. as a blast-cleaning abrasive and as a raw material for the production of glass (Geiseler 1998).

4.3 Use of slag products in road structures

The possibilities for using slag products in different parts of a road structure are summarized in Figure 9 and Table 4.
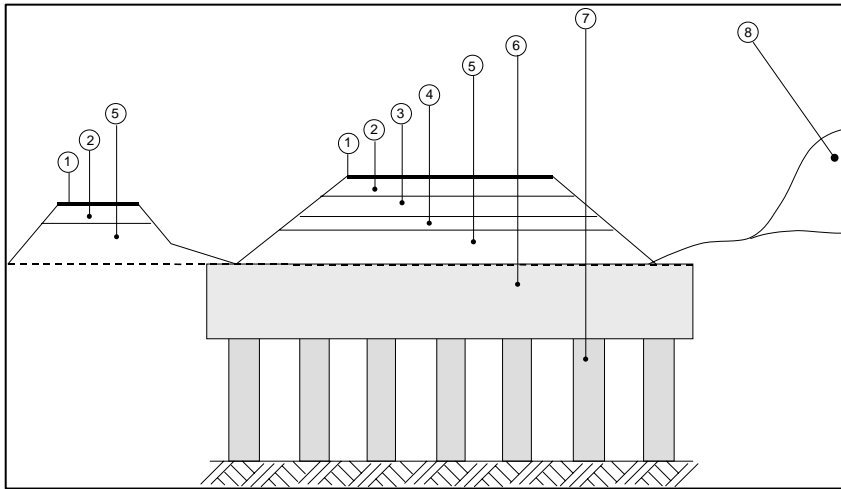
Fig. 9. Points at which slag products may be used in a road structure.

Table 4. Points at which slag products may be used in a road structure and related research (See Fig. 9 above).

Use	Slag product and application
1. Surfacing	Crushed steel slag, alone or with crushed natural rock, as an aggregate in asphalt (Lemass 1992, Huhtakallio 1999, Emery *et al.* 1992, Miklos 2000, Noureldin & McDaniel 1990, Jones 2000)
2. Base course 3. Sub-Base[1] 4. Filter course[1]	Crushed slag, lump slag, granulated blast-furnace slag (alone or as a binding agent) and ground blast-furnace slag (as a binding agent). - Structural layer composed of a single slag product (Geyer *et al.* 1996, Lemass 1992, Emery *et al.* 1992, Liimatta & Ehrola 1999, Harju-Autti & Viljas 1993, Thom & Brown 1989, Verhasselt & Choquet 1989, Holma 1997) - Structural layer of a slag mixture or slag/natural rock aggregate mixture (Eerola & Alkio 1980, Bullen 1979, Emery 1975, Nomura & Enokido 1981, Piret *et al.* 1980, Lappi 1991, Heaton & Cao 1998, Prunevieille 2000) - Layer stabilized with a slag-based binding agent (see section 4.6)
5. Embankment fill	Crushed slags, lump slag (Emery 1992, Lee *et al.* 1997, Ghionna *et al.* 1996)
6. Mass stabilization and 7. Deep stabilization	Slag-based binding agents, principally ground blast-furnace slag (Kukko & Ruohomäki 1995, Mácsik *et al.* 1997, Vähäaho 1996, Tielaitos 1993)
8. Landscaping	Crushed slags, lump slag (Emery 1992, Lee *et al.* 1997, Ghionna *et al.* 1996)
Other purposes	Granulated blast-furnace slag: filling around land drains (Peteri *et al.* 1994), dust retention (Lukin 1995), lightweight material for building on soft ground

[1] The sub-base and filter course can be combined into a single layer composed of one material.

The technical properties common to blast-furnace slag and steel slag (converter slag) are good bearing capacity, binding and heat insulation, but steel slags are much heavier,

harder, denser and less vesicular in nature than air-cooled blast-furnace slags (Montgomery & Wang 1993, Lewis 1982b) and have a high resistance to polishing and wear in pavement surfaces. The grindability of steel slag is lower than that of blast-furnace slag, but its hardness and specific gravity are greater (Montgomery & Wang 1993).

The main problem with steel slag in massive use is volume instability, which has often proved problematic to control. As suggested by Hartlén *et al.* (1996), the durability and long-term performance of steel slag in particular must be thoroughly tested. According to Emery (1976), the finer grain sizes (-4.75 mm or #4) show greater expansion than the coarser ones, and a complete gradation of finer sizes gives the greatest expansion of all. This is not a disadvantage, however, if the steel slag is used as a binding agent, as the amounts present are small relative to the material to be stabilized and the expansion is lost in the general mass. Kneller *et al.* (1994) even suggest that air-cooled blast-furnace slag is the only type that should be used as a sub-base in road construction, while Geiseler (1996) and Alexandre *et al.* (1993) point out that no restrictions on volume stability are necessary in certain applications, e.g. unpaved roads and parking areas, and similarly dams. In contrast, where slag is used in the unbound or bound layers of roads, volume stability must remain within strict limits. The free lime content is a classification criterion for fresh slags which have not been weathered, as slags with a free lime content of up to 7% may be used for the unbound layers of pavements and ones with a content up to 4% for the bituminous layers (Geiseler 1996).

The literature survey published by Mäkikyrö (1996) establishes that the preferred maximum grain size for steel slag in both structural layers and pavements would seem to be 20-25 mm, while the optimal weathering time is dependent on the use, grain size, manner of storage and manner of processing of the slag and on its original free lime content. Mixtures can be prepared from steel slag, granulated blast-furnace slag, crushed blast-furnace slag and/or natural rock aggregate in order to achieve a true semi-rigid structure, in which case the steel slag not only gains in strength itself but it also acts as a source of calcium for the hydration reaction that takes place in granulated blast-furnace slag, while at the same time the blast-furnace slag, natural rock or both will serve to even out any volume instability that may be associated with the steel slag. (Mäkikyrö 1996)

Where water cooled blast-furnace slag is used as a binding agent an activator is required to initiate the hydration reaction in most cases. Steel slag can function on its own in this capacity, provided that it is ground to a relatively fine powder. In the case of landscaping, the material requirements are not especially strict, and therefore attempts have been made to use for this purpose slags with technical properties that do not measure up to the criteria for structural layers.

4.4 Environmental and economical considerations affecting industrial production and road construction

Environmental protection has hitherto been understood largely in terms of the prevention of emissions in connection with industrial processes, and investments in this have mostly

been aimed at increased efficiency in the purification of effluents. Nowadays, however, the environment is implicated in everything that a company does, so that the principles on which it operates, its procedures and its instructions are all closely related to quality systems. Environmental matters pose threats to commercial activities, but also present challenges and opportunities, as good environmental management can enhance a company's market position and lead to cost-efficiency (Huhtinen 1997). The steel industry invested a great deal in environmental management in the 1990's in particular, on the grounds that a part of the quality of the end product, the steel, lies in the attention that the producer is able to pay to the environment at all stages from acquisition of the raw materials to disposal of the by-products, such as slags, the use of which can lead to savings in non-renewable natural materials. According to Lemass (1992) equally as significant as the structural and economic benefits gained through the commercial use of recycled slag products are the environmental impact benefits associated with the increased potential for conservation of our traditionally specified natural resources. The use of slag products means that the energy already expended on producing iron, steel and slag will enable a potential conservation of energy in the future.

Looked at from an environmental perspective, it is important to ensure that the quality of slag products is monitored continuously. As Hartlén *et al.* (1996) mention, quality control standards and assurance programmes are essential for maintaining product quality and avoiding undesirable environmental impacts, so that quality control testing for leaching characteristics, important (application-specific) physical parameters and durability should be carried out at predetermined intervals. Correspondingly, Emery (1992) notes that the economic, environmental and technical feasibility of the use of waste and by-products as construction materials is based on the quantity available being adequate to justify the necessary processing systems, with due consideration for savings through reduced landfill costs. Distances from processing locations to construction sites should be reasonable in terms of competition with conventional materials, the processed waste or by-products should be incapable of harming the environment, given proper handling and use, and their physical, mechanical and chemical characteristics should meet reasonable, applicable specifications for the contemplated bulk or cementitious applications.

One major incentive for developing environmentally friendly production methods has been environmental taxes levied on the producers of waste. "Zero emissions" and "zero waste" have become familiar concepts in the steel industry over the last ten years (Kat 2000, Maedgen & Hunt 1998, Wrona & Julien 1997), implying the ideal of a steel works that is able to make 100% use of its by-products, either by recycling them in the steelmaking process or by processing them for resale and use outside the steel works.

One method commonly used for determining the environmental impacts of products nowadays is Life Cycle Assessment (LCA), an approach in which consideration extends from the earliest origins of a product or process, in the earth so to speak, and follows it through all the processes involved in its production, distribution, use and disposal to the point where its final remains re-enter the earth (Young & Vanderburg 1994). According to Eskola *et al.* (2000), one barrier to wide-ranging utilization of the secondary products of energy production and industry in road construction has been uncertainty about the environmental impacts (Eskola *et al.* 2000). It is possible to carry out an LCA on the principal product itself, e.g. on steel, but it can be carried out to its fullest extent only on

products manufactured out of materials that have arisen in the course of actual production. One alternative for an LCA on the building of a road, for instance, is the use of slag in place of a natural material, in which calculations would have to be made of the impacts of the two products on the environment over the service life of the road.

According to Young & Vanderburg (1994), LCAs are expected to form a significant portion of the foundation of future environmental management systems, as they will be used by manufacturers, designers, purchasers and other decision makers in the course of all activities that involve environmental considerations. Thus LCAs can in principle be used as an instrument in marketing, if the intention is to compare alternatives.

Since environmental questions have received a great deal of publicity, and since legislation develops rapidly and makes new demands on companies, this has stimulated many companies into improving their handling of environmental matters. Environmental considerations come to the fore increasingly often in discussions between suppliers and customers, the customers usually being interested above all in the following (Huhtinen 1997):

- that the product or service to be purchased should improve the properties of the customer's product over the whole of its remaining life-span,
- that the design chosen by the supplier will entail savings in terms of costs for the customer throughout the service life of the product and
- that the supplier supports the customer's public image in environmental matters.

A knowledge of the customer's needs permits the supplier to predict the demands likely to be placed on the product and the threats and opportunities created by changes in the customer's needs. It also provides a basis for comparing the environmental properties of different products and for exploiting these in the context of marketing. The point of departure is that a company is not selling products to the customer but the solutions for products (Huhtinen 1997). Even so, although environmental matters are emphasized in connection with the manufacture and selection of products, it is economic facts that frequently weigh more heavily in the final decision. The ideal situation, of course, is one in which economic facts and attention to the environment can be made to coincide in a rational manner.

The purpose of product development as far as a company is concerned is to achieve the optimal utilization and development of its principal resources: finance, assets, raw materials, energy and labour. If profitability is viewed too narrowly, in terms of the short-term financial result alone, the company's competitive potential in the long term is likely to suffer (Jaakkola & Tunkelo 1987). Attention has begun to be paid to this aspect in the field of road construction nowadays, and more weight has been placed on the costs of maintaining a road over the whole of its service life, as reflected in the adoption of Life Cycle Cost Analysis (LCCA) methods for product development purposes.

Although road structures are designed and built to last for a long time, a typical design service life for a high-class road being 20 years, their economic viability is usually assessed over a very much shorter period. The aim should be to find the most economical overall structural solution for a given road in terms of maintenance measures etc. and their costs over the whole of its service life, as allowed for in the LCCA protocol, which is available as an aid to decision-making in this respect. This overall economic viability implies that the demands of the structural service level for which the road is designed can be maintained throughout its life at minimum costs (Kalliokoski *et al.* 1995). It should be

noted that Life Cycle Assessment (LCA) is a separate thing, involving evaluation of the impact of the road structure on the environment over the whole of its service life irrespective of economic factors. The ideal, of course, would be to combine these two life cycle analyses, which would provide a more realistic picture of the total situation.

In practice, future costs of using and maintaining roads are frequently ignored when making economic assessments, usually because of a lack of adequate information and calculation methods, so that attention is paid only to the initial investment costs. Similarly, it is not always realized sufficiently clearly what contribution maintenance costs can make to the overall expenditure on roads, in addition to which there have been deficiencies in the discounting of life cycle costs and in the clarity with which the results are presented. Omission of the costs of using and maintaining a road from the economic calculations made at the beginning of a project can easily lead, for example, to better quality construction materials being placed in a disadvantaged position. Thorough economic comparison by means of LCCA can ensure that all essential cost factors are taken into account when making the necessary decisions (Kalliokoski *et al.* 1995, Flanagan *et al.* 1989, Hyartt & Saari 1993).

It is assumed in life cycle cost analysis that there are three principal types of costs involved, construction costs, maintenance costs and rehabilitation costs, in addition to which it is necessary to consider driving costs and other costs, e.g. those arising from environmental impacts and benefits and drawbacks that are not measurable in monetary terms. In principle, the LCCA system allows all the above factors to be taken into account over the whole service life of the road when seeking the most economical solution (Figure 10; see also Kalliokoski *et al.* 1995)
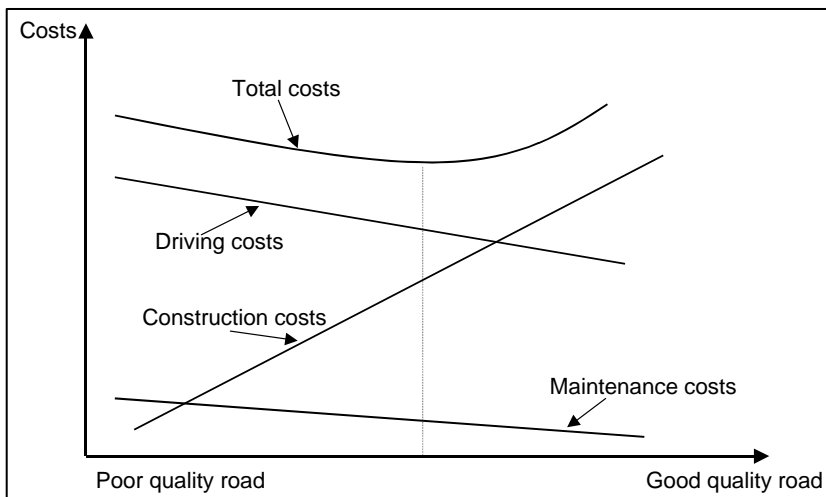
Fig. 10. Factors examined in a Life Cycle Cost Analysis of a road building project and total costs as a function of the quality of the road (Kalliokoski et al. 1995).

4.5 Loading of road structures, damage and factors affecting structural behaviour

The principal strains on a road structure are those caused by traffic loading and ground frost. A road structure must withstand these loads for decades without damage, and in order for it to fulfil the requirements set for it, the mechanical properties of the materials used in its construction must be of the highest possible quality. Contrary to general opinion, design of a road structure is a difficult task. The changes that might take place in it during its service life are much more difficult to determine than for a concrete beam, for example, as the conditions surrounding a concrete beam, such as moisture and temperature, are well known and can often be adjusted if necessary, whereas a road structure is entirely subject to the effects of weather, and the possibilities for influencing the moisture conditions are limited (Figure 11). In practice, it has often turned out that the requirements set for a road are not fulfilled and the structure is damaged before reaching its planned service life (Mäkikyrö & Hiltunen 1998).

According to Ullidtz (1987), the "condition" of a pavement can be divided into two categories: "functional" and "structural". A pavement design system (or rehabilitation design system) should not only indicate the structure needed in order to prevent an unacceptable degree of deterioration before a given time or number of traffic loadings, which is the minimum requirement for a design method, but it should also permit prediction of both the "functional" and "structural" deterioration with time. Functional condition has to do with ride quality and structural condition with bearing capacity. Pavement design is thus not a question of predicting a sudden failure (unless the pavement is grossly underdesigned) but of slow progress towards failure. The design system should be capable of assessing the condition of a pavement (functional and structural) at any time, and of predicting the consequences of changes to its structure, to the loading or to the environmental conditions.

Three main types of damage can be recognized: cracking, deformation and wearing course disintegration (Figure 12; see also Kalliokoski *et al.* 1995, Liimatta *et al.* 1990, Kurki *et al.* 1993). Cracking damage is brought about by volume changes in the pavement (thermal stress), excessive stress due to traffic loading, and depressions or frost heave in the substructure. Pavement fatigue is caused by repeated deflections brought about by heavy traffic loads, while deformation damage is a consequence of traffic-induced vertical or longitudinal deflections in the road surface. Disintegration of an asphalt wearing course will arise through traffic wear or mechanical and chemical reactions attributable to climatic conditions (Kalliokoski *et al.* 1995, Lehtipuu 1983, Liimatta *et al.* 1990).
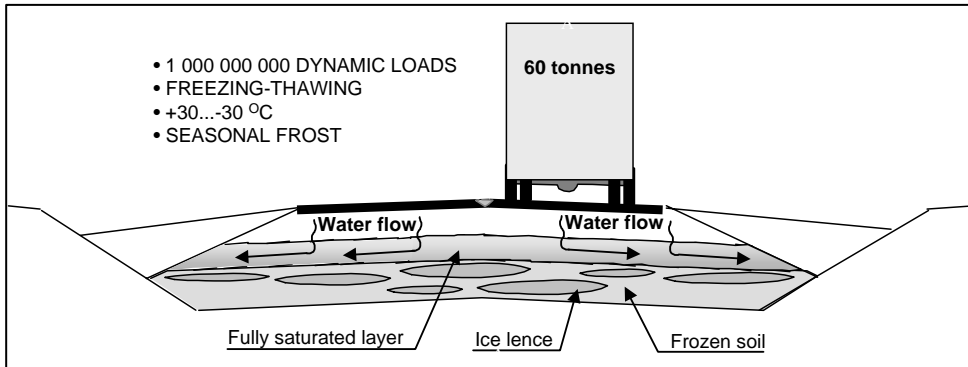
Fig. 11. Loads imposed on a road structure in areas affected by seasonal frost.
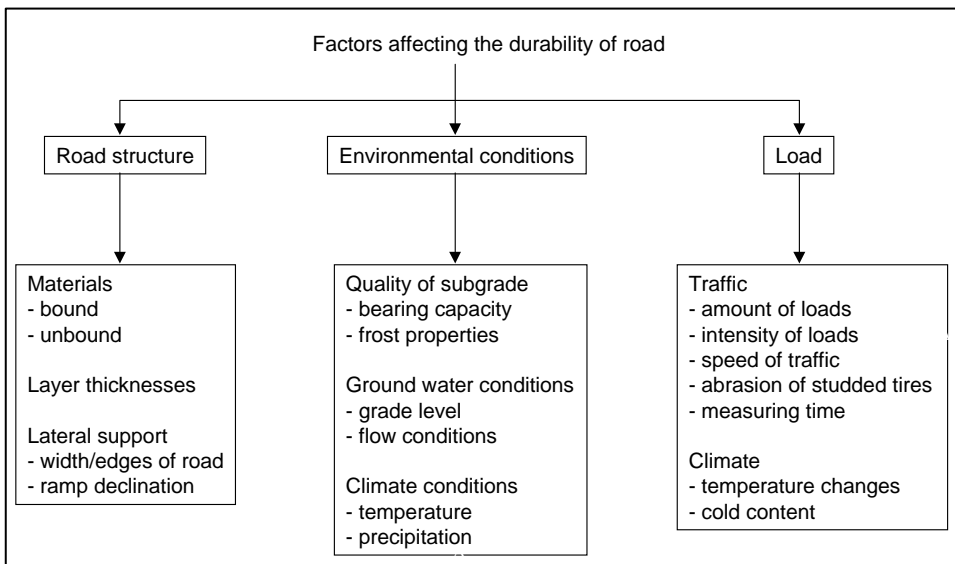
Fig. 12. Factors affecting the durability of a road structure (Kalliokoski et al. 1995).

The factors affecting the mechanical behaviour of a granular material include its stress state, density, water content, temperature, maximum grain size, proportion of fines, grain size distribution profile and the form, surface roughness and elastic and electrochemical properties of the grains (Kolisoja 1997). The influence of individual grains naturally diminishes in a stabilized structure as binding proceeds. On the other hand, water content, temperature and electrochemical properties can exercise a considerable effect on the progression of binding, and thereby on the rate at which the influence of individual grains diminishes.

It is common when describing the deformation properties of a road structure to employ the term "modulus" to refer to the relation between loading and the reversible

part of the deflection in a parallel direction. Efforts are made wherever possible nowadays to define the reversible and irreversible deflections caused by traffic loading, and to calculate the indices for these, by means of triaxial tests in the laboratory. Reversible deformation behaviour is represented in this test by the resilient modulus (M_R), first introduced by Hveem (1955) for the purpose of differentiating the terminology used for describing reversible deflections in coarse-grained materials from that normally used in connection with elasticity theory, because their deformation behaviour is influenced by many factors that are not normally connected with the elastic properties of materials (Kolisoja 1993).

The material parameters required for describing the state of stress and deformation in a coarse-grained rock aggregate in terms of linear elasticity theory are Poisson's ratio and the modulus of elasticity. Poisson's ratio is usually assumed for road construction purposes to be a constant of 0.35 (Holma 1997), but it can be determined on the basis of the axial vertical deflection and horizontal deflection of the specimen in response to the loading pulse, by means of Equation 4. A value to correspond to the modulus of elasticity is obtained from the resilient modulus for a loading pulse, calculated as the ratio of the deviatoric stress brought about by the pulse to the reversible part of the relative deformation, as shown in Equation 5.

$$\mu = -\frac{\varepsilon_3}{\varepsilon_1} \qquad (4)$$

μ is   Poisson's ratio
ε_3 reversible radial relative deformation
ε_1 reversible axial relative deformation

$$M_R = \frac{\Delta q}{\Delta \varepsilon} \qquad (5)$$

M_R is   resilient modulus (= dynamic modulus of elasticity)
Δq range of variation in deviatoric stress
$\Delta \varepsilon$ reversible axial relative deformation

The dependence of the resilient modulus of the stress state is most commonly described in terms of the power function shown in Equation 6.

$$M_R = K_1 \theta_0 \left(\frac{\theta}{\theta_0} \right)^{K_2} \qquad (6)$$

K_1 is   modulus
K_2 modulus exponent
θ sum of principle stresses ($3\sigma_3 + \Delta q$)
θ_0 reference stress

The deformation behaviour of a completed road structure is measured using a falling weight deflectometer, the idea of which is to reproduce as realistically as possible the

load exerted on the road structure by the wheels of a heavy vehicle, in terms not only of magnitude but also of duration, whereupon the condition of the road can be evaluated by reference to the deflection observed. This deflection consists of both elastic and plastic deformation components, although the plastic, or permanent deformation may be assumed to be negligible relative to the achievable accuracy of measurement in the case of individual loads on an asphalt pavement, so that the deflection can be regarded as being entirely of the elastic kind. Substantial plastic deformation can be observed at high pavement temperatures, however, especially if the underlying structural course is more rigid, e.g. cement-stabilized soil.

The KUAB falling weight deflectometer employed here, for instance, operates on the double weight principle, in which a 50 kN weight is dropped on top of another one, as a result of which the load impinges on a plate of diameter 300 mm that is divided into four sectors to ensure that the weight is eventually distributed as evenly as possible over the surface of the structure to be measured. The deflection caused by the loading is measured with sensors, of which the KUAB device has six, one in the centre of the loading plate and the others 200, 300 or 450, 600, 900 and 1200 mm away. The deflection detected by each sensor is assumed to represent that occurring at the corresponding depth in the road structure, e.g. that measured 1200 mm away from the loading point will correspond to the situation at a depth of 1200 mm. The greater the deflection at such a depth, the weaker the subgrade, and correspondingly, the greater the deflection at the surface, the weaker the upper parts of the structure.

About 60-80% of the maximum deflection of a road surface, i.e. that at the centre point of loading, consists of elastic deformation of the subgrade (Ullidtz 1987, Liimatta 1996), and the remainder mainly of compression of the unbound pavement layers, the bound pavement layers accounting for no more than a few percent of the maximum deflection. The figure for maximum deflection provides a good impression of the average situation in the whole structure. A second parameter descriptive of the condition of a road structure is the form of the deflection caused by loading of the surface, i.e. the deflection basin. This can be used to estimate the condition at different depths in the structure and to define E-moduli for the various structural layers by back-calculation. A stabilized slag course can be regarded as a semi-rigid layer with a behaviour under loading which deviates from that of bitumen and cement-bound courses. Thus the durability criteria laid down for rigid courses cannot be applied directly to a slag-stabilized one, as they are dependent on the tensile stress at the undersurface of the bound course (Liimatta 1996).

It is common nowadays to determine the E-moduli of the structural layers of roads *in situ* by back-calculation from the deflection profile, the information required for which consists of the material compositions, thicknesses and Poisson ratios of the layers and the temperature of the pavement (Figure 13). Data on the material composition of the layers are of importance mostly for setting initial values for the E-moduli, while the thicknesses have to be known precisely in the case of bound layers. Although the actual thicknesses of the unbound layers frequently deviate from the design thicknesses, their use in the back-calculations will not give rise to any appreciable error. A Poisson ratio of 0.35 may safely be used for bitumen-bound or unbound material, while figures of 0.25...0.35 are recommended for cement-bound layers, depending on their rigidity. (Liimatta 1996)

Falling weight deflectometer readings serve well to depict the behaviour of a road

structure in response to traffic loading, although the method does have the drawback of sensitivity to temperature, as the road will be surfaced with a thick bitumen pavement.
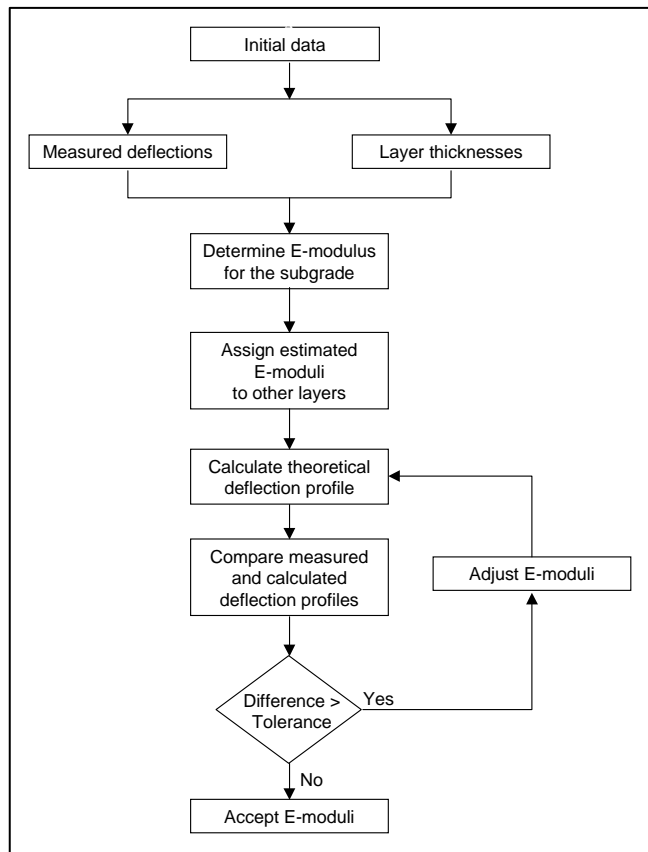
Fig. 13. Determination of E-moduli from the deflection basin (Liimatta et al. 1992, Liimatta 2000).

The effect of the road structure itself on durability is to a great extent a matter of choice, relevant factors which include the class of road concerned and thereby the service requirement placed upon it. The current trend is away from unbound structures and towards bound ones, together with an increase in the popularity of semi-rigid structures, which avoid the problems associated with rigid ones, such as cracking, but are able to take advantage of the improvement in bearing capacity to be achieved through binding.

Freezing of the road base and the resulting phenomenon of frost heave can constitute serious loading factors under conditions of seasonal frost, and numerous theories have been put forward to explain such effects. The concept put forward by Pufahl (1996) is that the freezing process involves a state of dynamic equilibrium in which the heat demand caused by the temperature gradient from the ground to the atmosphere is balanced by heat supplied to the freezing front in the form of sensible heat and the latent

heat of the phase change in the water being drawn upward. As long as this balance is maintained, an ice lens will grow near to the zero degree isotherm until changes in one or more variables cause the freezing front to move deeper into the soil. The process is repetitive, and ice lenses or ice bands are formed in the frozen soil, the thickness and spacing of these being dependent on the rate of cooling, moisture flow, soil characteristics and overburden pressure.

Pufahl (1996) refers to two earlier studies when describing the occurrence of frost heave. In the first place, it is generally accepted that three conditions must exist for heave to occur in soils (Johnston *et al.* 1980): (1) a frost-susceptible soil, (2) a supply of water, and (3) sufficiently low temperatures to cause the soil to freeze. Secondly, frost heave may be described in terms of four interrelated events (Lunardini 1981): (1) the freezing process, (2) moisture movement, (3) ice segregation, and (4) deformation or heave in the soil.

Perhaps the most widely used engineering model for predicting frost heave is that founded on the work of Konrad and Morgenstern (1980, 1981, 1982a, 1982b). For a system with a given soil, temperature gradient and surcharge load, the flow of water to the growing ice lens is proportional to the temperature gradient in the frozen soil.

The theory of Konrad and Morgenstern has been accepted by engineers for dimensioning purposes, perhaps largely because it is easy to use. On the other hand, of the frost susceptibility parameters, only segregation potential has been applied as such to the calculation of frost heave (Kujala 1991), although this point has also been criticised as far as its theory is concerned, as it may not necessarily hold good for all areas of the gradient. If the gradient is steep, the soil may freeze in a thick column at once without the formation of any lenses, while if it is very shallow, the upward suction predicted in the theory may not take place. In view of these facts it may be assumed that there are upper and lower limits to the gradient at which the theory holds good. The concept of ice lens formation in itself is widely accepted in the form proposed by Konrad and Morgenstern, and no substantial amount of criticism has been levelled at it.

Pufahl (1996) concludes from the literature that it is not difficult to imagine a substantial loss in strength when soils with segregated ice begin to thaw. Water may be temporarily trapped within the subgrade, resulting in large positive pore-water pressures and a concomitant reduction in the effective stresses. Deflection of a pavement surface during the spring thaw does not depend on the depth of frost penetration alone. For a given pavement structure, maximum deflections are based on combinations of ground water conditions and thermal gradients during freeze-thaw cycles in the upper subgrades. Thaw weakening occurs in fine-grained subgrade soils without any global increase in water content, being related to the redistribution of moisture within the soil mass. Frost crystals accumulate on the surfaces of the nuggets that are characteristic of the secondary macrostructure that forms in fine-grained soils after several freeze-thaw cycles, and this water is released during the thaw, causing a marked decrease in matrix suction. As a result, the resilient modulus is reduced, so that maximum deflection in the pavement surface is observed under these conditions.

Ambient conditions can be adjusted mainly by means of drainage systems and the choice of materials, the principal aim being to eliminate the problems caused by freezing in areas with seasonal frost. The most critical time of the year consists of a few weeks in the spring when the road surface has begun to thaw but the lower structural layers are

still frozen. This means that the water from the melting frost is unable to percolate out of the road structure and a saturated layer forms in which the bearing capacity is greatly reduced on account of a decrease in friction between the particles (Figure 11 above). As shown in the schematic representation of bearing capacity at different seasons of the year in Figure 14, such a road structure is in a normal state with regard to bearing capacity for only a few months in the year. At other times it is either frozen, thawing, returning to normal or in the process of freezing. There are scarcely any technical means available for influencing traffic loading on a particular stretch of road, so that the only approach will be through utilization policy decisions.

Jong *et al.* (1998) propose the following generalized description for the behaviour of the base and subgrade resilient moduli. The consistency between the trends in frost-thaw depths, deflections and resilient moduli suggests that there are two important transition times in the thaw-weakening and recovery process (Figure 14): the first period is referred to as the weakening stage, which begins when the base course begins thawing and continues until the pavement system is completely thawed. During this period the stiffness of the pavement system decreases. The second period, referred to as the recovery stage, begins when the pavement is completely thawed and ends when full stiffness is attained. The pavement stiffness increases during this stage.
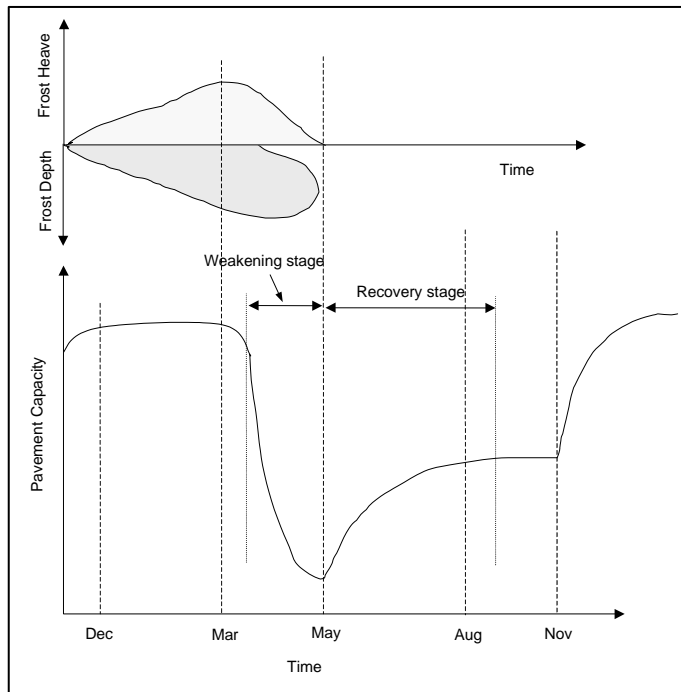
Fig. 14. Conceptual changes in pavement stiffness caused by freezing and thawing (Jong et al. 1998). Frost depth and frost heave has been added to the upper part of the diagram by the author.

Research has been conducted into the relation between frost, and particularly the thawing phase, bearing capacity and the associated factor of frost depth in order to assess variations in bearing capacity at different measuring times and determine the correct design bearing capacities for given road structures. The same experiments have also been used to assess the effects of the subgrade, and above all of its moisture content, on bearing capacity (Andrew *et al.* 1998, Bergstedt 1990, Ehrola *et al.* 1990, Janoo & Berg 1990, Jong *et al.* 1998, Simonsen *et al.* 1998, Van Deusen & Newcomb 1994, Wiman & Jansson 1990).

4.6 Stabilized structural layers

4.6.1 Purposes and aims of stabilization

When confronted with *in situ* material at a job site, an engineer has three choices (Lay 1986):
a) produce a design using their inherent properties;
b) remove the local material and use better, imported material; or
c) treat (i.e. stabilize) the *in situ* material.

   Although the simplest stabilization processes are in effect compaction, drainage, improved grading and the use of additives (Lay 1986), the term is usually understood as referring to a technique devised for strengthening a structural layer by adding a binding agent to improve its bearing capacity. The granulated blast-furnace slag stabilization technique is used extensively in Finland to improve bearing course material that has become susceptible to frost. In the narrow sense of the term, stabilization is the use of hydraulic or bituminous binding agents to strengthen the upper structural layers of a road, the most commonly used agents being cement or bitumen, which bind the grains to one another, although lime stabilization is also an accepted technique.

   The main effects of stabilization (Lay 1986) are:
1. to provide a working platform in soft material,
2. to improve the properties of sub-specification material,
3. to reduce moisture susceptibility,
4. possibly to produce a bound layer, and
5. to reduce permeability (provided that no cracking occurs).

   The principal aim in the stabilization of the structural layers of roads is shown in Figure 15 below. The purpose of the structural course created by stabilization is to improve the load-bearing capacity of the whole road, and as a consequence of the stabilization the upper part of the pavement structure, the bearing course and sub-base, will increase in rigidity and distribute the traffic loads over a broader area than would be the case with an unbound structure. This heightened rigidity in the upper part will also reduce deflection in the subgrade.

   Stabilization can also exert an effect in the opposite direction, i.e. its purpose may be to even out the loads caused by frost heave arising from below the road structure and

thereby to reduce the risk of frost damage to the pavement. It is also possible to install a reinforcement net in the stabilized layer to further reduce the likelihood of cracks in the pavement.
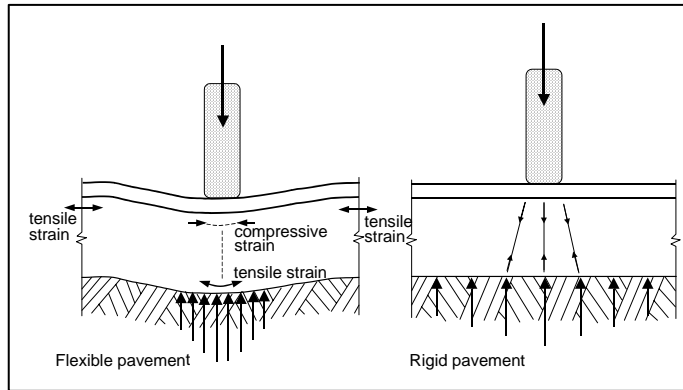
Fig. 15. Response of two pavement types to loading (Lay 1986).

Research into alternative binding agents and stabilization techniques increased markedly in the 1990's, when particular attention was devoted to the use of industrial by-products for this purpose and to the development of *in situ* mixing techniques in an attempt to achieve the same quality as with cement station mixing.

Ylipiessa (1998a), citing the research of Lindgren (1980) and the National Roads Administration (Salmenkaita 1993), reports that contrary to the situation with concrete, the strength of which depends on that of the rock from which the cement was derived, the strength of stabilized soil is based on the internal angles of friction created by the cement and the increase in cohesion between the soil particles. A cement-stabilized structure does not have the rigidity of concrete, but might almost be said to be flexible. This may be attributed to hairline cracks appearing in it.

It must be said, however, that the term stabilization means different things in different cases. Most researchers speak of stabilization when a structural layer is produced simply by mixing various slag products together, obviously in a desire to indicate that this is a binding structural layer that has to a great extent the same properties as a bound layer.

4.6.2 Factors affecting the outcome of stabilization

Although most research into factors affecting stabilization is based on the use of cement as a binding agent, the same regularities hold good for all hydraulic agents, although some factors may be of greater or lesser importance. The detrimental effects of humus in the material are greater with granulated blast-furnace slag stabilization than with cement stabilization, for instance, on account of the fact that a source of lime is required to neutralize the humus as well as to provide an activator to initiate and maintain the granulated blast-furnace slag reaction.

Chemical factors affecting strength include the chemical composition of the fines in the base material and the organic content of this material. Also, sulphates and sulphites contained in the base material and carbon dioxide present in the pores of the stabilized soil can detract from the result of stabilization under certain conditions, while calcium carbonate in the base material will improve the strength. Although the majority of soils requiring stabilization consist mostly of gravel and sand fractions, these materials as such have no effect in stabilization, on account of their small specific surface. The chemical composition of the fines in some soils requiring stabilization can affect the outcome, however (Ylipiessa 1998a, Sherwood 1995). Organic matter is to be found to a significant extent only in surface soil, and even then the nature of this is more important than its amount, as not all organic substances interfere with hydration (Ylipiessa 1998a, Sherwood 1995, Lindgren 1980, Salmenkaita 1993). Humic substances make up a high proportion of the organic matter contained in soils, perhaps as much as 80-90% (Rantala 1990), so that they are of considerable significance for stabilization. The outcome can also be affected by impurities in the water used in the stabilization process, again largely organic matter, and this can increase the organic content of the stabilization material and thereby prevent hydration.

Balcázar (1998), noted that LD slag is composed principally of calcium silicates and ferrites, so that hydrolysis in water and the presence of H^+ ions in acidic soils produces divalent ions (mainly Ca^{2+} and Mg^{2+}), which become part of the soil's humus-clay exchange complex. The main chemical reactions between the mineralogical components of a slag and the protons that cause acidity in the soil are shown in Figure 16. These reactions, which form the basis of the action of the slag on the soil, involve the removal of protons from the humus-clay exchange complex and their replacement with calcium and magnesium cations (Balcázar 1998). The use of slags as binding agents in stabilization involves to a large extent the same reactions, except that the amounts of slag present have to be greater for binding to take place.

CHEMICAL REACTIONS IN THE SOIL:

$$SiO_5Ca_3 + 6H_+ \longrightarrow 3Ca_{+2} + SiO_2 + 3H_2O$$
$$SiO_4Ca_2 + 4H_+ \longrightarrow 2Ca_{+2} + SiO_2 + 2H_2O$$
$$2CaO.Fe_2O_3 + 10H_+ \longrightarrow 2Ca_{+2} + 2Fe_{+3} + 5H_2O$$
$$FeO.MnO + 4H_+ \longrightarrow Fe_{+2} + Mn_{+2} + 2H_2O$$
$$CaO + H_2O \longrightarrow Ca(OH)_2$$
$$Ca(OH)_2 + 2H_+ \longrightarrow Ca_{+2} + 2H_2O$$
$$MgO + 2H_+ \longrightarrow Mg_{+2} + H_2O$$
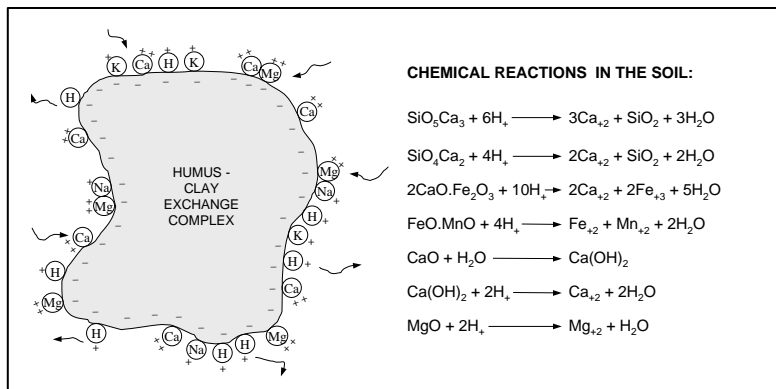
Fig. 16. Exchange of protons in the soil exchange complex and typical chemical reactions in the soil (Balcázar 1998).

The main physical factors affecting the strength of cement stabilization are the density and moisture content of the stabilization mass, the amounts of fines in the binder and the

material to be stabilized and any additives present (Ylipiessa 1998a, Sherwood 1995). Cement begins to hydrate and form bonds immediately on the addition of water, but these bonds break as the stabilization mass is compacted, so that their contribution to stabilization strength disappears. This means that compaction should take place as quickly as possible after mixing (Ylipiessa 1998a, Sherwood 1995). Slag-based binding agents, however, react more slowly than cement, so that compaction is not such an urgent matter, especially since granulated blast-furnace slag is able to form new bonds, allowing re-binding at points where a once hardened structure has been broken.

When using cement for stabilization purposes, the water content affects both compaction and the hydration of the cement. The best strength values are usually achieved by compaction at a certain optimum water content in the stabilization mass, which gives the maximum dry density for the material. The compression strength of cement stabilization increases with increasing maximum grain size in the aggregate, the highest values being attained when the grain-size curve for the soil is close to the densest packing curve, in other words the grain-size distribution enables the greatest bulk density to be attained in the compacted mixture. It is also necessary for the fines content of the material to be at the optimum level, as deviations from this will reduce the compression strength. (Ylipiessa 1998a, Sherwood 1995, Lindgren 1980, Salmenkaita 1993)

4.6.3 Stabilization techniques and their applications to the use of slags

Stabilization may be carried out *in situ* or by preparing the material at a mixing station. Employing the *in situ* technique, mobile equipment is used to prepare the material at the site to be stabilized by first spreading the binding agent and then mixing it with aggregate existing there or brought to the site separately, after which the material is compacted, levelled off and surfaced. In the opposite case, the material is prepared at a mixing station, from which it is transported to the site, spread, compacted and finally surfaced. The most significant advantage is that the material can be mixed very much more evenly in this way, while the main drawback is that the method is more expensive. As noted by Höbeda (1988), one essential factor for achieving good results is that the material should be mixed and spread so efficiently that it is adequately homogeneous in composition throughout. Problems can arise if extremely small amounts of binding agent are added. The treatment of weak structural layers with a slow hydraulic binding agent is a more or less "foolproof" method, but only under favourable substrate conditions. One should avoid materials with a high frost susceptibility, as this will lead to detrimental cracking effects.

The model for successful stabilization with granulated blast-furnace slag may be taken as being the French *grave laitier* technique, which involves the preparation in a mixing plant of a gravel-sand base composed of an 0/D gravel and a certain quantity of vitrified slag (granulated, pelletized or ground) together with a basic activator, normally at a rate of 1%, for a water content varying from 6 to 8%. The aggregate is obtained by crushing massive rock or an alluvium reservoir. This is followed by screening into classes to obtain a well-defined grading curve inside the mixing plant (Pascal 1993). Based on

information from Howard (1988), Heaton & Cao (1998) report that 15-20% unground granulated slag plus 1% lime activator can be used as the binder at sites close to an iron works, where transportation costs are low. Further away, 8-10% coarsely ground granulated slag plus 1% lime activator is used, and still further, 3.5-5% slag cement. Low-grade gypsum soda is often substituted for the lime activator. Several of the more interesting "*grave laitier*" blends tested have incorporated BOS steel slag as a source of lime to activate the granulated iron blast-furnace slag. One of the most promising mixtures contained fly-ash in addition to granulated slag and steel slag (Heaton & Cao 1998). The *grave laitier* technique involves preparation of the material at a mixing station and transportation over long distances in some cases, whereas stabilization with granulated blast-furnace slag in Finland almost always takes place *in situ*. Kennedy (1987) reported that the same approach was also used in Great Britain in the 1980's.

Research into the stabilization of the structural courses of roads has provided a great deal of information on the use of cement for this purpose, while corresponding projects involving industrial by-products have mostly been concerned with the use of powdered binding agents instead of cement. A typical example is the work of Wong (1992). Practically no research has been done, however, into the use of more coarse-grained slags, as the idea has been to develop strengthened structures comparable to those achieved by cement stabilization. Thus the use of slag-based binding agents that react more slowly than cement has represented attempts to extend the workability time of the structural course as well as to reduce binding agent costs. Moffat & Sharp (1999) report this kind of experiments in Australia when using granulated slag, fly ash and lime in road stabilization.

The most common topic of such research has been the use of ground blast-furnace slag with various activators. According to Wilmot (1994), slag and fly ash both require 8% to 10% hydrated lime to activate the reaction in full, but in order to ensure that all the fly ash and slag is fully consumed in the reaction, it is normal for blends to contain a minimum of 12% to 15% hydrated lime. Further hydrated lime in the blend is then available to stabilize the clay fines present in the host material. Gravels containing a relatively high clay content have typically been successfully stabilized using a 50/50 blend of slag and hydrated lime, or alternatively, a triple blend containing 50% hydrated lime, 30% fly ash and 20% slag. There is an optimum blend for the hydrated lime percentage in the mix, which occurs when the maximum amount of slag (or fly ash) is available to react with the free hydrated lime after some hydrated lime has been used in the modification of the clay fines. It follows that good-quality non-plastic materials will respond best with a mix such as 85% slag and 15% hydrated lime (Wilmot 1994). The problem with the use of lime is that it may detract from the eventual strength of the structure if applied in excess. Similarly, the use of too many components in the binding agent can cause difficulties in preparing a homogeneous material, leading to excessive scatter in strength properties within the structure.

Although the majority of the existing research has been concentrated on blends of ground blast-furnace slag and lime, other combinations have also been tried. Akinmusuru (1991) studied the use of steel slag in powdered form as a stabilizer for soil, and Viljas (1996) examined the suitability of lime and sulphate-activated ground blast-furnace slag for stabilizing the materials forming the bearing course of a road.

5 Studies, materials and methods

5.1 Studies

5.1.1 Basis for the research programme

The point of departure for the present work has been that slags are industrial by-products which will be developed into products. As a result of this development work we have not only slag-derived binding agents but also road structures with improved bearing capacity brought about by the use of these binding agents to create a stabilized structural layer and a procedure for dimensioning such structures, Figure 17.

Looked at from the perspective of the road builder and designer, perhaps the most significant product of all is the dimensioning procedure, although even this is not an unambiguous thing. As Lay (1986) points out, design procedures for stabilized pavements fall into one of two categories:

1. The pavement is designed to be flexible, with perhaps an increase in the assumed stiffness of the stabilized layer. This permits little advantage to be gained from stabilization other than to bring naturally sub-specification material to within the specification requirements for an unbound course. This enhancement of the properties of poor material is the more common of the two categories.

2. The pavement is designed on a structural analysis basis, utilizing the new characteristics of the stabilized layer, particularly its tensile strength. There is unfortunately no simple design procedure for stabilized pavements. In all major cases, any enhanced properties to be used in design should be established by prior testing of samples and with recognition given to the consequences of subsequent shrinkage cracking. (Lay 1986)

The *in situ* technique of stabilization with granulated blast-furnace slag considered here is to some extent a combination of these two categories, depending on what the purpose of using the granulated slag is. If it is used for stabilization together with an activator, the aim is quite clearly to produce a bound structure, although not one that is as rigid as with soil cement produced by traditional cement stabilization. Frequently,

however, blast-furnace slag is employed as such, without an activator, in which case it accounts for a higher percentage of the material to be stabilized than when activated. The aim then is primarily to improve the grain-size distribution of the material making up the bearing course, which may have gained an excessive proportion of fines and even become susceptible to frost heave. It is also possible at the same time that the granulated blast-furnace slag may begin to harden, depending on the base material, in which case the stabilized layer will at least to some extent become semi-rigid.
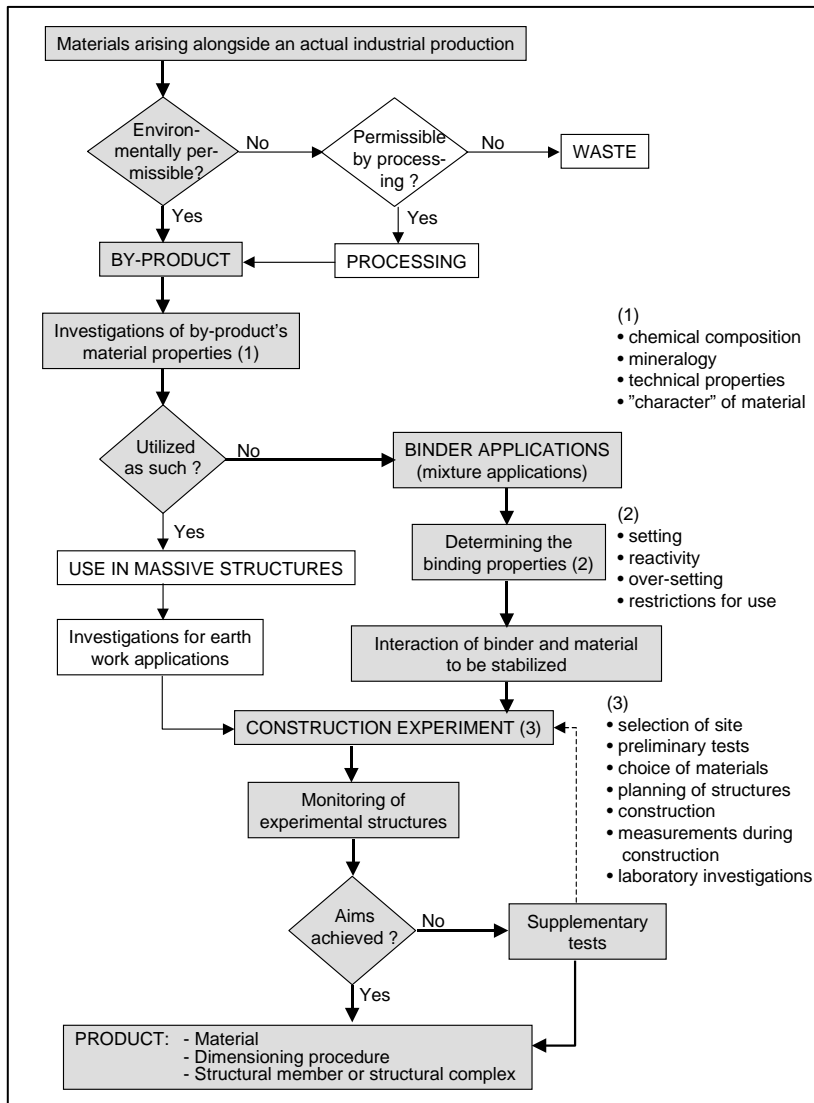

Fig. 17. Scheme for the commercialization of a by-product to be used in civil engineering and road construction in terms of its physical properties. The commercialization routes considered in this work are shaded.

The procedure set out in Figure 17 presupposes that the granulated blast-furnace slag and LD steel slag that arise as by-products of iron and steel production are environmentally permissible materials as such, without any further processing, although the former can be further processed by either crushing in a roller mill or grinding to a powder, while the latter can be crushed, magnetically separated and sieved to obtain a 0-3 mm crushed LD steel slag, which can then be further crushed in a roller mill or pulverized. Thus granulated blast-furnace slag and LD steel slag may be regarded as by-products that are developed into products either as they are or by crushing or grinding, or else by developing various mixtures that include them. The range of potential products is in principle infinite, because it is always possible to vary the components or proportions of the mixtures proposed to yield an endless number of slightly different slag-based binding agents.

The first stage in the commercialization process consists of determining the material properties of the by-product, for both blast-furnace slag and steel slag. This involves chemical analyses and mineralogical determinations, on the basis of which it is possible to define the "basic physical properties" of the material, including the degree to which it is capable of binding and whether it contains any volatile components. In other words, in addition to factual information, a picture is formed of the "nature" of the by-product. One should then be able to decide on the basis of this account of the material properties of the slag whether it is possible or rational to use it in the massive structural layers of roads, either as such or after further processing, or whether added value can be achieved by using it in a mixture or as a binding agent. This choice may also be influenced to a great extent by the market situation, in that use in massive structures enables far greater quantities of product to be utilized of than use as a binding agent, but at the same time the value of the additional processing and the returns obtained on the material will be lower.

The second phase of the commercialization process in the case of use as a binding agent will consist of determining the binding properties of the material, mostly by means of laboratory experiments. This comprises assessments of how powerfully it is capable of binding at economic concentrations, how rapidly binding takes place and which mixtures of binding agents can be expected to yield the best results in relation to cost, the market situation and the binding requirements. An essential part of this stage comprises determination of the factors related to the material to be stabilized that will affect the functioning of the binding agent, as this will enable limitations to be placed on its use in terms of what kinds of material it is in general feasible to stabilize using it. Once the properties of the binding agent and the limitations imposed by the material to be stabilized have been determined, a study must also be made of the behaviour of the stabilized layer, i.e. how it is strengthened and to what extent. The criterion at this point is not necessarily how much strengthening is achieved, but rather whether this strengthening is sufficient. When activated granulated blast-furnace slag is used as a binding agent, for example, a lower strength level is adequate than in cement stabilization, because the material is capable of re-binding and the aim is to achieve a semi-rigid structure.

The third phase consists of a construction experiment designed to verify the laboratory findings. Following selection of a site, preliminary tests are performed in the laboratory, consisting of the analysis of samples from the site, determination of factors affecting

stabilization in the base material and preliminary selection of suitable binding agents based on the results of these tests. If the factors affecting stabilization in the given material are known accurately for particular binding agents, considerable savings in time can be achieved at this point, as the preliminary choice of binding agents can be made fairly accurately, i.e. it can be determined from the material properties what binding agent will function best and how much of it will be required to obtain the desired strength. This will also mean that the preliminary choice will come relatively close to the final choice, and it will also mean that uniaxial compression tests performed with the preliminary material will suffice to enable the correct "fine adjustment" to be made to the proportions of the materials without the need for any further experimentation. Once the binding agents have been selected on the basis of the laboratory tests, the structures themselves can be designed. Since it is naturally impossible to attempt any actual dimensioning, the most sensible approach is to produce the experimental structures in connection with an existing project, in which case the same thicknesses can be used for the most part as in the project proper, although some deviations can be made in order to obtain additional information, and the structure implemented in the actual project can serve as a reference. The necessary measurements required for product development purposes can be made in the course of construction, and samples can be taken for supplementary laboratory tests. The results of the preliminary tests, of the monitoring of the test structures and of the laboratory tests on samples taken in the course of the work can then be combined to enable the findings for the whole process to be analysed and compared. This may enable some preliminary tests to be excluded in future as being unnecessary, while other new ones may be added if the need arises.

Monitoring of the experimental structure should continue for a sufficient length of time. Two years is an absolute minimum for slag-based binding agents, for instance, for it is only then that it becomes possible to assess whether the initial aims have been achieved. In the worst case it may be necessary not only to perform supplementary laboratory tests but also to produce new experimental structures, which will mean that the commercialization process will be prolonged by some years. If the aims are achieved, however, and no supplementary tests are required, a set of products may now be defined, comprising materials (binding agents), a dimensioning procedure, or even a whole set of instructions for use, and a structural component or entire structure.

5.1.2 The research programme

This research set out from the idea that improvement of an old road structure should make as efficient use as possible of the materials already in existence at the site and of industrial by-products, in this case steelmaking slags. The principal issue was thus stabilization of the road structure from the strength viewpoint. In the light of earlier experiences with both granulated blast-furnace slag stabilization and cement stabilization, it was decided that principal emphasis should be placed on the examination of compression strength. As observed by Wilmot (1994), "while UCS has little relevance to pavement design, it is a relatively inexpensive indication of the effectiveness of

various binders in the host material." The drawback with the compression test, of course, is that it does not tell us very much about the functioning of the structure, but compression strength analysis was nevertheless selected for use here because the test is simple and quick to use, easily repeatable and does not call for complicated equipment or experimental arrangements. In addition, the results are unambiguous and easy to interpret by comparison with dynamic triaxial tests, for instance. Compression strength was also selected as the parameter to be examined because when deciding on the dimensioning of a road structure requiring stabilization, and especially when seeking new solutions to the problem, it is found that compression strength in combination with the monitoring of a test structure will lead to results of a quality that will allow suitable dimensioning parameters to be defined.

The research was divided into three consecutive parts, setting out from results obtained by the author in his earlier research (Mäkikyrö 1995). This meant that the research programme comprised four themes, as depicted in Figure 18.

The first stage was to evaluate the results obtained in the author's licentiate thesis with regard to factors affecting the strength achieved in a road base stabilized with blast-furnace slag and the significance of these factors. A series of experiments were then performed to check the validity of these results. The aim was to obtain a set of about ten base materials for stabilization, selected on the criterion that they should be difficult to stabilize. This criterion was adopted because it was hoped in the confirmation of the results of the first phase to assess above all the applicability of the stabilization classification to base materials whose stabilization properties are uncertain. The plan at that stage was to determine for each base material only those properties that had been shown in the first stage of the work to be essential for the choice of binding formulae and proportions.

The third stage involved extending the range of binders to encompass LD slags as well. In accordance with the plan, the aim was to choose the LD slags so as to be as closely similar to blast-furnace slag as possible, i.e. approximately the 0-3…5 mm fraction, the product of crushing this fraction and the further product after grinding. The idea as far as the crushed product in particular was concerned was that it should be possible to produce blast-furnace slag and LD steelmaking slag by the same technique. In the end, however, a coarser grain size was accepted for the ground LD slag than for the blast-furnace slag, on account of the fact that the former was intended as an activator, and as such its activating capacity could scarcely have been increased significantly by grinding it any finer. The plans included prior determination of the functioning of the binding agents alone and in combination by means of maximally large-scale paste tests. Given that each component was to serve alone as a binding agent in the tests, these included six tests with just one component, i.e. granulated blast-furnace slag (GBS), partially ground granulated blast-furnace slag (PGBS), ground granulated blast-furnace slag (GGBS), LD slag 0-3 mm (LD1), partially ground LD1 (LD2) and ground LD1 (LD3). These components were then to be used in various combinations, of which 12 were prepared (see Table 6 below), each in five proportions, to yield a total of sixty slag-based binder combinations.

The plans at the third stage also included paste tests on the well functioning binding agent combinations to assess their performance in the stabilization of crushed rock aggregate, selected as being a maximally "pure" base material that would be free of

humus, which is apt to retard stabilization. The aim was to test 5-10 new binding agent combinations in addition to those used at the second stage.

The final stage then consisted of the construction of an experimental road, together with the necessary preliminary and follow-up tests and measurements, the accent being on analysis of the behaviour of the experimental structures. The purpose here was to determine under laboratory conditions the applicability of the combinations chosen on the basis of the results of the third stage as eventual binding agents. The plan was to obtain four base materials capable of effective stabilization and to perform stabilization tests on them with a sufficient variety of binder formulae. Proceeding according to this plan, a total of 12 binder combinations emerged as suitable for testing, in addition to which tests were carried out with cement at three concentrations. Still in accordance with the original plans, seven binding agent combinations were selected for "final" testing, employing cement at two concentrations as a reference. Thus the intention in the final tests was not to use all the base materials with all the binding agents but to select the necessary and most functional formula for each base material in accordance with its particular properties.

The intention in the field tests was to produce one test structure with each slag-based binding agent, with a cement-bound structure as a reference. Departing from the original plan, however, it was decided to produce two test structures using granulated blast-furnace slag activated with LD slag, in order to determine to what extent it was possible to compensate for poor strength properties in other layers by increasing the thickness of the stabilized layer. Thus five test structures were produced in all and monitored over a period of three years by means of bearing capacity measurements carried out three times each summer, damage surveys every spring, frost heave levelling once a year and vehicle-mounted borings two years after construction. The monitoring scheme is presented in Figure 18. It should be noted that the work reported here is a case study of a single road construction project that comprised five experimental structures.
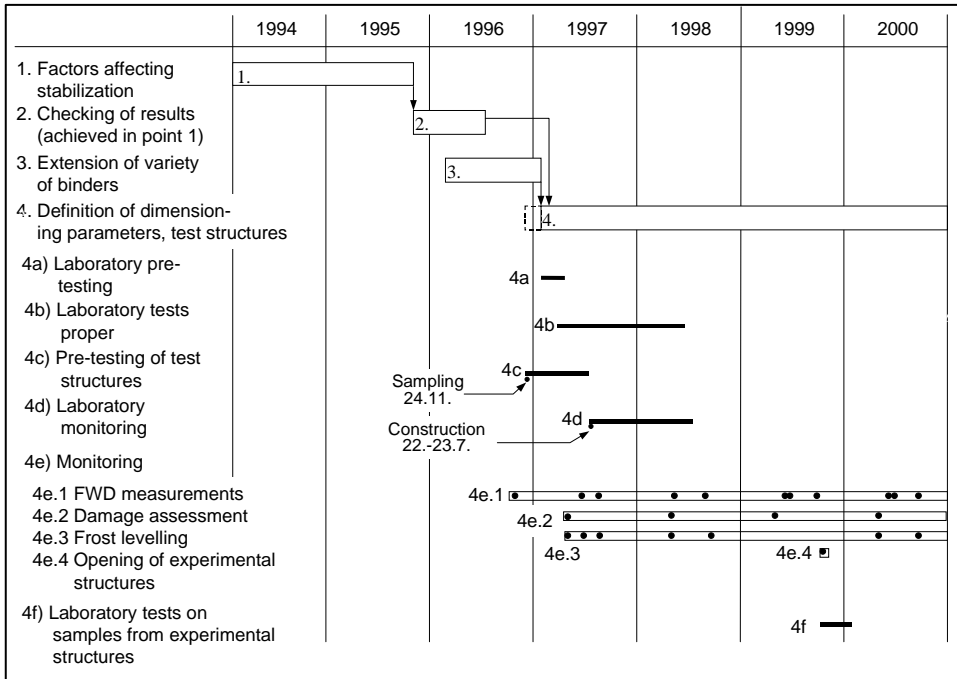
Fig. 18. The research programme which was divided into three consecutive parts, setting out from results obtained by the author in his earlier research (Mäkikyrö 1995).

5.2 Materials

5.2.1 Study of factors affecting stabilization and strength classification

The first phase of the research involved the selection of 38 old road construction materials and 4 new materials comprising various types of crushed rock aggregate (Mäkikyrö 1995) and the determination of their grain-size distributions, humus content, pH, electrical conductivity, humus classification, modified humus classification, exchangeable cations, rock types and minerals, shape coefficients, maximum dry density, optimal moisture content and mineral composition. These determinations were used to calculated the following indices:

- sieving results, 27 factors
- parameters calculated from grain-size curves, 11 indices
- parameters calculated from grain shapes, 3 indices
- properties dependent on organic content and chemical conditions, 16 factors

Thus a total of 57 factors affecting the achievable stabilization result and indices describing these were studied, the results being analysed statistically by forming a

correlation matrix for all the factors and indices by means of the SPSS for Windows program.

The main binders investigated were 0-3 mm granulated blast-furnace slag (GBS) and ground granulated blast-furnace slag (GGBS) from the Raahe Steel Works of the Rautaruukki Company, although some tests were also performed on partially ground granulated blast-furnace slag (PGBS). The aim was to obtain preliminary information on the effects of crushing the slag on its properties as a binding agent when activated with cement (CEM I 52.5), employing ordinary Portland cement (CEM II A 42.5) as a reference material and testing the specimens at ages of 28 and 91 days. The binder formulae were the following:

- GBS: aggregate 89%, **g**ranulated **b**last-furnace **s**lag **10**%, activator **cem**ent **1**% (= GBS10/CEM1)
- GGBS: binder content **3**% of dry weight of aggregate, binder composed of **70**% **g**round **g**ranulated **b**last-furnace **s**lag and **30**% activator **cem**ent (= GGBS-CEM 3(70/30))
- PGBS: aggregate 94%, **p**artially ground **g**ranulated **b**last-furnace **s**lag **5**%, activator **cem**ent **1**% (= PGBS5/CEM1)
- CEM: **cem**ent binder content **3**% of dry weight of aggregate (= CEM 3)

The binder content for granulated blast-furnace slag and partially ground granulated blast-furnace slag is indicated as a proportion of the mass of the aggregate/binder mixture, the moisture content in all the above formulae being 7%.

The factors emerging from the statistical analysis as being those that had a clear influence on stabilization were used to construct a rough classification of old road materials in this respect by dividing them into three groups: 1) poorly stabilizable (poor), 2) moderately stabilizable (moder) and 3) well stabilizable (well). The purpose of this was to enable a binder formula to be chosen for any material on the basis of a few straightforward determinations.

5.2.2 Revision of the stabilization classification

At the second stage in the research this classification was revised and validated by selecting nine materials for the definition of grain-size distribution, maximum dry density, optimum moisture content, humus content, humus classification and pH, on the basis of which the necessary indices were calculated for assignment to a stabilization class, i.e. grain sizes corresponding to sieving proportions of 40% and 50%, humus content of fines, modified humus classification and pH of the under 2 mm fraction. Other parameters calculated which had at least some impact on stabilization were the amounts of fines, sand and gravel present. The results of these determinations are presented in Appendix 1.

The binding agents considered were the same as in the first stage of the research, with the exception of the activator cement, which had to be changed for CEM II A 42.5R, as manufacture of the CEM I 52.5 type had been discontinued. CEM II 42.5R was selected as this is composed almost entirely of Portland clinker, i.e. it contains a minimum of

additives such as limestone. The granulated slag was from the same blast-furnace as in the first stage, but alterations in production methods meant that it had been produced in a new direct granulation plant. The binder formulae were the same as at the first stage, in addition to which some variations were introduced into the amounts and proportions of the materials. A further test was performed with ground granulated blast-furnace slag, for instance, by adopting two formulae with binder amounting to 3% of the dry weight of aggregate, one with the binder proportions the same as at the first stage and the other with them altered to 50% ground granulated blast-furnace slag and 50% activator cement. The granulated blast-furnace slag and partially ground granulated blast-furnace slag formulae are presented in Table 5.

Table 5. Binder formulae for the granulated blast-furnace slag (GBS) and partially ground granulated blast-furnace slag (PGBS) used to verify the stabilization classification.

Binder formula	GBS (%)	PGBS (%)	CEM (%)	Aggregate (%)
GBS10/CEM1	10	-	1	89
GBS10/CEM2	10	-	2	88
GBS5/CEM2	5	-	2	93
GBS7.5/CEM1.5	7.5	-	1.5	91
PGBS5/CEM1	-	5	1	94
PGBS10/CEM1	-	10	1	89
PGBS5/CEM2	-	5	2	93
PGBS7.5/CEM1	-	7.5	1	91.5

5.2.3 Extension of the range of binding agents

The third phase in the research was carried out in two parts. Firstly, an extensive set of uniaxial compression tests were performed to assess the binding efficiency of a variety of previously untested combinations of binding agents, and secondly, various new combinations were used to stabilize crushed rock aggregate, taking for reference purposes some of the cement-activated blast-furnace slag products used in the earlier binding formula experiments. This was done in order to obtain a basis for comparing the efficiencies of the new binding mixtures with those of the already established ones. The method chosen was again a series of uniaxial compression tests.

Seven binding agent components were used in these experiments, six of which were slag products from Raahe. The activator cement was of the CEM II A 42.5R grade throughout. The properties determined for the binding agent components are listed in Appendix 1.

The slag-derived binding agent components comprised granulated blast-furnace slag, partially ground granulated blast-furnace slag and ground granulated blast-furnace slag, of which the partially ground variety was prepared from the same granulated slag as was used as such in the experiments. The LD steel slag components were crushed LD steel slag (0-3 mm), partially ground LD steel slag and ground LD steel slag, the latter two

being derived from the first. All the products were prepared in the Rautaruukki research centre in Raahe. Although the pregrinding process scarcely rendered LD slag any finer than it had been in its crushed (0-3 mm) form, it was decided to go ahead with separate tests on this material as planned.

The following abbreviations will be used below for the binding agents and components:
- GBS is **g**ranulated **b**last-furnace **s**lag
- PGBS **p**artially ground **g**ranulated **b**last-furnace **s**lag
- GGBS **g**round **g**ranulated **b**last-furnace **s**lag, Blaine 400 m^2/kg
- LD1 crushed LD steel slag, 0-3 mm (= LD steel slag, or LD slag)
- LD2 partially ground crushed LD steel slag (= partially ground LD steel slag, or partially ground LD slag)
- LD3 ground crushed LD steel slag, Blaine ~ 150 m^2/kg (= ground LD steel slag, or ground LD slag)
- CEM Rapid **cem**ent CEM II A 42.5R (= cement)

The abbreviations are selected deliberately so as to differ in form between the blast-furnace and steel slag products, in order to emphasize the difference in their functions with respect to stabilization, the blast-furnace slag products serving as binding agents and the steel slag products as activators. It is true that the latter also has some binding properties, but the majority of the binding effect is clearly achieved by means of the former. When steel slag is used it has to be present in larger amounts than the blast-furnace slag in order to attain sufficient activation.

Paste tests were carried out on a total of 12 binding agent combinations and four binder components alone. The binding agents were divided into coarse and finely ground ones, the former being ones in which the grains of the components were clearly distinguishable by eye and the latter ones in which ≥85% of at least one of the components was of a grain size less than 0.063 mm. In practice, the coarse binder components comprised granulated blast-furnace slag, partially ground granulated blast-furnace slag, LD steel slag and partially ground LD steel slag, while the fine components were ground granulated blast-furnace slag, ground LD steel slag and cement.

A further basis for this classification was reactivity, which is to a considerable extent dependent on grain size. The finely ground materials had proportionally a much larger reactive surface area than the coarse ones, an effect which is particularly relevant in the case of slag-based binding agents, as blast-furnace slag products in particular react at their surfaces rather than dissolving to form a paste as does cement.

Test specimens were made of the coarse binding agents as such, at optimum moisture content and at a compaction of 95%, while the finely ground materials were tested by using them to stabilize pure crushed rock aggregate of grain size 0-4 mm (sample CR4) at a binder content of 3.0% of the dry weight of aggregate, at the optimum moisture content for the aggregate and at a compaction of 95%.

The combinations of binding materials used in the paste tests are always expressed with the proportion of the blast-furnace slag component first (Table 6). Thus the designation GBS-LD1 (70/30) denotes a binder comprising 70% granulated blast-furnace slag and 30% crushed LD steel slag by weight. In the case of a binder composed of LD steel slag and cement, the proportion of slag is always shown first. Where a single binding agent is used, its abbreviation appears alone, e.g. GBS, denoting solely

granulated blast-furnace slag. The proportions of binder components are indicated in percentages, e.g. 90/10, 70/30, 50/50, 30/70 or 10/90.

Table 6. Coarse and finely ground binders used in the paste tests.

Coarse binding agents (Specimens composed of binder alone)	
GBS	granulated blast-furnace slag
GBS-LD1	mixture of granulated blast-furnace slag and crushed LD steel slag
GBS-LD2	mixture of granulated blast-furnace slag and partially ground LD steel slag
PGBS	partially ground granulated blast-furnace slag
PGBS-LD1	mixture of partially ground granulated blast-furnace slag and crushed LD steel slag
PGBS-LD2	mixture of partially ground granulated blast-furnace slag and partially ground LD steel slag
LD1	crushed LD steel slag
LD2	partially ground LD steel slag

Finely ground binding agents (binder content 3.0% of dry weight of crushed aggregate to be stabilized)	
GBS-LD3	mixture of granulated blast-furnace slag and ground LD steel slag
PGBS-LD3	mixture of partially ground granulated blast-furnace slag and ground LD steel slag
LD1-CEM	mixture of crushed LD steel slag and cement
LD2-CEM	mixture of partially ground LD steel slag and cement
LD3-CEM	mixture of ground LD steel slag and cement
GGBS-LD1	mixture of ground granulated blast-furnace slag and crushed LD steel slag
GGBS-LD2	mixture of ground granulated blast-furnace slag and partially ground LD steel slag
GGBS-LD3	mixture of ground granulated blast-furnace slag and ground LD steel slag

The finely ground binding agents were used to stabilize crushed rock aggregate, a product of grain size 0-4 mm (sample CR1) delivered in three batches in May 1996. Determinations performed on this material are presented in Appendix 1. The grain-size determinations and Proctor tests indicated that the three batches differed so little that they can be regarded as being of comparable composition as far as stabilization was concerned.

The maximum dry densities and optimum moisture contents of the binder components and binder combinations were determined by means of the improved Proctor test according to existing geotechnical laboratory instructions (Suomen geoteknillinen yhdistys 1985). The Proctor test was performed on all the coarse binding agent components (GBS, PGBS, LD1 and LD2) and with binder ratios of 30/70 and 70/30. Maximum dry densities and optimum moisture contents for the other binder ratios were calculated from the resulting figures on the assumption that these parameters vary in a linear manner with the binder ratio. The results of the Proctor tests for crushed rock aggregate were also applied to the stabilizations with ground materials.

Seven new binding agents were selected for comparison on the grounds of the paste tests and seven others that had been examined previously were used for reference

purposes. The aggregate used for the comparison was crushed rock of grain size 0-16 mm (sample CR2), Appendix 1.

In the case of the coarse binding agents, the formula indicates the binder component as a percentage of the binder-aggregate mixture by weight. The designation GBS10/CEM1, for instance, shows that the stabilization mixture contains 10 wt-% granulated blast-furnace slag, 1 wt-% cement and 89 wt-% aggregate. The following formulae were used when testing the coarse binding agents (those marked with an asterisk * were selected on the strength of the paste tests):

- GBS10/CEM1   * GBS10/LD3 1
- GBS 11   * GBS5.5/LD1 5.5
- PGBS5/CEM1   * LD1 11
- PGBS 5   * PGBS 3 / LD1 3

In the case of the finely ground binding agents, the figure appearing after the binding component denotes the concentration of binding agent as a percentage of the dry weight of the aggregate to be stabilized and the figures in parentheses the proportions of the components in the binder mixture. The finely ground binding agents were always used in an amount corresponding to 3.0% of the dry weight of the aggregate to be stabilized, as follows:

- GGBS 3   * LD3 3
- CEM 3   * GGBS-LD1 3(70/30)
- GGBS-CEM 3(70/30)   * GGBS-LD3 3(70/30)

The LD3 3 formula was chosen in order to assess the differences in stabilization efficacy between LD3 alone and cement alone.

Uniaxial compression strengths were determined at ages of 7, 28, 91 and 182 days for the binder pastes and at 7, 28 and 91 days in the binder comparisons.

5.2.4 Determination of dimensioning parameters

5.2.4.1 Division of the research

The research belonging to the fourth and last stage was divided into two parts: 1) laboratory experiments, and 2) test road structures and their monitoring. The materials chosen for the laboratory experiments were two crushed rock aggregates (samples CR4 and CR5), one crushed gravel (CG2) and one crushed till (CT1), Appendix 1. The aggregates were stabilized with three slag-based binders, taking cement as the reference.

Altogether five test structures were produced, using binding agents selected on the basis of preliminary experiments. Measurements and samples were taken in connection with the construction work for monitoring purposes, and the behaviour of the structures themselves was monitored from the time of their construction in 1997 onwards.

5.2.4.2 Laboratory test methods

The preliminary tests performed before the experiments proper at the fourth stage of the research employed the same aggregates for stabilization as the experiments themselves, for which purposes their maximum dry density, optimum moisture content, specific gravity, pH, bulk density, humus content and humus classification were determined. All four aggregates were well stabilizable. The slag components of the binding agents were all products of Rautaruukki's Raahe Steel Works, as at the earlier stages in the work. The aggregates to be stabilized and the binders used are set out in more detail in Appendix 1, and the binder formulae used in the preliminary tests in Tables 7 and 8.

The uniaxial compression strengths of samples in the preliminary tests were determined using a Geonor compressor at an age of 28 days. The compressions for the tests proper were applied to samples of age 28, 91, 182 and 273 days.

Table 7. Formulae for coarse binders used in the preliminary tests for determining the dimensioning parameters.

	Binder formula (mixture ratio in brackets)							
Sample	GBS10/ CEM1	GBS5/ CEM1.5	GBS5/ CEM1.75	GBS7/ CEM1.5	GBS5.5/ LD5.5 (50/50)	GBS5/ LD1 10 (~ 30/70)	GBS10/ LD1 10 (50/50)	GBS7.5/ LD1 10 (~ 40/60)
CT1	X	X	X		X	X	X	
CG2	X	X	X		X	X	X	
CR4	X	X		X	X	X		X
CR5	X	X		X	X	X		X

Table 8. Formulae for finely ground binders used in the preliminary tests for determining the dimensioning parameters.

	Binder formula						
	GGBS-CEM				CEM		
Sample	3 (70/30)	2 (70/30)	3 (50/50)	2 (50/50)	1.5	2.5	3.5
CT1	X	X	X		X	X	X
CG2	X	X	X		X	X	X
CR4	X	X		X	X	X	X
CR5	X	X		X	X	X	X

The binding agent formulae selected for the experiments proper on the strength of the preliminary tests are set out in Tables 9 and 10.

Table 9. Formulae for coarse binders used for determining the dimensioning parameters in the actual experiments.

	Binder formula (mixture ratio in brackets)				
Sample	GBS10/ CEM1	GBS7/ CEM1.75	GBS7/ CEM1.5	GBS10/ LD1 15 ($\sim$ 30/70)	GBS10/ LD1 10 (50/50)
CT1	X			X	
CG2		X		X	
CR4			X		X
CR5			X		X

Table 10. Formulae for finely ground binders used for determining the dimensioning parameters in the actual experiments.

	Binder formula			
	GGBS-CEM		CEM	
Sample	2.5 (70/30)	2 (70/30)	2.5	2
CT1	X		X	
CG2	X		X	
CR4		X		X
CR5		X		X

5.2.4.3 Methods for studying and monitoring the test structures

The site chosen for the five stabilization experiments was main road 63 between Ylivieska and Sievi. The stabilization and provision of an asphalt concrete AC 14/40 layer took place on 22.7.1997 and the actual surfacing with AC 18/80 the following week. Stabilization had been decided upon in order to improve the bearing capacity of the road, the original material had become finer and more susceptible to frost, so that the previous surfacing had sustained serious damage. There had also been bearing capacity problems at the site. Stabilization involved mixing of the material *in situ*, employing a mixture of the old surfacing and base course as the material to be stabilized. The width of the stabilized structure was 8.5 m. The binding agent in two of the test stretches was granulated blast-furnace slag activated with LD steel slag, while in the other three experiments it was cement-activated granulated blast-furnace slag, cement-activated ground granulated blast-furnace slag and cement. The granulated blast-furnace slag and LD steel slag were applied with a spreader specially developed for this purpose, and the ground granulated blast-furnace slag and cement were mixed in a batch mixing plant and applied with a cement spreader. Data on the structures are provided in Table 11.

Table 11. Stabilized structures produced at the experimental site.

Exp. structure, Pole space	Binder components	Amount of binder (mixture ratio proportion in brackets)		Planned depth of stabilization (mm)
		(%)	(kg/m^2)	
Trial 1 = T1 ps. 7012-7112	Granulated blast-furnace slag LD steel slag	4.5 (30) 10 (70)	60 28	250
Trial 2 = T2 ps. 7112-7212	Granulated blast-furnace slag LD steel slag	4.5 (30) 10 (70)	50 22	200
Trial 3 = T3 ps. 7212-7312	Granulated blast-furnace slag Cement (CEM II B 42.5)	4.5 1.2	22 6	200
Trial 4 = T4 ps. 7312-7412	Cement (CEM II B 42.5)	2.0	10	200
Trial 5 = T5 ps. 7412-7512	Ground Granulated bf-slag Cement (CEM II A 42.5 R)	2.0 (70/30)	10	200

Old surfacing is tarmac (in 1988); new surfacing AC 14/40 + AC 18/80.
Cross section 8.5/7.5; Mean daily traffic = 1473 (in 1994).

Groundwater pipes and frost gauges were installed at stakes 7062 and 7260, so that it was possible to check in the spring whether there was still any frost in the ground.

Samples for preliminary tests were taken from the approx. 200 mm thick base course at stakes 7000 and 7150 on 24.10.1996. The fact that stake 7000 eventually lay outside the test scheme is due to the fact that the sites had to be moved a few dozen metres at a later stage on account of the junction arrangements. Samples of the surfacing (tarmac) were also taken at the same time, and mixed with base course aggregate in a ratio of 25% tarmac / 75% aggregate for the production of test specimens. The binding agents and their proportions in these specimens, which were tested at ages of 28, 91 and 182 days, were as follows:

GBS / CEM*	GBS / LD1	CEM**	GGBS / CEM*
4.5 / 1.0	5 / 10	3	3 (70/30)
4.5 / 1.5	5 / 15	1.5	3 (50/50)
4.5 / 2.0	7.5 / 10		1.5 (70/30)
* CEM II A 42.5R		** CEM II A 42.5	1.5 (70/30)

Density and moisture content were measured with a Troxler device in the centre of each carriageway and down the central seam at distances of 10 m, 30 m, 50 m, 70 m and 90 m from the beginning of each test stretch during the construction work (i.e. at 15 measurement points per stretch). These measurements were made immediately after the last compaction run.

At the same time, samples of the stabilized material were taken from the centre of each carriageway and from the central seam at distances of 30 m, 50 m and 70 m from the beginning of each stretch for further laboratory tests (i.e. 9 samples per test structure) and corresponding samples of the pre-mixed material from both carriageways (6 samples per test structure).

The following processes and tests were applied to the *pre-mixed* material:
- The samples from each carriageway of each structure were combined and each composite sample was used to produce a specimen (10 specimens in all).
- The grain-size distribution of each specimen was determined by wet sieving.
- A compression test sample was prepared from each specimen employing the same stabilization formula as for the test structure in question. Tests were performed on the samples at ages of 7, 28, 91, 182 and 273 days.

  The following processes and tests were applied to the *stabilized* material:
- The samples from each carriageway of each structure were combined, and similarly the samples from the central seam, and each composite sample was used to produce a specimen (15 specimens in all).
- A compression test sample was prepared from each specimen, employing the same stabilization formula as for the test structure in question. Tests were performed on the samples at ages of 7, 28, 91, 182 and 273 days.
- The combined specimens for the two carriageways were further combined to obtain one specimen for each test structure, and samples were prepared from these for dynamic triaxial tests at ages of 28 and 182 days.

The test structures themselves were monitored by means of falling weight tests, damage surveys and frost heave levellings, and samples for further laboratory tests were taken from the structural layers in autumn 1999 through four holes with a vehicle-mounted borer. The thickness of each layer was also assessed from the same holes.

5.3 Methods

5.3.1 Laboratory tests

Determination of grain size

The grain-size distributions of the coarse materials were determined by wet sieving, in accordance with the current geotechnical laboratory instructions (Suomen Geoteknillinen yhdistys 1985). The grain-size distribution of the finely ground materials was determined using the Rautaruukki Company's Coulter LS230 particle size analyser, which works on a light scatter principle (Rautaruukki Ltd 1995, Mäkikyrö 1995).

Improved Proctor test

The maximum dry density (γ_{dmax}) and optimum moisture content (w_{opt}) of the <16 mm fractions of the samples were determined by the improved Proctor method according to the current geotechnical laboratory guidelines (Suomen Geoteknillinen yhdistys 1985).

Loss on ignition

Loss on ignition (humus content / organic matter content) was again determined according to the geotechnical laboratory instructions (Suomen Geoteknillinen yhdistys 1985). In the present case, however, a temperature of +600°C, applied for a period of

four hours, was preferred in order to minimize the amount of hygroscopic water lost (Rantala 1990, Soveri 1951), so that this would not need to be taken into account in the calculations.

NaOH test

The humus classification was determined for a 500 g sample of the <16 mm fraction of each specimen in accordance with the geotechnical laboratory guidelines (Suomen Geoteknillinen yhdistys 1985), which provide for a division into four classes on the basis of colour. A fifth class was added for the present purpose, however, to account for solutions that turned entirely black (= Modified Humus Class = MHC).

Density determination

Density was determined with an open pycnometer, according to existing civil engineering research and design instructions (Tie- ja vesirakennushallitus 1974).

pH determinations

The pH of the road construction materials was measured in suspensions of the oven-dried <16, <2 and <0.063 mm fractions in distilled water in a ratio of 1:2. The sample was allowed to stand overnight and the pH of the suspension was measured the following day with a Schott Cg 822 pH meter fitted with a flat-ended electrode.

Chemical analyses

The chemical analyses were performed by the laboratory services of the Rautaruukki Company using an XRF PW 2404 X-ray diffraction apparatus in accordance with the Rautaruukki Steel instruction handbook (1995). The CaO, MgO, Al_2O_3, SiO_2 and Fe_{tot} results for the slag binder components are quoted in Appendix 1, as it is easy from these alone to deduce what type of slag is concerned in each case.

Uniaxial compression test

The material for the specimens was prepared by adding the binding agent or mixture to dry aggregate with a mixer attached to a vertical drilling machine and then adding water and continuing agitation until the material was homogeneous. Each mixture was used to prepare two specimens for testing at different ages. This was done to ensure that any weighing errors would be detected at the testing stage at the latest, in the form of scatter in the results. The cylindrical specimens of height and width 100 mm were formed with an ICT revolving compactor, two specimens for each testing age.

The compaction target at the first stage was 21.5 kN/m^3 (dry density), and the moisture content of the stabilized material was regulated to 7%. The specimens were stored in plastic bags at a temperature of +22±2°C. The compression test was performed by increasing the loading by approximately 2.5 kN/s.

At the other stages in the research the specimens were prepared to the optimum moisture content for the aggregate as shown by the Proctor test and to a compaction of 95%. These specimens were stored at +22±2°C in a box over a water bath, Figure 19. The alteration in storage conditions was due to the observation at the first stage that the moisture retained in the plastic bag was not sufficient to support the binding reaction when storage continued for over four months. The uniaxial compression strength was

determined with a Geonor compressor at a speed of 2 mm/min, and was expressed initially in kilogrammes and converted to MPa units by reference to the known force exerted and the size of the surface perpendicular to that force.
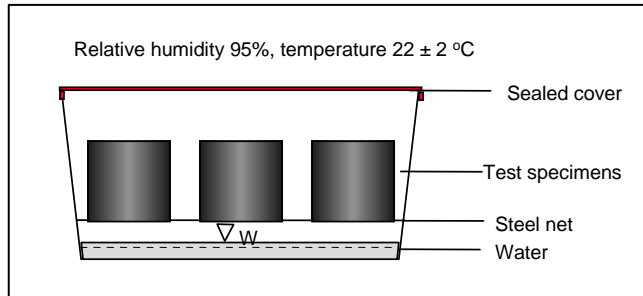

Fig. 19. Storage of specimens for the uniaxial compression test.

Dynamic triaxial test

The specimens for the dynamic triaxial test were prepared from samples taken from the stabilized materials from the experimental constructions and were cylindrical in shape, with a height of 200 mm and a diameter of 100 mm. They were formed in PVC tubes with a vibration compactor to the 95% Proctor density and optimum moisture content defined for the aggregate concerned, the mould being removed just before commencement of testing with a servo-controlled hydraulic piston.

In this case loading took the form of dynamic blows, giving rise to a state of stress that resembles traffic loading better than does static loading. The specimen is held for the duration of the test under a regulated pressure in a triaxial cell, this cell pressure being varied so that each specimen was tested under a range of five pressures (10, 20, 40, 70 and 140 kPa), with 3 or 4 steps in the dynamic loading at each cell pressure. The highest dynamic loading was restricted to a level at which the specimen would not undergo any plastic deformation. Each step in the dynamic loading comprised 300...400 sine-wave loading pulses. The loading was measured with a strain-gauge transducer, vertical axial deflection of the specimen with a LVDT transducer attached to the loading piston and horizontal deflection at the centre of the specimen with three inductive sensors attached to the cell. Pore pressure at the upper and lower surfaces of the specimen was measured with diaphragm pressure gauges. The dynamic triaxial apparatus is depicted in Figure 20.
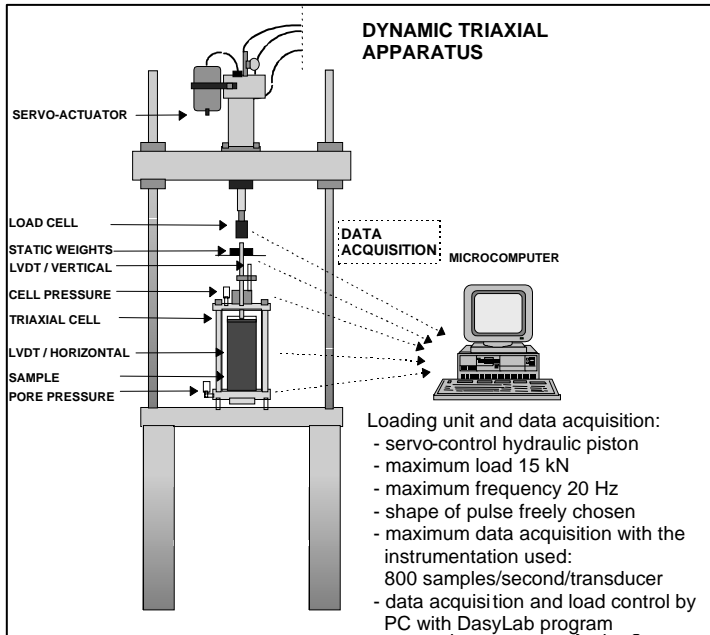
Fig. 20. Triaxial apparatus used to determine the dynamic modulus of elasticity.

The term resilient modulus is used here to refer to the 'dynamic modulus of elasticity'. The fitting function used in the sum of main stresses - dynamic modulus of elasticity expression to describe the dependence on the stress state is a power function obtained by means of a regression model. Two parallel tests were performed at each specimen age, the results of which are presented as a combined series in the diagrams. A fitting function was then constructed for the combined series in the form of an exponential curve by means of a regression model. The diagrams also contain indications of the equations for the curves and their degrees of significance.

5.3.2 Field tests

Measurements of density and moisture content

Density and moisture content were measured with a radioactive radiation-based Troxler device (Troxler 1981) at points located in the centre of each carriageway and down the central seam of the road at distances of 10 m, 30 m, 50 m, 70 m and 90 m from the beginning of the test stretch (i.e. at 15 points per test structure). The measurements took place immediately after the last compaction rolling.

Falling weight deflectometry

Bearing capacities were measured with a KUAB falling weight deflectometer (FWD) belonging to the Oulu Depot of the National Roads Administration (Spoof & Petäjä 2000). The measurements of bearing capacity were made in both carriageways at intervals of 20 m, starting 10 m from the beginning of the test structure, and thus the results presented here, in the form of averages for a whole test structure, usually represent means of 10 readings.

The readings obtained here with the National Road Administration's KUAB falling weight deflectometer may be corrected to a reference temperature of 20°C by means of the following formulae (Ruotoistenmäki & Spoof 1999):

$$k\_d0 \ = 1 - (AC\_T - 20) \times (0.0000975 \times AC\_th) \tag{7}$$
$$k\_d20 = 1 - (AC\_T - 20) \times (0.0000598 \times AC\_th) \tag{8}$$
$$k\_d45 = 1 - (AC\_T - 20) \times (0.0000476 \times AC\_th) \tag{9}$$
$$k\_d60 = 1 - (AC\_T - 20) \times (0.0000408 \times AC\_th) \tag{10}$$

where k_dx is   the correction coefficient for a deflection d at distance x
           AC_T the temperature of the pavement in ^{o}C
           AC_th the thickness of the pavement in mm

The above equations are based on the following assumptions (Ruotoistenmäki & Spoof 1999):

-   There is a clear linear dependence of falling weight deflectometer readings on pavement temperature.
-   The effect of pavement temperature on the results is still detectable 600 mm away from the loading plate, but no correction is required at greater distances (900 and 1200 mm in their measurements).
-   The magnitude of the correction is explained by the thickness of the pavement and the level at which the deflection is measured (rigidity of the whole structure).
-   The mean temperature of the pavement can be calculated from its surface temperature.

Frost heave levelling

Frost heave was assessed by means of normal altitude levellings performed in spring, when the ground frost is at its deepest, and in summer when any frost heave has evened out and the road structure has returned to its normal contours. The levellings were performed along the centre line of the road, down the middle of each carriageway and about 0.5 m from the edge of the pavement.

Damage surveys

Damage to the test structures was surveyed by walking the length of each and recording and measuring all instances of damage to the pavement and edges of the road that could be observed. The observations were recorded on drawings of the road made on graph paper and contained details of the type and extent of the damage in each case. Cracks were drawn as they were visible on the road itself, and their widths recorded in figures next to the lines. Observations were also made on the efficiency of drainage and on any cracks or other signs of damage to the embankments or slopes. The surveys were made in the spring, when frost heave was at its greatest, as all forms of damage are most

conspicuous at that time. A similar damage survey method was later adopted in the Finnish national TPPT research programme (Onninen 2002).

Vehicle-mounted drilling

Holes were drilled in both carriageways of the road structures by means of a borer mounted on the back of a heavy transport vehicle, beginning at stake 7040 and ending at stake 7480. This meant that two holes were bored in each carriageway of each test structure.

The holes were used for measurement of the thicknesses of the structural layers and for sampling of the layers for grain-size determination. The grain-size distributions were assessed by washing and sieving and areometrically, in accordance with the geotechnical laboratory guidelines (Suomen Geoteknillinen yhdistys 1985). The results obtained from the drillholes were incorporated into longitudinal cross-sections of the test structures, from which the thicknesses of the layers could be defined for back-calculation purposes, on the assumption of linear changes between drilling points. A similar method was later adopted in the Finnish national TPPT research programme (Korkiala-Tanttu & Onninen 2001).

5.3.3 Back-calculations

The KUAB deflection readings obtained here were corrected for temperature using the National Road Administration's Karhu-97 program, which operates according to Equations 7-10 above.

The principle of the back-calculation procedure is that moduli should be found for the various structural layers that provide a theoretical deflection basin that corresponds as closely as possible to that measured with the falling weight deflectometer. This can be applied equally well to slag-stabilized layers. The work was done using the PAS program devised by P. Ullidtz, in which the modules for the layers are adjusted manually and the correspondence between the resulting profile and the measured basin can be followed simultaneously in graphical form. When a sufficient correspondence has been attained (0...3% in practice) the moduli concerned are accepted and saved. (Liimatta 2000)

The stages in the back-calculation procedure when performed using the PAS program are thus:
- specify number of geophones and their distances from the centre of loading
- enter loading data
- enter deflection readings
- enter number of layers and their thicknesses
- enter Poisson ratios
- iterate to obtain moduli
- save results

The PAS program is based on elasticity theory and is subject to the following initial assumptions:

- that the layers are linearly elastic
- that the construction materials are isotropic and homogeneous
- that the layers are of infinite extent on a horizontal plane

The structural layers and subgrade of a road can be assessed in terms of surface moduli calculated from the deflections, where each modulus represents the E-modulus of the structure at a depth equivalent to the distance from the centre of loading at which it was measured. In the case of a homogeneous, linearly elastic road structure, this equivalent depth corresponds directly to the structural depth and the moduli defined on the basis of the deflections measured at different distances from the centre of loading are at the same time surface moduli (the E-modulus remains constant in a vertical direction). In normal structures, where the rigidity of the layers increases from bottom to top, their equivalent thicknesses will be greater than their actual thicknesses, while the surface modulus calculated for the deflection at the centre of the loading plate at a Poisson ratio of 0.5 will correspond to the bearing capacity. The following equations are used to calculate the surface moduli (Liimatta 2000):

$$E_0 = \frac{2 \cdot \left(1 - \mu^2\right) \cdot \sigma_0 \cdot a}{d_0} \tag{11}$$

$$E_r = \frac{\left(1 - \mu^2\right) \cdot \sigma_0 \cdot a^2}{r \cdot d_r} \tag{12}$$

where
- E_0 is the surface modulus at the centre of the loading plate, in MPa
- E_r the surface modulus at distance r from the centre of the loading plate, in MPa
- d_0 deflection at the centre of the plate, in mm
- d_r deflection at distance r from the centre of the plate, in mm
- a radius of the loading plate, in mm
- σ_0 impact pressure, in MPa
- r geophone distance, in mm
- μ Poisson's ratio

The dimensioning of slag stabilization in accordance with the expected loading can be accomplished by the traditional analytical, bearing capacity procedure, as for the corresponding unbound layers, the target bearing capacity being determined by the traffic loading and the class of road concerned. Bearing capacity can be calculated from the E-moduli of the subgrade and structural layer materials by means of Odemark's (1949) two-layer formula (Liimatta 2000). Thus the initial data required for dimensioning purposes are the bearing capacity (E-modulus) of the subgrade, the modulus and thickness of the material to be used and the target bearing capacity.

In the case of improvements to an existing road, the bearing capacity of the subgrade must be calculated from the falling weight deflection basin. Since when stabilizing an old road structure, the subgrade will shift downwards by an amount equivalent to the thickness of the stabilized layer, the modulus used for the substrate will have to be the surface modulus calculated from the deflection D450, that is:

$$E_{substrate} = \frac{\left(1 - \mu^2\right) \cdot \sigma_0 \cdot a^2}{450 \cdot d_r} \tag{13}$$

where $E_{substrate}$ is   the modulus for the subgrade at the centre point of the loading plate, in MPa ($=MN/m^2$)

d_r deflection at 450 mm from the centre of the plate, in mm

a   radius of the loading plate, in mm ($=150$ mm)

σ_0 impact pressure, in MPa ($=MN/m^2$)

μ Poisson's ratio ($=0.35$)

The thickness of the stabilized layer in these dimensioning calculations is determined from the modulus of the substrate and the target bearing capacity, taking the bearing capacity of the substrate to be that of the old road structure remaining below the stabilized layer. The curves indicate that the thickness of the layer to which slag stabilization is to be applied will vary in the range 150...300 mm.

6 Results and discussions

6.1 Research methods

It is important when carrying out laboratory tests on industrial by-products that the researcher should be fully acquainted with the material concerned, and in the case of slag products two facts have to be borne in mind that can influence the results, and above all their interpretation: the slow process of hardening and the volume expansion of steel slag. It is important to be aware of the slow hardening of slags, because conclusions reached on the basis of excessively short binding times (e.g. 7 days) usually prove erroneous. It is reasonable to analyse the results achieved with powdered binders only after an interval of at least 28 days, and those achieved with coarse-grained slag binders only after 91 days. Also, the expansion of steelmaking slags has to be taken into account when storing the samples, e.g. by preparing the sample in a protective tube, which will exert the necessary lateral loading to prevent cracking, and removing it from this tube just before testing.

Two principal modes of performing the uniaxial compression test have been proposed: with a constant compression speed or with a constant loading increment. A test run here were designed to assess at the same time the effect of compression speed on the results, by testing two specimens with the Geonor compressor at a fixed speed of 2 mm/min and another two at a fixed loading of approx. 2.5 kN/s. The correspondences between the resulting compression strengths are shown in Figure 21. The regression equation points to a highly significant correspondence, and the regression line is practically straight, although with a certain amount of scatter at the higher strength values.

The size and shape of the specimen also influenced the strengths recorded, as seen in Figure 22. With the exception of one binding agent, the shorter cylindrical specimens of diameter 100 mm exhibited higher compression strengths than the taller ones in the stabilization of both pure aggregate and aggregate mixed with bitumen.
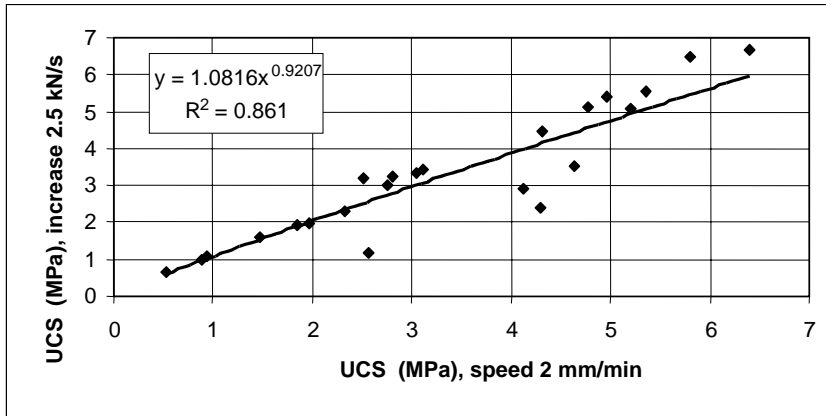
Fig. 21. Correspondence between compression strengths obtained for the specimens at ages of 7, 28 and 56 by two methods (loading speed 2 mm/min and fixed loading 2.5 kN/s).
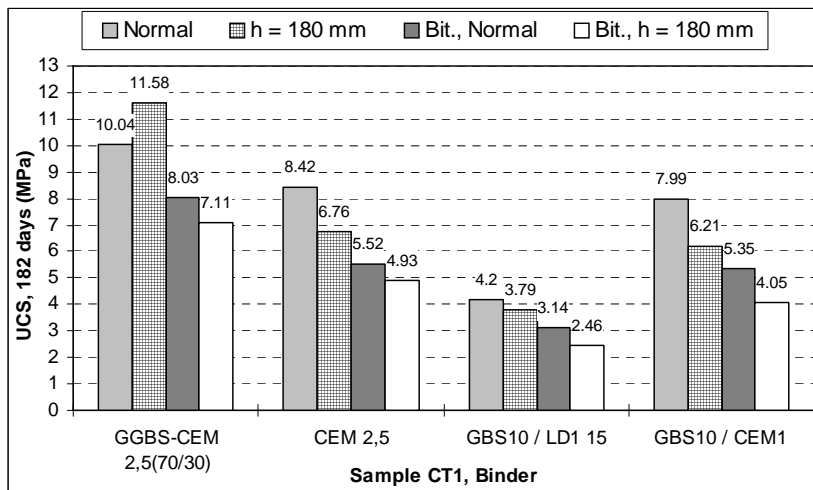
Fig. 22. Effect of height of the specimen on uniaxial compression strength (Normal = specimen height 100 mm, Bit = aggregate mixed with bitumen from the old surfacing).

The results presented in Figure 22 are greatly affected by the mode of failure of the specimen, Figure 23. In the case of the shorter specimens the fracture reaches the surface of the upper plate, whereupon the resistance to failure increases (case a). When there is a large amount of binding agent present, e.g. 10% granulated blast-furnace slag, and the strength is nevertheless poor, failure takes place in the manner shown in case b, the specimen bulges outwards slightly and the most prominent fracture sets out from its sides and rises to a point at its upper surface. A large number of smaller vertical cracks also

appear. When the specimen is sufficiently tall, cracking is able to take place freely and a distinct angle of fracture can be defined for the specimen (case c).
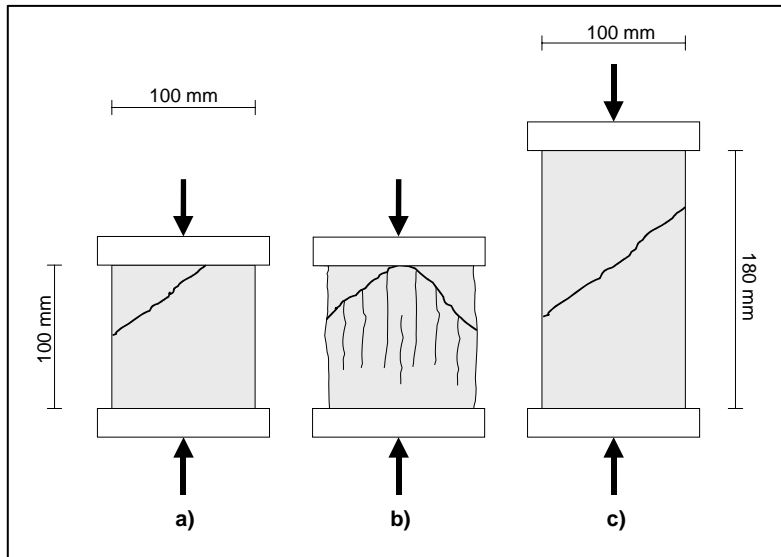

Fig. 23. Modes of specimen failure in the uniaxial compression test.

Correction factors have been proposed for rendering specimens of different sizes comparable in the compression test, but the use of these often proves a somewhat dubious matter. It has been shown by Ylipiessa (1998c) that the compression strength of specimens decreases with an increasing ratio of height to diameter, the necessary correction factors deviating markedly from those quoted in the Instructions for Cement Stabilization (Tielaitos & Tiehallitus 1992). One can only conclude that rather than using correction coefficients, it would be more practicable to eliminate this effect by recommending a constant h/d ratio for specimens in uniaxial compression tests (e.g. 1.0).

In order to obtain better assessments of compression strength, it has also been proposed that freeze-thaw behaviour should be incorporated into the test. This is indeed quite justifiable, as the stabilized material is mostly located in the surface parts of the road structure and is therefore exposed to repeated freezing and thawing. The strength of the specimen after freeze-thaw stressing should reflect the nature of the stabilized material better than does a specimen tested directly after storage. The significance of freeze-thawing stress for the strength of stabilized materials was observed very clearly in the present series of tests (Figure 24), to the extent that not even the mixing of bitumen from the old surfacing course into the aggregate was able to improve their resilience. It was also noted that cement behaved slightly differently from the slag-based binding agents in this series of tests, in that where stabilized crushed rock alone lost a significant amount of its compression strength following cyclic freeze-thaw treatment, the presence of old bitumen in the mixture improved its durability considerably.

The findings of Ylipiessa (1998b) suggest that there is no optimal compression

strength that will guarantee resistance to freezing and thawing, but rather a cement content of 3-5% would seem to be sufficient for this purpose in the case of most materials. It may similarly be concluded from the investigations by Wong (1992) that erodability decreases significantly with the amount of binder, although only up to a certain percentage. Cement provides a good erodability result with as little as 1% binder, but no further significant improvement is observed even with five times that amount. Materials stabilized with slags have high erodability at a low binder content, and the situation improves with increasing amounts of binder, up to about 4%, but becomes less significant beyond that point. It appears that a higher percentage of slag additives would have to be used to achieve the same erodability performance with cement-stabilized materials.
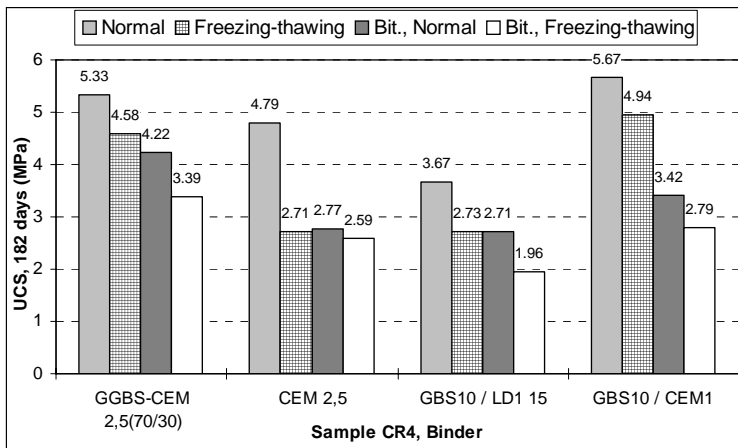

Fig. 24. Effects of freeze-thaw cycles on uniaxial compression strength (Bit = aggregate mixed with bitumen from the surfacing). The test was performed largely according to the German protocol TP HGT-StB 86 (1986).

When studying the effects of curing time and stress state on the deformation characteristics of slag used as a base course material, Yoshida *et al.* (1998) performed repeated loading triaxial compression tests on a compound slag comprising a mixture of steel slag, blast-furnace slag and granulated slag in the ratio 5:3:2 by weight. They arrived at the following observations:
1. The resilient deformation modulus of compound slag increases with the mean principal stress and decreases with the deviatoric stress.
2. The resilient deformation modulus increases with curing time due to the development of cementation inherent in slag material.
3. The permanent axial strain increases with the number of load applications, rapidly at the beginning and then gradually at numbers of load applications greater than 2000.

6.2 Factors affecting stabilization

One outcome of the first phase of the research was the observation that the stabilization achieved with the cement-activated ground granulated blast-furnace slag (GGBS) and granulated blast-furnace slag (GBS) was the sum of a great number of contributory factors (Mäkikyrö 1995) and could not be explained by any one parameter alone. The most prominent of these factors in the case of both slag-based binding agents were nevertheless the humus content of the fines, humus classification, pH of the <2 mm fraction of the whole body of material to be stabilized and the grain sizes corresponding to sieving proportions of 40% and 50%. Further significant factors when using ground granulated blast-furnace slag were the pH of the fines and the grain size corresponding to a sieving proportion of 30%. A low fines content and the proportions of the sand and gravel fractions also had some effect on the result.

It was quite possible to find clay minerals in samples on the basis of the mineralogical examinations, but only in small quantities, and it was concluded that scarcely any secondary minerals produced by chemical weathering were to be found in the basic aggregate used in road structures but that the fines present were predominantly derived from mechanical grinding of the rock material (Hiltunen 1994). This finding is of significance for the use of ground binding agents for stabilization purposes, because in this case the number of components participating in the active binding reactions is smaller and binding is likely to be more reliable. It was concluded from Hiltunen's findings that it would be unnecessary in the future to determine the mineralogy of the material.

Other factors that had a clear influence on the stabilization strength in addition to the above were the calcium and magnesium exchange capacities of the fines and the cation exchange capacities of univalent metals, although it must be admitted that these are parameters that are rarely measured in practice when considering the design of road structures. It is nevertheless worthwhile performing such determinations when attempting to ascertain in greater depth what is the nature of the interaction between binding agents and the material to be stabilized. It was eventually decided that these factors would not be pursued further in the present context, however.

It was also found that the quantity of partially ground granulated blast-furnace slag required for a given stabilization strength was a half of the quantity of granulated blast-furnace slag required, on account of the higher reactivity of this finer-grained material.

The results were used to construct a rough classification of road building materials in terms of the compression strength attainable in their stabilization (Table 12). The boundary values employed in this classification with respect to the properties of the various binding agents were selected so as to ensure that those emerging in the "well stabilizable" class had a sufficient compression strength to meet all the requirements likely to be placed on a stabilized material. The "moderately stabilizable" class, on the other hand, covers a wide range of compression strengths such that even the lowest levels subsumed in it are sufficient to yield adequate strength with a basic stabilization formula. The "poorly stabilizable" class contains formulae that are incapable of providing the necessary strength, even with time, unless altered radically.

The traditional class IV in the improved humus classification was divided here into

two parts, solutions that were obviously black in colour being assigned to a new class V. It should be noted that extreme values for the indices and contributory factors in this classification cannot be extended indefinitely, e.g. the grain size corresponding to a sieving proportion of 40% cannot increase to an infinite degree.

Table 12. Strength classification used in defining the binding formulae (Mäkikyrö 1995).

Contributory factor	Boundary values		
	Well stabilizable	Moderately stabilizable	Poorly stabilizable
d_{40} (mm)	> 1.0	0.5 – 1.0	< 0.5
d_{50} (mm)	> 1.75	0.75 – 1.75	< 0.75
Humus content of fines (%)	< 1.8	1.8 – 3.0	> 3.0
Improved humus classification	0 – IV	III – V	V
pH of <2 mm fraction	> 7.25	6.25 – 7.25	< 6.25
Poorly stabilizable: GBS10/CEM1: 91 d < 0.7 MPa, GGBS-CEM 3(70/30): 28 d < 0.4 MPa			
Moder. stabilable: GBS10/CEM1: 91 d 0.7 – 2.8 MPa, GGBS-CEM 3(70/30): 28 d 0.4 – 3.2 MPa			
Well stabilizable: GBS10/CEM1: 91 d > 2.8 MPa, GGBS-CEM 3(70/30): 28 d > 3.2 MPa			

6.3 Verification of the stabilization classification

As a consequence of observations made in the first phase of the research, the compression strengths were examined by comparing the results for the specimens stabilized with cement-activated granulated blast-furnace slag and partially ground granulated blast-furnace slag at an age of 91 days and those stabilized with cement-activated ground granulated blast-furnace slag at 28 days. The eventual strength of a material stabilized with granulated blast-furnace slag can be assessed only at 91 days on account of the slowness of hardening, whereas a period of 28 days is adequate with ground granulated blast-furnace slag.

Compression strengths varied greatly depending on the amount of binding agent used, the proportions in which the mixture was made and the material to be stabilized, and considerable variability was found even between specimens representing the same formula. This is a clear indication that marked variation can exist within old road material with regard to the potential for stabilization. Two of the specimens, OM4 and OM5, could not be stabilized successfully with any of the formulae available here, and thus evidently represent materials for which slag-based stabilization is not an economically viable alternative.

The highest compression strengths for cement-activated granulated blast-furnace slag at an age of 91 days were obtained with 10 wt-% slag and 2 wt-% cement (GBS10/CEM2), Figure 25, while the second highest strengths on average were achieved with the formula GBS5/CEM2 and almost as good results with GBS7.5/CEM1.5. By far the poorest compression strength was that recorded for GBS10/CEM1. The influence of the quantity of cement present is revealed very clearly by a comparison of the GBS10/CEM1 and GBS10/CEM2 results, where the strength was significantly greater in

all the specimens that contained the larger amount of cement, the difference being a six-fold one in the case of OM8.

Alterations in the quantity of granulated blast-furnace slag did not have such a marked impact on strength at 91 days as did those in the quantity of cement. An increase from 5% to 10%, for instance (formulae GBS5/CEM2 and GBS10/CEM2), improved the strength by a factor of 1.24 on average, without any very substantial variation between specimens, the only noticeable deviations being in specimens OM3, where strength declined slightly, and OM8, where the figure was almost doubled.
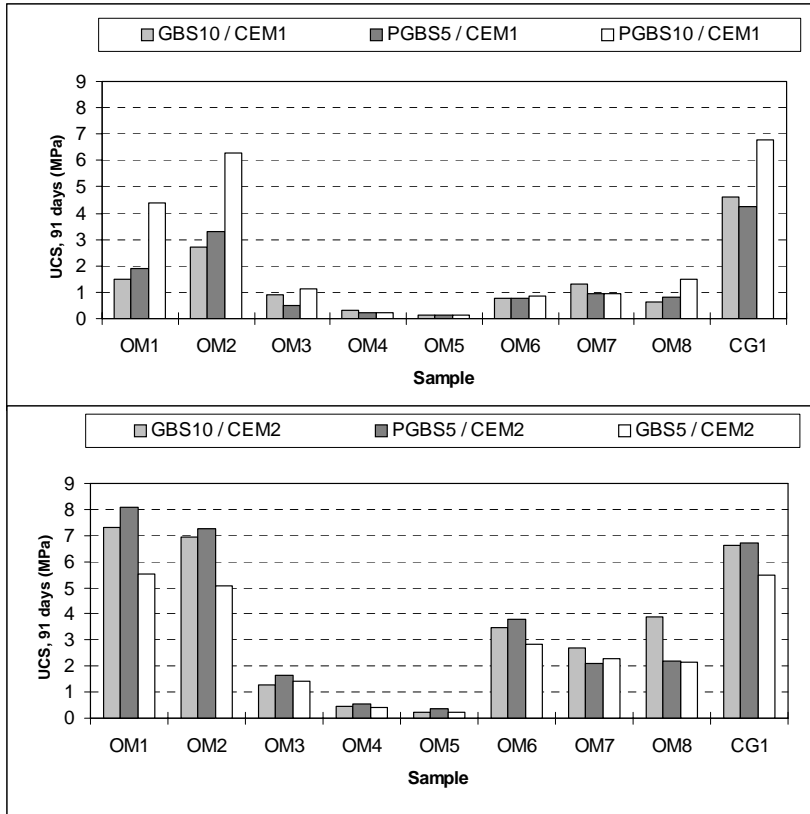

Fig. 25. Compression strengths of specimens stabilized with cement-activated granulated blast-furnace slag and partially ground granulated blast-furnace slag at the age of 91 days.

When the quantity of granulated blast-furnace slag was increased by 2.5 percentage points and that of cement reduced by 0.5%-point (formulae GBS5/CEM2 and GBS7.5/CEM1.5) the result was improved strength in three specimens and reduced strength in four, while two showed the same strength with both formulae, but when further steps were introduced in the same directions all the specimens except for OM7 gave a lower strength. Thus the amount of cement contained in the formula GBS10/CEM1 may be said to be inadequate to activate the reaction with granulated

blast-furnace slag to the same extent as that contained in the GBS7.5/CEM1.5 formula. The greater quantity of slag is of no avail in this case, as it merely slows down stabilization or in practice causes it to cease altogether. When considering these proportions, of course, we are operating on the margins of practicability of the method, where even small changes can have a major effect on the outcome.

The highest compression strengths with partially ground granulated blast-furnace slag were recorded with binding formulae that involved 5% of this material and 2% cement. As the material is more reactive than its unground equivalent and the formula allows for as much as 2% cement, it can be assumed that a substantial proportion of the eventual strength of the stabilized material would have been achieved by 91 days. The second highest strength values were those for the formula PGBS10/CEM1. The strengths recorded in these tests as a whole ranged from almost 7 MPa to only 0.12 MPa, i.e. virtually no stabilization whatsoever, the lowest figures being characteristic of the formulae PGBS7.5/CEM1 and PGBS5/CEM1. This binder failed to stabilize specimens OM4 and OM5 at all, as even the best formula, PGBS5/CEM2, gave values of only 0.54 and 0.36 MPa.

The significance of the activator cement for the proper functioning of the binding agent is again emphasized in the results for stabilization with partially ground granulated blast-furnace slag, for given 5% of the latter in the mixture, an increase in activator from 1% to 2% (formulae PGBS5/CEM1 and PGBS5/CEM2) led to a substantial improvement in strength. This trend was most clearly visible with specimens OM1 and OM6, where the increase was more than 4-fold, and least marked with specimen CG1, for which it was about 1.6-fold. Conversely, a doubling of the quantity of partially ground granulated blast-furnace slag present (from the formula PGBS5/CEM1 to PGBS10/CEM1) led to an increase in strength in five specimens and an unchanged situation in four. In other words, about a half of the specimens failed to show any improvement in strength even with a substantial increase in binder but required a higher input of activator before this could be achieved.

The best compression strengths with cement-activated ground blast-furnace slag at 28 days, almost 9 MPa, were obtained with a formula in which the binder content was determined in accordance with the Instructions for Cement Stabilization (Tielaitos & Tiehallitus 1992) and its composition was 70% ground granulated blast-furnace slag and 30% cement. These instructions presuppose measurement of the resulting strength at an age of 7 days, but this is obviously too early for a slag-based binding agent. Specimen OM5 had not stabilized in spite of containing almost 10% binding agent, whereas specimen OM4 had stabilized slightly even at a content of only 8.6% (Figure 26).

It is impossible to compare the present results with the compression strengths of the order of 5 MPa claimed in the instructions for 7 days, because of the difference in age at testing, but it should be noted that the strength discrepancies found in certain specimens may be due to use of the wrong proportions of activator to binding agent rather than the wrong amount of binding agent as such.

Comparison of the results obtained with 3% binding agent added to the aggregate but with the slag and cement in different proportions (50/50 vs. 70/30) suggests that slightly better strengths were obtained with the 50/50 mixture. At least with some of the specimens, this would appear to be due to the fact that the agent with the higher proportion of slag was unable to stabilize the specimen satisfactorily within the trial

period of 28 days, i.e. a low degree of binding was achieved. It is highly probable, however, that better binding would have been observed if these specimens had been examined at 91 days.

The specimens are classified according to the factors known to affect compression strength in Table 13 and according to their mean values and eventual categories in the stabilization classification when stabilized with cement-activated granulated blast-furnace slag and ground granulated blast-furnace slag in Table 14. The classification is performed on the basis of the results obtained in the first stage of the research (Mäkikyrö 1995).

The boundary compression strength values for the stabilization classes were achieved with the following binding agent formulae:

- granulated blast-furnace slag: GBS 10%, cement 1% and aggregate 89% of the total mixture by weight (GBS10/CEM1),
- ground granulated blast-furnace slag: binding agent content 3% of dry weight of aggregate, binding agent composition 70% ground granulated blast-furnace slag, 30% cement GGBS-CEM 3(70/30).
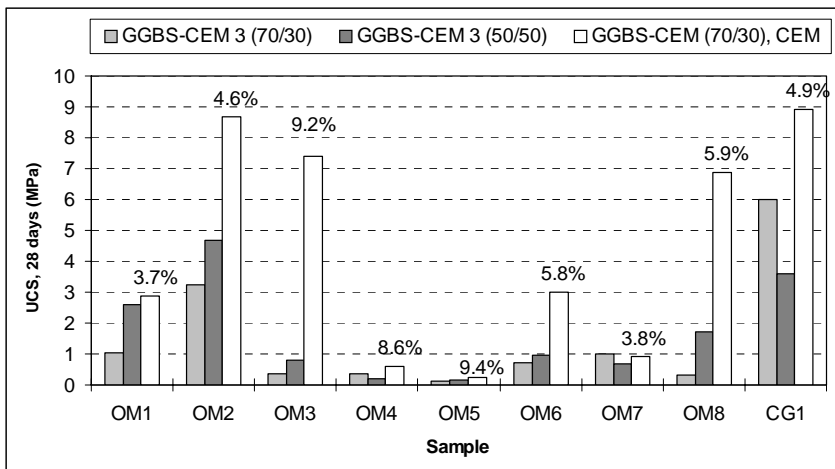

Fig. 26. Compression strengths of specimens stabilized with cement-activated ground granulated blast-furnace slag. The percentage above the white column denotes the amount of binding agent used, calculated according to the Instructions for Cement Stabilization (Tielaitos & Tiehallitus 1992). The binding agent was composed of 70% ground granulated blast-furnace slag and 30% cement.

Table 13. Stabilization classification of specimens in relation to each factor affecting compression strength.

| Specimen | Grain size | | Organic content | | |
	d_{40}	d_{50}	Humus content of fines	Improved humus classification	pH of <2.0 mm fraction
OM1	moder	moder	well	moder	poor
OM2	moder	moder	well	moder	moder
OM3	poor	poor	moder	poor	poor
OM4	poor	poor	poor	poor	poor
OM5	poor	poor	poor	poor	poor
OM6	poor	moder	moder	poor	poor
OM7	poor	moder	moder	moder	moder
OM8	moder	well	moder	poor	moder
CG1	well	well	well	well	poor

Poorly stabilizable: GBS10/CEM1: 91 d < 0.7 MPa, GGBS-CEM 3(70/30): 28 d < 0.4 MPa
Moder. stabil.: GBS10/CEM1: 91 d 0.7 – 2.8 MPa, GGBS-CEM 3(70/30): 28 d 0.4 – 3.2 MPa
Well stabilizable: GBS10/CEM1: 91 d > 2.8 MPa, GGBS-CEM 3(70/30): 28 d > 3.2 MPa

The effect of fines in detracting from compression strength is taken into account only in the case of specimen OM3, which contained less than 5% material of a grain size <0.063 mm, only 4.7% in fact. Taking all the grain size indices into consideration, specimens OM1 and OM2 are moderately stabilizable, OM3, OM4 and OM5 are poorly stabilizable and CG1 is well stabilizable. The remaining specimens are more complex, OM6 being moderately stabilizable in terms of d_{50} but poorly stabilizable in terms of d_{40}, OM7 is moderately stabilizable in all respects except d_{40}, for which it is poorly stabilizable, and OM8 is moderately stabilizable in all respects except for d_{50}, on which score it is well stabilizable.

Regarding pH, it should be noted that this can be affected by the aggregate as well as by the organic matter present. Only specimens OM4 and OM5 are poorly stabilizable with respect to all three organic matter factors, while OM7 is consistently moderately stabilizable. OM1, on the other hand, has a different classification for every factor, and all the others have one factor on which their classification differs. The eventual classification is performed by reference to a combination of numerous factors, but it is interesting to observe that the situation with regard to grain size factors is clearer than that prevailing in the case of organic matter factors. Predicted stabilization classes based on the mean performance of each specimen with respect to the whole range of factors are set out in Table 14. Here it may be seen, for example, that specimen OM6 could be "rounded off upwards" to fall into the moderately stabilizable class with respect to grain size but was poorly stabilizable in terms of organic matter content. Viewed overall, it is poorly stabilizable on three scores and moderately stabilizable on two, so that its overall predicted class is "poorly stabilizable".

Table 14. Mean stabilization classifications for specimens stabilized with cement-activated granulated blast-furnace slag at 91 days. The predictions for the grain-size classification take account of the influence of the sand and gravel fractions in borderline cases.

Specimen	Grain size, prediction	Organic matter, prediction	Overall predicted class	Realized class	
				GBS/CEM	GGBS/CEM
OM1	moder	moder	moder	moder	moder
OM2	moder	moder	moder	moder	well
OM3	poor	poor	poor	moder	poor
OM4	poor	poor	poor	poor	poor
OM5	poor	poor	poor	poor	poor
OM6	moder	poor	poor	moder	moder
OM7	moder	moder	moder	moder	moder
OM8	moder	moder	moder	poor	poor
CG1	well	moder	well	well	well

The stabilization predictions based on grain-size properties are realized in the tests in the case of seven of the specimens stabilized with cement-activated granulated blast-furnace slag and cement-activated ground granulated blast-furnace slag (Table 14), whereas the classification based on organic matter was realized for only five specimens and the overall classification for six.

The specimen CG1, which was predicted overall to stabilize well with either binding agent, did in fact do so, while two specimens predicted to stabilize poorly, OM3 and OM6, proved to do so to a moderate extent and one predicted to stabilize moderately well, OM8, stabilized poorly. The other sample predicted as moderately stabilizable, OM2, proved to stabilize well.

Specimen OM8, which was poorly stabilized by both binding agents in spite of a moderate stabilization classification, was clearly well adapted to stabilization in terms of its grain size, but the humus content of its fines was close to the borderline for poor stabilization and its improved humus classification was V, and it is these latter characteristics that were evidently responsible for the poor outcome. OM6, on the other hand, which was predicted to be poorly stabilizable but in fact stabilized moderately with both binding agents, was moderately stabilizable in terms of its grain-size parameters and very close to moderate in terms of organic content. OM3 also stabilized moderately with the cement-activated granulated blast-furnace slag binder, although its forecast was poor, but in this case it is difficult to perceive any reason for this unexpected result on the basis of the classification parameters. In the case of OM2, however, which stabilized well with cement-activated ground granulated blast-furnace slag even though only moderate success was predicted, the reason can be seen from the details of the classification to have been the satisfactory organic content parameters, which came close to the "well stabilizable" level.

6.4 Extension of the range of binding agents

Of the components of coarse binding agents, the best strength results were achieved with partially ground granulated blast-furnace slag (Figure 27) and the poorest with granulated blast-furnace slag (GBS), although by an age of 182 days this latter had achieved almost the same level of strength as partially ground LD steel slag (LD2). The strength achieved with LD steel slag (LD1) was virtually the same as that provided by its partially ground product (LD2) after 7, 28 and 91 days, but LD1 was distinctly superior by the age of six months in spite of being coarser in texture. Partially ground granulated blast-furnace slag (PGBS), LD steel slag and partially ground LD steel slag are practically identical in grain size, but the former gave a very much better compression strength, i.e. it is more reactive than the others. The differences in strength results between these coarse binding agent components even out with time, however, although the weakening recorded for the partially ground granulated blast-furnace slag at 182 days is almost certainly attributable to a test error.

The best compression strengths at 91 and 182 days among the specimens prepared from LD steel slag and granulated blast-furnace slag were those with a binding agent formula GBS-LD1 (50/50), while the results obtained with binding agent / activator proportions of 10/90, 30/70 and 70/30 were of the same order of magnitude. The results obtained with granulated blast-furnace slag alone were considerably poorer, however, as was also the case with LD steel slag alone, with the exception of one binding agent mixture, although the initial rate of stabilization was higher with the binders that contained large amounts of LD slag.
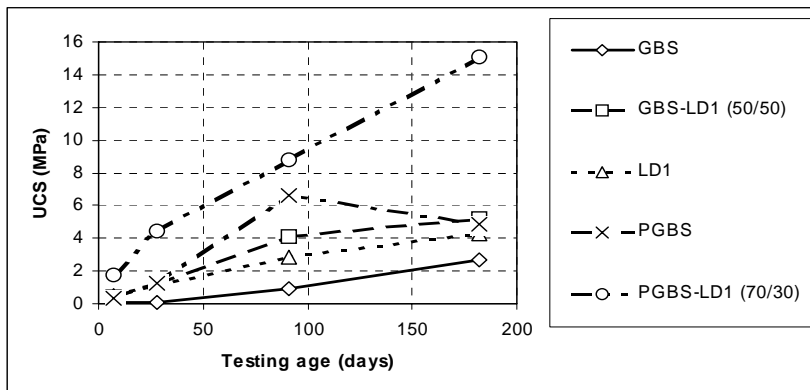

Fig. 27. Uniaxial compression strengths achieved with coarse-grained binding agents.

The binding agent mixtures GBS-LD1 (10/90), GBS-LD1 (30/70), PGBS-LD1 (10/90) and PGBS-LD1 (30/70) and LD1 alone featured cracks at the ages of 91 and 182 days that had presumably been caused by the volume instability of the LD steel slag, but these had become re-bound.

The binding agents composed of granulated blast-furnace slag and partially ground LD steel slag (GBS-LD2) did not yield as good stabilization results as the GBS-LD1 mixtures, although they were still more efficient than granulated blast-furnace slag alone. On the other hand, partially ground LD steel slag alone was a better binding agent than the mixtures, with the exception of GBS-LD2 (50/50) and GBS-LD2 (30/70) after 182 days. Initial strength results with the GBS-LD2 mixtures were also better the higher the proportion of partially ground LD steel slag contained in them.

The results suggest that pre-grinding of the LD steel slag does not provide any improvement in reactivity; in fact its binding properties deteriorate as a consequence. The reason for this presumably lies in the fact that the proportion of fines in the material does not essentially increase as a result of pre-grinding, whereas the grain-size distribution becomes less favourable as far as reactivity is concerned.

The binding agents composed of partially ground granulated blast-furnace slag and LD steel slag or partially ground granulated blast-furnace slag and partially ground LD steel slag gave very much better strength than the granulated blast-furnace slag - LD steel slag mixtures, suggesting that, on the contrary, pre-grinding of the granulated blast-furnace slag can have a considerable effect on reactivity.

The initial compression strengths attained with the PGBS-LD1 binding agents were of the order of 1...1.5 MPa, and contrary to the situation with granulated blast-furnace slag as such, it would seem that the result improves with increasing proportions of partially ground granulated blast-furnace slag in the mixture. With one exception, all the specimens in this group showed a decline in strength in the interval 91-182 days, although no unambiguous explanation can be found for why this should be so.

The initial strength obtained with the binding agents based on partially ground granulated blast-furnace slag and partially ground LD steel slag (PGBS-LD2) were slightly lower than those with PGBS-LD1 mixtures, suggesting that here, too, pre-grinding of the LD steel slag did not improve reactivity. The results also indicate that better stabilization is in general achieved with binding agent - activator mixtures than with a single component.

Both PGBS-LD1 and PGBS-LD2 would seem to function best on average in ratios of 50/50 or 70/30, whereas the best proportions for GBS-LD1 and GBS-LD2 were 50/50 and 30/70. One reason for this may be that less LD steel slag is required to activate granulated blast-furnace slag when the latter is partially ground, so that an excess of the more reactive partially ground granulated blast-furnace slag will function better than an excess of its less reactive unground counterpart.

The compression strengths of the specimens stabilized with granulated blast-furnace slag and ground LD steel slag (LD3) or partially ground granulated blast-furnace slag and ground LD steel slag were evidently dependent on the proportions of the slag components (Figure 28). The larger the blast-furnace slag component, the higher the compression strength. LD3 clearly functioned as an activator, and even a small amount of activator was sufficient to trigger the blast-furnace slag reaction, since the material to be stabilized was pure crushed rock aggregate.

All the specimens containing LD steel slag products mixed with cement as the binding agent had very similar stabilization properties, their strength increasing as the proportion

of the LD steel slag component in the mixture decreased. The absolute differences in the strengths of the various mixtures were of the same order of magnitude in the case of the initial and final strengths.

No clear logic was detectable in the relations between the initial strengths of the aggregate specimens stabilized with mixtures of ground granulated blast-furnace slag and LD steel slag products depending on the proportions of the components, other than the fact that the ratio 90/10 gave the poorest results in every instance, even though it enabled the best final strengths to be attained. It is important to note that in many cases no appreciable stabilization can be detected at 28 days with these slowly reacting slag-based binding agents and that real strengthening of the structure becomes apparent only at an age of 91 days. The behaviour of the other binder-activator ratios followed the pattern described for the mixtures of LD steel slag and cement, i.e. the more ground granulated blast-furnace slag was present, the higher the compression strength.
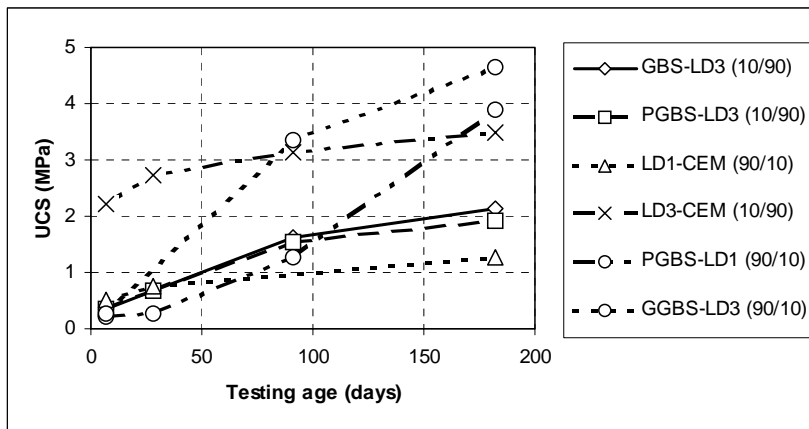

Fig. 28. Uniaxial compression strengths of finely ground binding agents.

The highest compression strengths of all in the stabilization of crushed rock aggregate (CR2) were achieved with cement-activated granulated blast-furnace slag (Figure 29), a level of almost 4 MPa being reached by 91 days, where the second best result, with the GBS5.5/LD1 formula, was around 2.5 MPa. By far the poorest strength result at 91 days was that given by stabilization with granulated blast-furnace slag alone, even the performance of LD steel slag alone being superior to this. Ground LD steel slag does not emerge as a particularly good activator by comparison with cement on the basis of the results presented in Figure 29, although it undeniably has an activating effect.
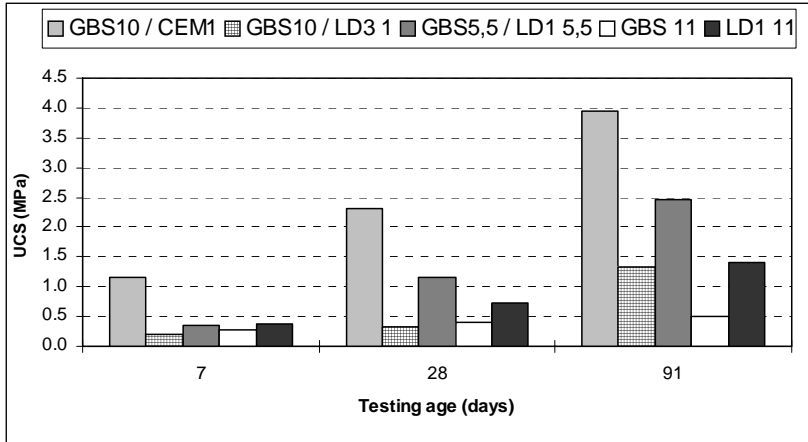
Fig. 29. Uniaxial compression strengths for crushed rock aggregate (CR2) stabilized with GBS/LD/CEM binding agents.

The compression strengths measured for CR2 aggregate samples stabilized with unactivated and activated partially ground granulated blast-furnace slag, as presented in Figure 30, point to distinctly better results with cement activation than with LD slag activation, the differences between all the groups being considerable.
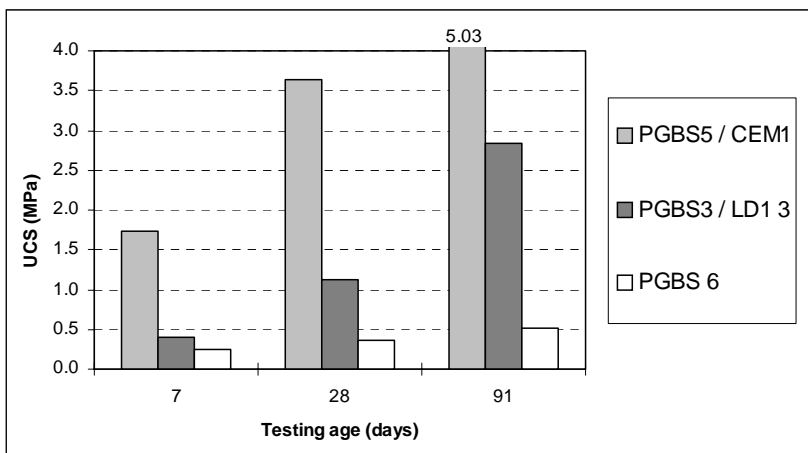

Fig. 30. Uniaxial compression strengths for crushed rock aggregate (CR2) stabilized with PGBS/LD/CEM binding agents.

The tests performed on finely ground binding agents similarly showed the cement-activated blast-furnace slag product, ground granulated blast-furnace slag, to be the most efficient (Figure 31), as it succeeded in binding the crushed rock aggregate better than

did cement alone and achieved a compression strength of about 3.7 MPa at the age of 91 days. The results for that point in time clearly indicate that unactivated ground granulated blast-furnace slag is the poorest of these alternatives, a reminder that this binding agent requires an activator even when used to stabilize a homogeneous material. The next poorest strength values were those obtained for ground granulated blast-furnace slag activated with ground LD steel slag and for ground LD steel slag alone.

Just as the ground LD steel slag alone stabilized the crushed rock aggregate better than did a mixture of LD steel slag and ground granulated blast-furnace slag, so coarse LD slag (LD1) provided a better result than the ground equivalent (LD3).
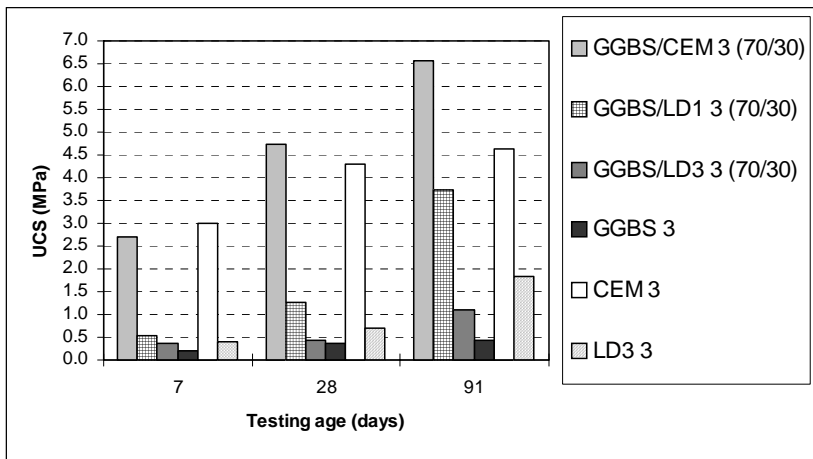

Fig. 31. Uniaxial compression strengths for crushed rock aggregate (CR2) stabilized with finely ground binding agents.

The results presented in Figures 29, 30 and 31 above suggest that the most reliable binding agents are cement-activated granulated blast-furnace slag products, while the most promising of the more recently introduced alternatives would seem to be one of the granulated blast-furnace slag products activated with 0-3 mm LD steel slag, although these do not achieve a performance comparable to their cement-activated counterparts in the proportions in which they were used here.

6.5 Determination of design parameters

6.5.1 Laboratory tests

6.5.1.1 Preliminary tests

As stabilization of the crushed till CT1 with all the GBS/CEM combinations proved excessive, the "basic" formula GBS10/CEM1 was eventually chosen for preliminary testing. On the other hand, all the GBS/LD1 combinations gave deficient stabilization, so that the formula chosen to represent this group was GBS10/LD1 15. Correspondingly, with the formulae GGBS-CEM 3(70/30) and GGBS-CEM 3(50/50) resulting in over-stabilization of the CT1 material and GGBS-CEM 2(70/30) in slight under-stabilization, it was decided to accept GGBS-CEM 2.5(70/30). The same amount of cement alone was also taken as a binding agent, in order to ensure the best possible basis for comparison.

The same binding agent formulae were used in the preliminary tests on the crushed gravel material CG2 as for CT1, and this was also true of the tests proper, with the exception of cement-activated granulated blast-furnace slag, where a formula with a relatively large amount of activator cement, GBS7/CEM1.75, was preferred for use with CG2, having given results in the preliminary tests that came reasonably close to the desired strength.

The GBS/LD1 binding agent again gave poor stabilization (Figure 32), while comparison of the stabilization achieved with cement-activated ground granulated blast-furnace slag and cement alone indicated that half a percentage unit more of the latter was required to give the same strength when used with CG2. It is also probable that the effect of GGBS/CEM intensifies with time, whereas the initial reactions take place very much more rapidly with cement alone.

Determination of the properties of the crushed gravel CG2 as an aggregate suggested that this should be more amenable to stabilization that the crushed till CT1, but the stabilization tests indicated that the opposite was true. The reason for this may well lie in the grain shapes, grain-size distribution and high fines content of the till, which means that it provides good stabilization at once, but this is only an apparent effect, as the sample will disperse if immersed in water, for instance.

The compression strengths of the crushed rock aggregate CR4 samples achieved in the preliminary tests are presented in Figure 32. The specimens stabilized with GBS/CEM combinations achieved fairly even strengths at all the proportions tested, and the formula GBS7/CEM1.5 was chosen for the actual tests. Moderate strengths were also attained with LD slag-activated granulated blast-furnace slag, and here the formula GBS10/LD1 10 was selected, i.e. a slightly lower proportion of activator LD slag was used than with the "less pure" crushed gravel and crushed till materials.

Of the cement-activated ground granulated blast-furnace slag formulae, GGBS/CEM 2 (70/30) was preferred, as this gave a relatively good strength in the preliminary tests, and again the same proportion of cement alone was selected, to facilitate comparison, although optimum stabilization would probably warrant a slightly greater addition.

Since the strength values obtained with the crushed rock aggregate CR5 were very similar to those given by CR4, the same binding agent formulae were chosen for use with this material, which would again favour comparability between the tests.
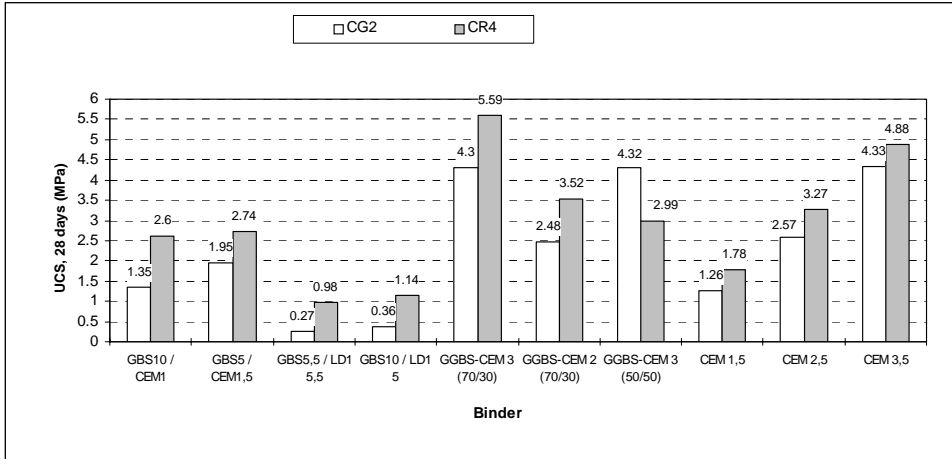

Fig. 32. Uniaxial compression strengths of the CG2 and CR4 specimens in the preliminary tests.

6.5.1.2 Laboratory tests proper

All the compression strengths recorded for the material CT1 were too high for stabilization purposes, except for that given by the binding agent GBS/LD1 (Figure 33). The values achieved with the finely ground binding agents GGBS/CEM and CEM were of the same order as lean concrete, that measured for cement alone being slightly greater than that for the GGBS/CEM mixture at an age of one month, whereas the latter had proved more effective by 91 days.

The laboratory tests proper gave similar results to the preliminary tests at 28 days, although it was not possible to predict from the preliminary tests that the subsequent increase in strength would have been as great as it actually proved to be. One factor contributing to this, at least, was that the slag component in each slag-based binding agent was quite high, so that once stabilization had begun well it then progressed well over a considerable period of time, leading to an eventual situation in which an excessive compression strength had developed.
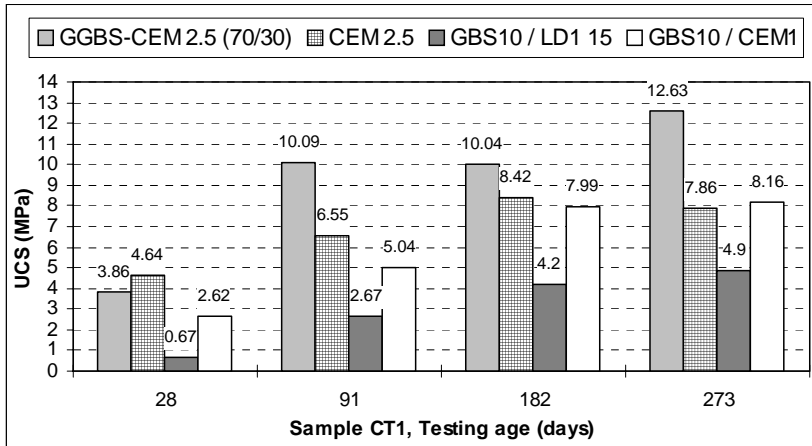
Fig. 33. Compression strengths for the aggregate material CT1 obtained with various binding agent formulae in the laboratory tests proper.

The compression strengths recorded for the material CG2 are presented in Figure 34. Here the figures of well over 5 MPa recorded for the cement-activated granulated blast-furnace slag binder at 91 days imply that again a somewhat excessive level of stabilization had been reached, whereas the specimens stabilized with cement alone represented a fairly suitable level, given that the target was a compression strength of the order of 3.0...3.5 MPa at 91 days.

By contrast, the strength readings obtained for the formula GBS10/LD1 15 are slightly on the low side, although the result is fairly acceptable in the long term. If road structure design were to be based on compression strengths at 28 days, the amounts of binding agent prescribed would be so large that the structure could well be said to be grossly over-estimated. Practically the optimum trend in compression strength was observed in these tests with the formula GBS7/CEM1.75, which gave a figure close to 3.5 MPa at three months and one of the order of 5 MPa after six months.

In view of the slightly different amounts of binding agent used, the compression strength at 28 days were marginally lower in the laboratory tests proper than in the preliminary tests, although the differences were not great.
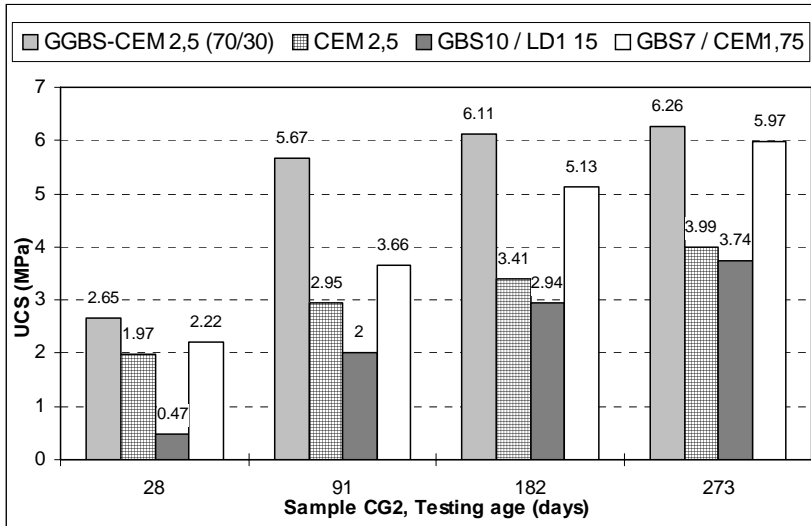
Fig. 34. Compression strengths for the aggregate material CG2 obtained with various binding agent formulae in the laboratory tests proper.

The strength values recorded for the pure crushed rock aggregate CR4 stabilized with the binding agent GGBS/CEM were only slightly higher than for CG2, although it must be borne in mind that the amounts of binder used were half a percentage unit lower (Figure 35). With the other binding agents the situation was the reverse, i.e. the compression strengths attained with CG2 were slightly lower than with CR4. In fact, with the exception of the GBS10/LD1 10 formula, the CR4 specimens may be said to have been over-stabilized.

The strengths recorded in the laboratory tests proper were of the same order of magnitude as in the preliminary tests, with the exception of GBS/LD1 binding.

Fig. 35. Compression strengths for the aggregate material CR4 obtained with various binding agent formulae in the laboratory tests proper.

The compression strengths obtained for the material CR5, presented in Figure 36, were of the same order of magnitude as for CR4, but the differences between the binding agents were smaller. Allowing for the slightly different amounts of binding agents used, the results of the laboratory tests proper are comparable to those of the preliminary tests.
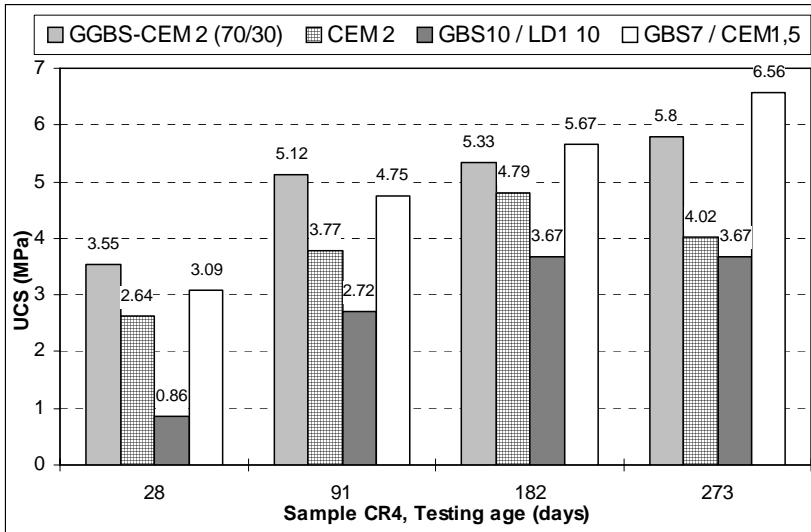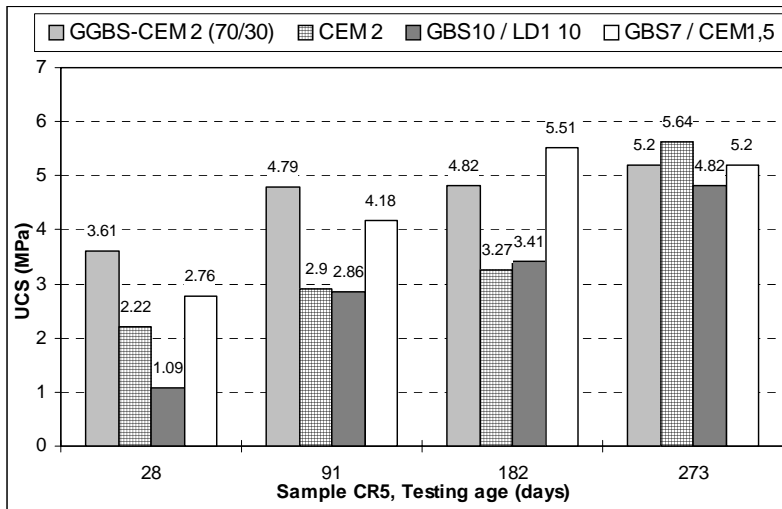

Fig. 36. Compression strengths for the aggregate material CR5 obtained with various binding agent formulae in the laboratory tests proper.

6.5.2 Test structures

6.5.2.1 Preliminary tests

The maximum dry density of the aggregate in the bearing course alone was 26.16 kN/m³ and its optimum moisture content 3.04%, the corresponding figures for the <16 mm fraction being 24.23 kN/m³ and 5.23%. The aggregate belonged to humus class I and the humus content of its fines (loss on ignition) was 1.32%. The pH of the sample was 8.72 and its specific gravity 2.95 t/m³. Grain-size distributions for the whole sample and the <16 mm fraction are presented in Figure 37. Thus the material available at the site could be said to be of good quality for stabilization purposes by the standards of old road materials in general, and definitely falls into the "well stabilizable" category.
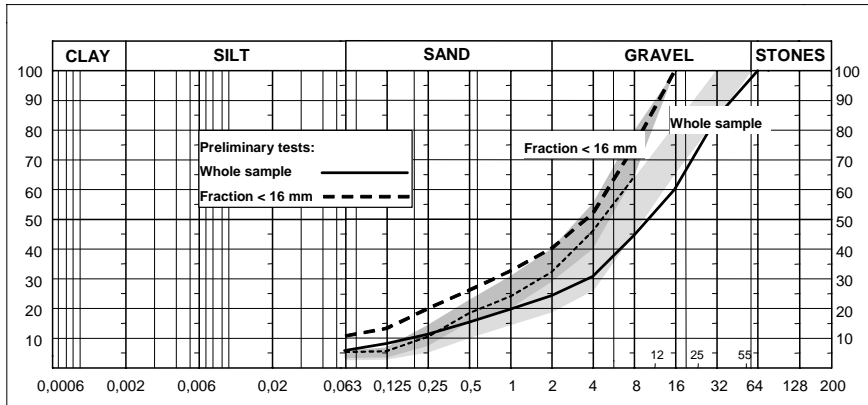

Fig. 37. Grain-size distributions for the material examined in preliminary tests on the bearing course of the experimental road site (solid and broken lines) and range of grain sizes of samples taken during the construction work (pre-mixed samples containing some old surfacing material).

The results of the preliminary tests are presented in Figure 38 for the binding agent formulae that come closest to those used in the test roads. The GBS4.5/CEM2 formula gave excessive binding relative to the target strength of approx. 3...3.5 MPa at 91 days, whereas the use of 1.0% or 1.5% cement provided relatively good strength results. The eventual formula chosen was thus GBS 4.5/CEM1.2.

As could be expected, the strengths obtained with the GBS/LD1 formula fell short of those achieved in the other tests, and it was decided to adopt the binding agent GBS4.5/LD1 10, in order to keep the amount of granulated blast-furnace slag the same as in the GBS/CEM structure. This was because the primary purpose of using LD steel slag had been to replace cement as an activator, so that it was more appropriate to compare these structures. A 10% addition of LD1 was chosen because this amount was deemed sufficient and any large amount would have represented a much less economic solution and could also have led to volume instability problems. It was also decided that two test

structures of different thicknesses should be produced according to the formula GBS4.5/LD1 10, in order to determine whether additional thickness could be used as a means of compensating for the poorer strength achieved.

Of the four ground granulated blast-furnace slag/cement stabilizations tested, the 3% addition gave excessive strength values, but the 1.5% alternative was relatively good. Eventually a binder content of 2% was chosen, with 70% GGBS and 30% cement. This gave a slightly high strength reading, but it was assumed that mixing of a small quantity of binding agent *in situ* would not be as efficient as in the laboratory, so that the result would not be so homogeneous. At the same time, a finely ground binding agent would exert its stabilizing effect more quickly at first. The same quantity of cement was also chosen, for the sake of comparability, and this again produced fairly good strength values, although well below those achievable by following the existing cement stabilization instructions.
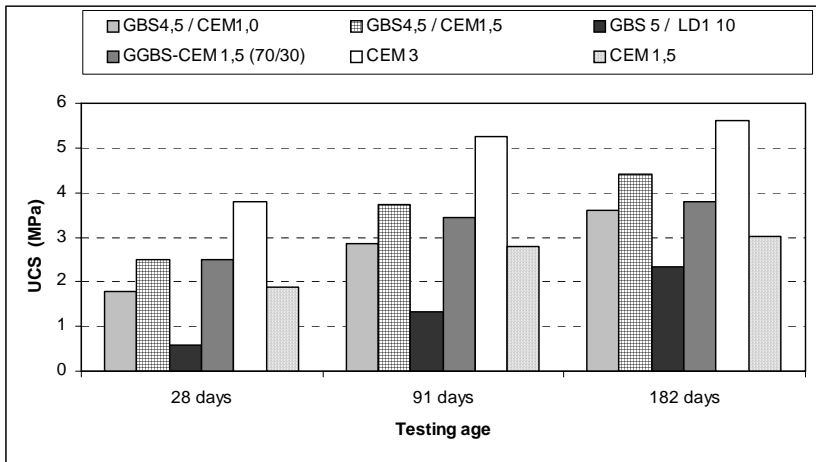
Fig. 38. Compression strengths obtained in preliminary tests at the experimental site.

6.5.2.2 Measurements made during the construction work and monitoring results

The grain-size ranges of the samples taken during the construction work, as shown in Figure 37 above, reflect the fact that preliminary mixing of the material had taken place by that time, in that the proportion of fines is smaller than usual, the material from the old surfacing having caused an increase in the 0.5...1-16 mm fractions, partly due to the fact that the smaller grains would have been bound together more tightly by the bitumen present.

The results of the Troxler measurements are presented in Figures 39 and 40. The density value of 2616 kN/m^3 used to calculate the degree of compaction is not entirely accurate, as it does not allow for the influence of the binding agent, but the results are

indicative of certain trends and can be used to assess the evenness of the work between the stretches of test road.

Considerable differences in the degree of compaction are observed between the measurement points down the centre of the road and in both carriageways in all the stretches of road, while the moisture content readings are most even at test site 5 and show the greatest scatter at sites 1 and 4. The results for site 1 would in fact have been relatively even if it had not been for the very low readings obtained for the left-hand carriageway.
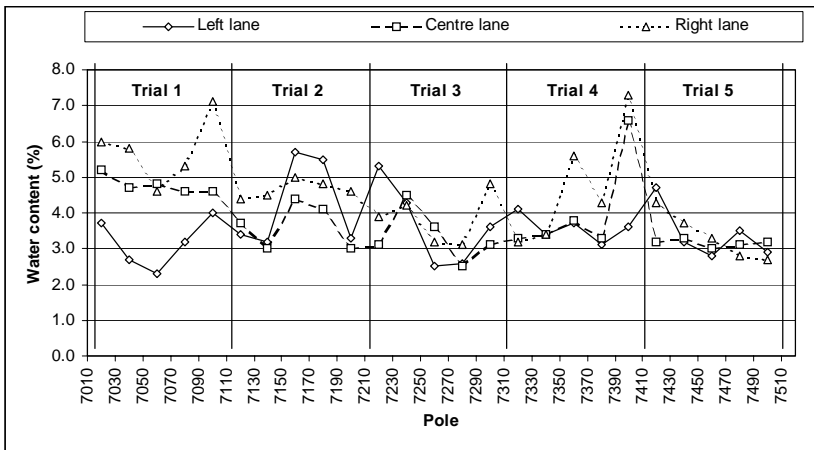

Fig. 39. Moisture content readings determined by Troxler device.
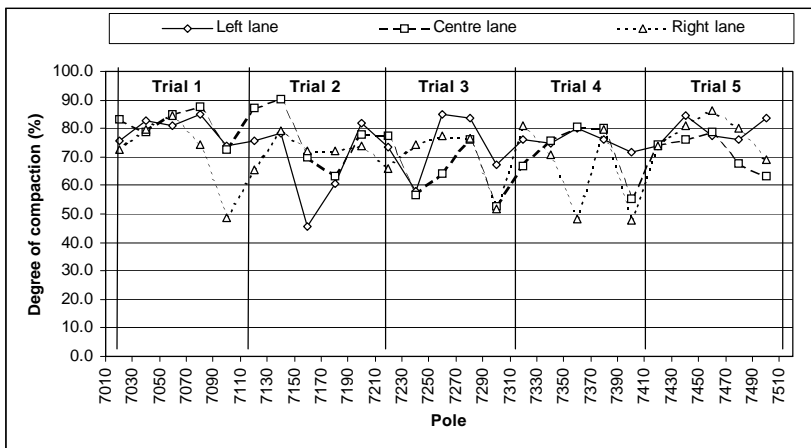

Fig. 40. Degree of compaction determined by Troxler device.

The compression strengths of specimens prepared from stabilized pre-mixed material from the test sites at 91 days are presented in Figures 41 and 42. Binding agents were added to the pre-mixed material in the same proportions as for stabilization *in situ*.

All the samples taken from the test structures gave higher strength values than in the corresponding laboratory tests with binding agents added in the same proportions, although the differences were relatively small. The values for sites T1 and T2 are rather low relative to the functional requirements, but it should be remembered that their binding agents consist entirely of slag. The samples from the centre line of the road do not differ in strength to any essential degree from those representing the two carriageways, but it would seem that the material in the right-hand carriageway of both sites T1 and T2 was stabilized to a greater degree than that in the left-hand carriageway.

By contrast, the laboratory-prepared specimens from the right-hand carriageway of site T3 gave poorer strength values than those from the left-hand carriageway, whereas the samples mixed at the site itself showed the poorest strength values to exist along the centre line. The trend in strength with time came very close to the intended pattern.

The specimens of cement-stabilized material from site T4 showed relatively small differences in strength between the two carriageways and the central line. The results also demonstrate the speed of reaction of cement relative to the slag-based binding agents. The strengths achieved are fairly close to the design values, i.e. slightly higher than those obtained with the coarser material at sites T1-T3. The results for T5, on the other hand, point to the influence of the slag component, in that the initial strengths were lower than those obtained with cement at site T4 whereas those achieved after 273 days were larger than in the latter case. Again it may be said that the choice of binding agent was relatively successful. The final strength values were a little on the large side, but this must be accepted in order to avoid excessively low initial strengths.

Fig. 41. Compression strengths of specimens prepared from pre-mixed and stabilized samples from test sites 1, 2 and 3 at 91 days of age (PRE = pre-mixed material from the site stabilized in the laboratory, STAB = stabilized material from the test site).
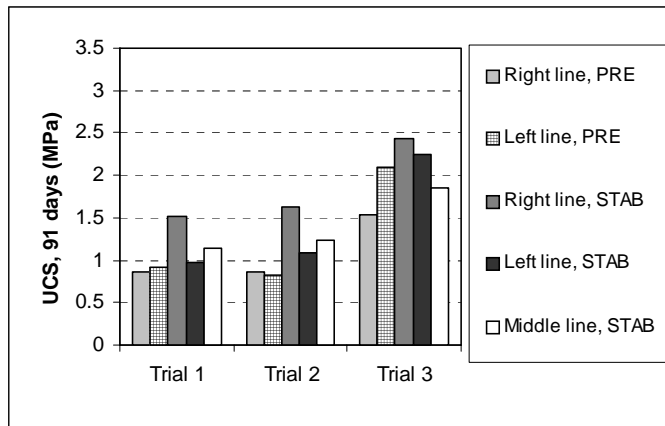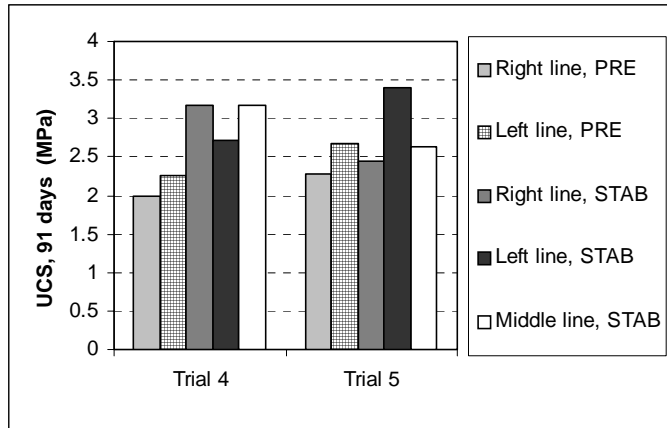
Fig. 42. Compression strengths of specimens prepared from pre-mixed and stabilized samples from test sites 4 and 5 at 91 days of age (PRE = pre-mixed material from the site stabilized in the laboratory, STAB = stabilized material from the test site).

The materials stabilized with LD slag-activated granulated blast-furnace slag had compression strengths of very much the same order of magnitude as the specimens prepared in the preliminary tests, although a larger amount of binding agent was required at site T1 than at T2 (Figure 43, upper diagram). All the strength results were similar up to an age of 91 days, and it was only at 182 days that the preliminary test results were somewhat higher. The amounts of binding agent used were also slightly greater in the preliminary tests than in the experimental roads, where the formulae were GBS4.2/LD1 9.0 for T1 and GBS3.2/LD1 7.2 for T2 (Figure 46). Also, the relatively large amount of binding agent contained in the stabilized material as a whole serves to even out the differences in the strength results.

The trend in stabilization at site T3 with respect to the preliminary tests and samples of material stabilized *in situ* is highly logical, bearing in mind that the amount of binding agent added *in situ* was less than had been planned, giving an eventual formula of GBS3.5/CEM0.9 (Figure 43, middle diagram).

The compression strengths obtained for the cement-stabilized material at a binding agent content of 1.5% were slightly lower in the preliminary tests than at site T4, where the actual binder content proved to be 1.8% (Figure 43, lower diagram). The result gained with a binder content of 3% in the preliminary tests was naturally very much higher. The opposite results were obtained with cement-activated ground granulated blast-furnace slag, however, although the eventual amount of binding agent used at site T5 was the same as with cement alone. The differences were nevertheless sufficiently small that they could be attributed to the normal scatter in stabilization results.

When examining the results depicted in Figure 43 it is important to remember that the stabilized material had been passed through a 16 mm sieve prior to preparation of the specimens, which will mean an increase in the proportion of binding agent, as the larger grains will have been removed whereas the binding agent will scarcely have contained

any grains of that size. In other words, the quantity of binder will have remained practically the same but that of aggregate will have diminished in absolute terms. The preliminary tests will not have been subject to any "error" of this kind, as grains larger than 16 mm will have been removed from the aggregate prior to adding the binding agent.
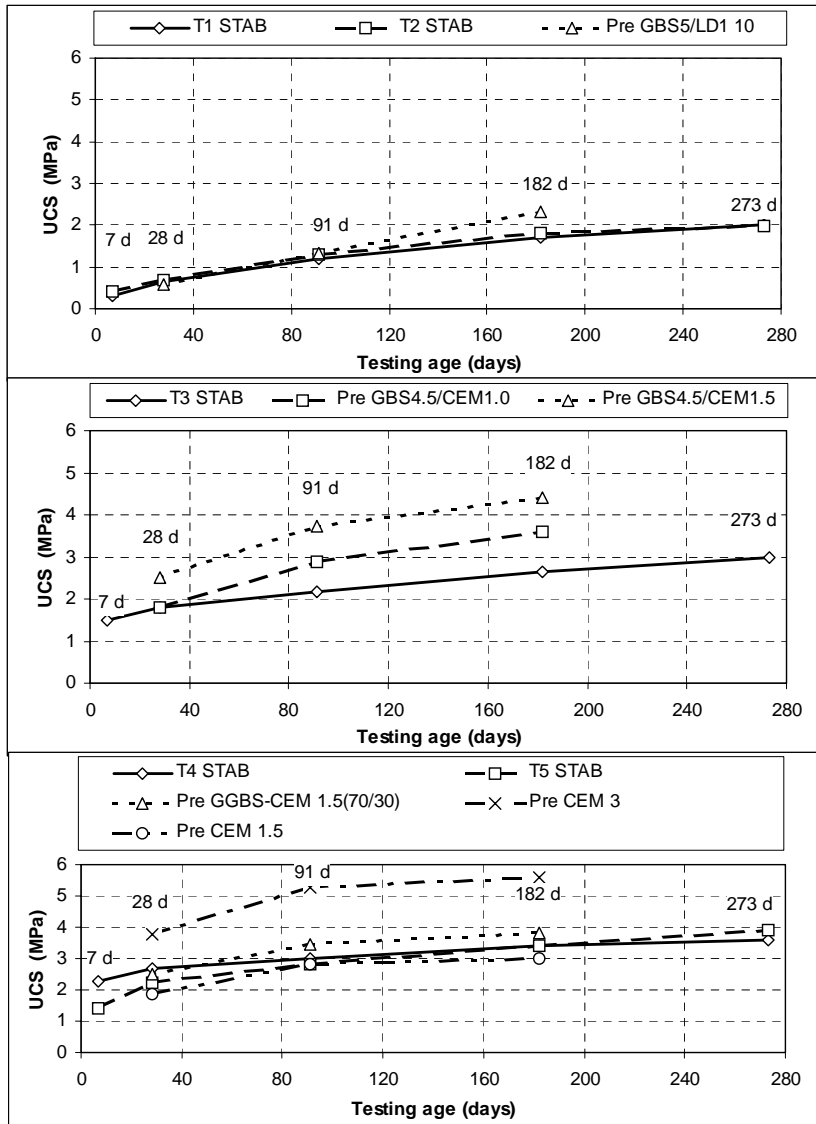
Fig. 43. Mean compression strengths of stabilized material from the test structures and compression strengths obtained for the various stabilizer formulae in the preliminary tests (Pre = pre-mixed material from the site stabilized in the laboratory, STAB = stabilized material from the test site).

The resilient moduli (M_R) for the specimens of stabilized material prepared for the dynamic triaxial tests are quoted as a function of the sum of the principal stresses in Figure 44. If we assume that the sum of the principal stresses in a bearing course is of the order of 140-280 kPa (Kolisoja 1993), we obtain the following resilient moduli for the stabilized structures:

- T1&2 (GBS/LD1): 490...650 MPa
- T3 (GBS/CEM): 710...880 MPa
- T4 (CEM): 630...780 MPa
- T5 (GBS-CEM): 730...900 MPa

These results lie fairly close together and are somewhat illogical, as the values for T4 are lower than those for T3 in spite of the fact that the compression strength results would suggest the opposite. It should be remembered, of course, that compression strength does not necessarily tell us very much about the functioning of a structure, a fact that may be still more relevant than ever when we are moving in the range of semi-rigid structures. The results obtained by the various testing methods are compared in more detail in section 6.6.

Fig. 44. Dependence of the resilient moduli (M_R) of samples of stabilized material from the test structures on the sum of the principal stresses.

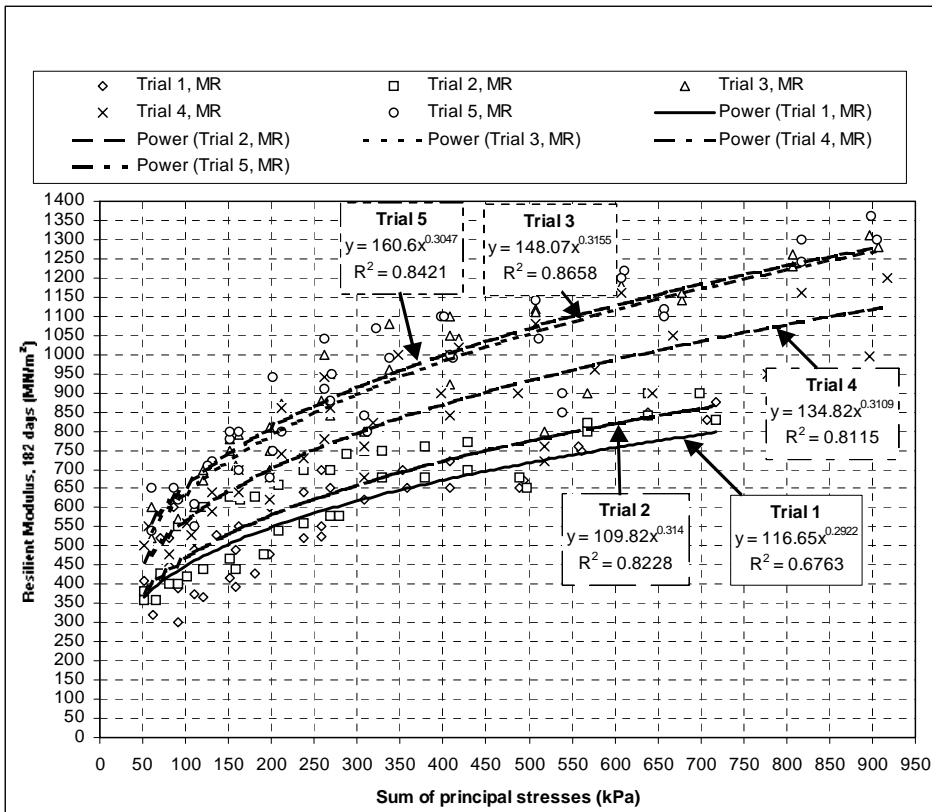The outcome of the vehicle-based drillings in the test structures was a set of data on the eventual thicknesses of the structural layers and their material composition. The results reflect well the challenge facing *in situ* stabilization when nothing is done to the other structural layers of the road. The layers vary greatly from one point to another (Figure 45), with an older bitumen surfacing to be found beneath the bearing course in the majority of places, as the old road had been repaired at some time in the past by building a new layer of crushed rock aggregate on top of it and resurfacing this with a fresh bitumen wearing layer. The aggregate used for this purpose had nevertheless proved within a few years to be of rather poor quality, and this had been the reason for the present decision to attempt *in situ* stabilization with granulated blast-furnace slag.
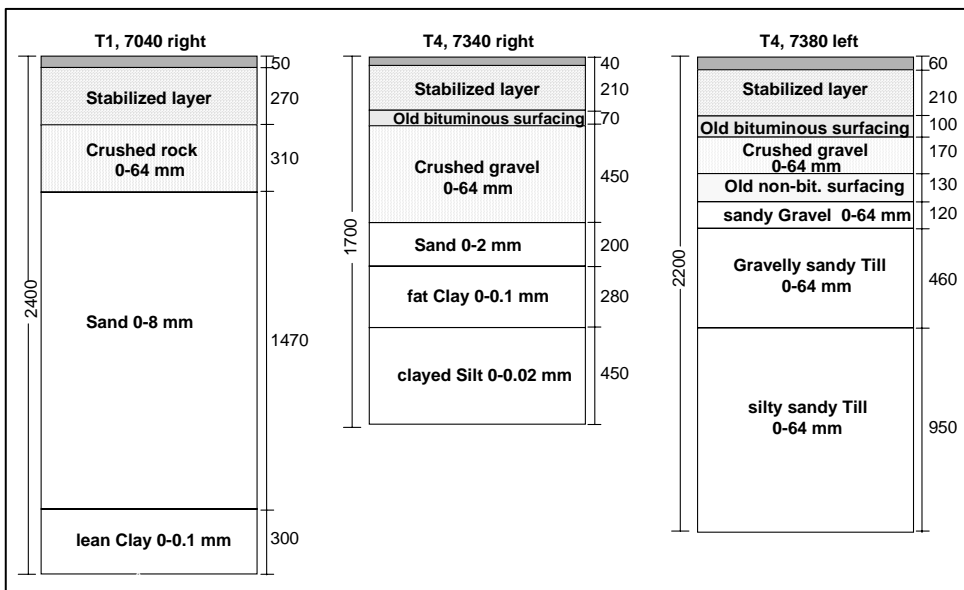
Fig. 45. Typology of cross-sections for the test structures.

The aim of the stabilization had been to produce a stabilized layer of thickness 200 mm at each site except for T1, where it was to be 250 mm, in order to determine whether the poorer strengthening achieved with LD slag-activated granulated blast-furnace slag could be offset by a thicker stabilized layer, so that adequate bearing capacity could be achieved.

The intended thickness was achieved fairly well on average in the case of site T1 (Figure 46), the thickest stabilized point revealed by the drilling being 280 mm and the thinnest 230 mm. In the case of structure T2, on the other hand, the stabilization depth (average 265 mm) proved to be too great at every point except one, and was mostly thicker than in T1. There was only one drilling point at which the target thickness of 200 mm prevailed, and there was in any case considerable scatter in the thicknesses. The thickness of the stabilized layer also exceeded the design figure by an average of 43 mm in the case of structure T3.

The stabilization depths in T4 and T5 were close to the planned values, and the scatter between the points investigated was relatively small.

Figure 46 also contains details of the eventual binding agent concentrations. The contractor was notified of the concentrations to be used in kg/m^2, defined on the basis of the design thicknesses of the stabilized layers and the Proctor density results obtained in the preliminary tests, and the corresponding data, i.e. layer thickness and Proctor density results in the monitoring tests, were taken into account when calculating the actual concentrations. In consequence, the actual binding agent concentrations determined at test sites 1, 4 and 5 were relatively close to the design values, although admittedly the low concentrations used in the latter two cases meant that the figure was more susceptible to fluctuations in layer thickness. In the case of T2 the thickness of the stabilized layer was considerably greater than planned, and the binder concentration consequently very much lower. The concentration was similarly far too low in T3. These discrepancies will be borne in mind when developing the dimensioning procedures further, as such procedures should be drawn up on the basis of the real, implemented concentrations.

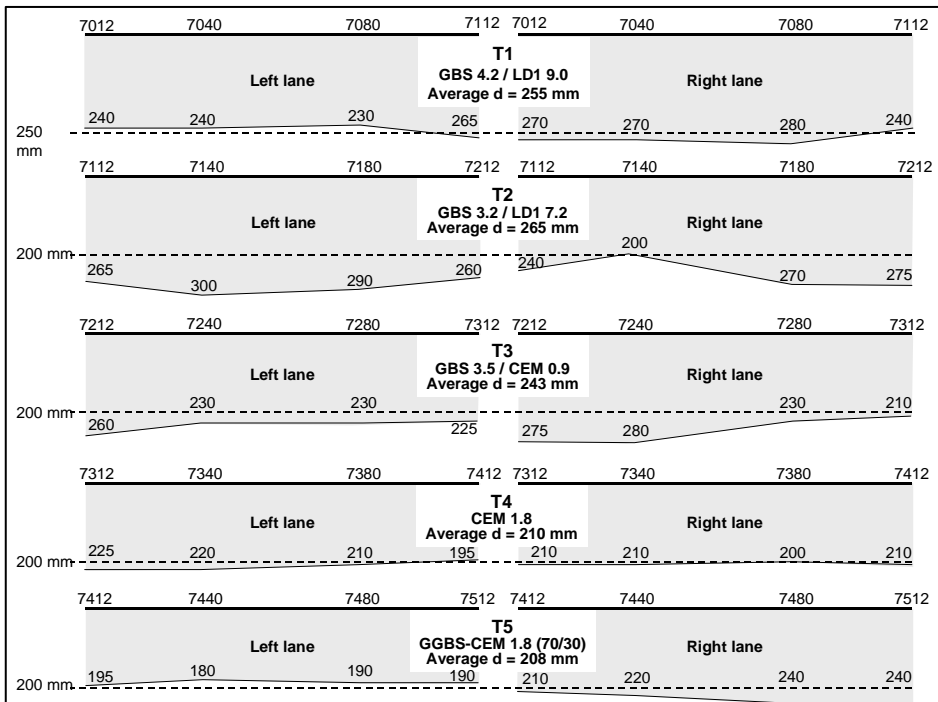**Fig. 46. Calculated actual thicknesses of the stabilized layer and mean binding agent concentrations in the test structures.**

The E_{TOT}-moduli depicting the bearing capacities of the whole road structure had been of the order of 230...330 MPa prior to stabilization (Figure 47), with the best initial bearing capacity existing at site T1 and the figure deteriorating in numerical order, so

that the poorest capacity was recorded at site T5. This was due to the fact that the structural layers of the old road had been poorer at the far end and the subgrade more susceptible to frost damage, as shown in Figure 49. The highest incidences of frost heave each year had been recorded at the end of section T4 and throughout T5.

Since the road structure was improved entirely by intervention in the bearing course and pavement, considerable annual scatter is observed in the E_{TOT} -moduli, on account of the formation and thawing of frost in the ground and the effects of moisture on the road structure, as it is also more susceptible to environmental effects of this kind than an entirely new road would be, on account of deficiencies in drainage and the nature of the materials remaining from the structural layers of the old road.

The E_{TOT} results nevertheless also reflect differences in stabilization between the binding agents. The least effect a month after stabilization was observed in the case of the agents containing slag alone, i.e. in test structures T1 and T2, where the strength values increased by 6% and 15% respectively. The increase in T3 was 33%, that in T4 75% and the strength value in T5 had more than doubled. By the end of the summer of the following year the trend had continued steadily in the case of T1-T3 but had slowed down in T4-T5, and it is evident from the results that the E_{TOT} bearing capacities had become established at certain levels. It is particularly significant that the monitoring results for the year 2000 lie relatively close together, with only T1 standing out as distinctly better than the others. The reason for this may lie in the fairly thick stabilized layer and the use of a larger proportion of binding agent than in the other cases. It was observed in spring 1998 that the road structure was still frozen in places, as shown by the frost gauges, and this is evidently reflected in the higher bearing capacities recorded, while the higher values in autumn 1999 than at other times on average may well be attributable to the moisture situation in the road structure.
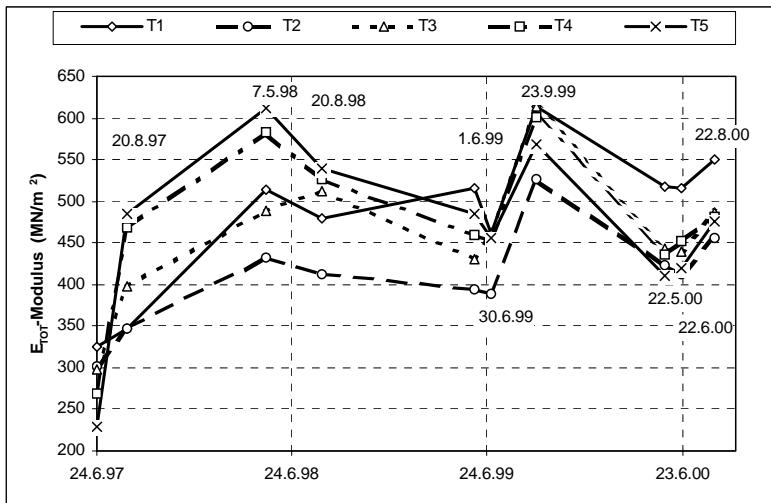

Fig. 47. E_{TOT} moduli, describing the total bearing capacity of the road structure, at different points in time. The values are means for the ten measurement points on each stretch of test road at the given time.

The bearing capacities of the whole road structure a year after stabilization had attained the following levels:

Trial	T1	T2	T3	T4	T5
E_{TOT} (MPa)	500	420	475	495	495

It may thus be said that bearing capacities of approximately the same order of magnitude were achieved in all the test structures regardless of the manner of stabilization or the binding agent used, the only exception being T2, where the actual amount of binding agent present was considerably less than that intended. The thickness of the stabilized layer was greater in the places where coarse binding agents were used than with finely ground binding agents, so that the poorer strength results in these cases may be said to be compensated for by thickness.

Falling weight deflection basin diagrams for structure T5, in which the initial maximum deflection was greatest, at approx. 700 μm, are presented in Appendix 2. The initial values at the other sites were all in the range 500...600 μm. A year after stabilization the deflections were of the same order of magnitude at all the sites, again with the exception of T2, where they were somewhat higher. It was also clear that the deflection basins for T1 and T2 were more shallow than for the others, i.e. there was less difference between the minimum and maximum values. Similarly, the scatter was somewhat less at site T3 than at T4 or T5. The smaller scatter is a consequence of the existence of better structural courses beneath the stabilized layer, and also of a higher proportion of binding agent in the case of the coarser binding materials.

A figure of 400 mm was used for the thickness of the bearing course in the E_{TOT}-modulus calculations performed before stabilization, on account of the fact that backcalculation gives unrealistically large modulus values for the bearing course of a road structure with a thin pavement if the assumed thickness is less than 300 mm. In the case of the calculations performed after stabilization, however, the actual layer thicknesses were used.

The moduli for the stabilized layers (E_{STAB}) had settled at given levels by about a year after stabilization, as seen in Figure 48. Thus where the moduli for T1 and T2 increased by 50% and 70% respectively within the first month, that for T3 by a factor of 1.5, that for T4 9-fold and that for T5 14-fold, the situation by August 1998 was that they had all increased further and that only the cement-stabilized structure was showing any signs of a slowing down in this effect.
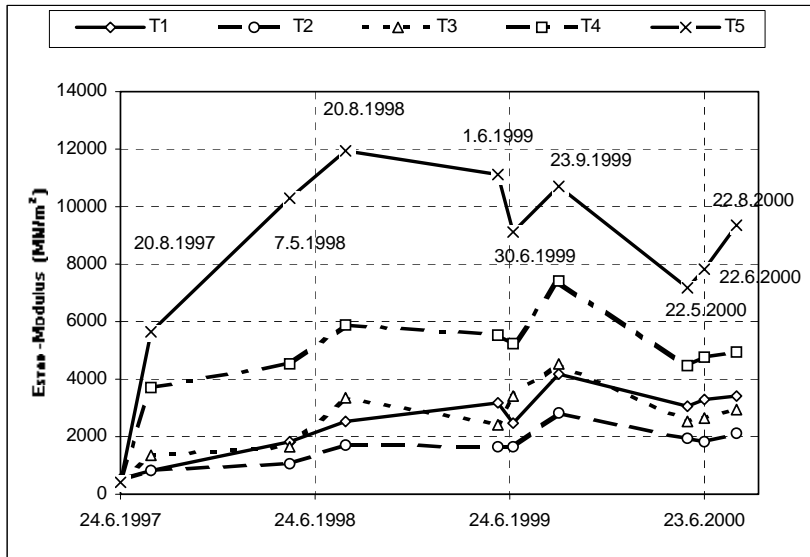
Fig. 48. Back-calculated E$_{STAB}$ moduli, describing the bearing of the stabilized layer, at different points in time. The values are means for the ten measurement points on each stretch of test road at the given time.

The E-moduli of the stabilized layer (E$_{STAB}$) had reached the following levels a year after stabilization:

Trial	T1	T2	T3	T4	T5
E$_{STAB}$ (MPa)	3150	1950	3150	5450	9600

Thus the binding agent combinations GBS/LD 1 (T1) and GBS/CEM (T3) yielded similar results in terms of the E-modulus of the stabilized layer, while T2 gave by far the poorest result, on account of the deficiency in the quantity of binding agent used. The cement-activated ground granulated blast-furnace slag gave a distinctly better modulus result than any of the other treatments, almost twice the figure achieved with cement stabilization.

The modulus for the stabilized layer was examined both by calculating it as a mean of the values for all the points measured, as above, and by calculating it as the mean for the six measurement points at the centre of each test structure, eliminating those which were closest to the adjacent structures. The latter method was designed to eliminate the effects of contamination between binding agents in the course of spreading or mixing. The results obtained by the two methods were nevertheless very similar, except in the case of T5, for which differences of almost 1000 MPa were recorded depending on the method of calculation. This is probably due to the fact that stabilization with the cement-activated ground granulated blast-furnace slag was so pronounced that even the scatter between measurement points was greater.

The damage surveys provided evidence of the effects of ground frost in promoting cracking, which was most pronounced in structures T4 and T5, where the subgrade was highly susceptible to frost (Figure 49). No actual shrinkage cracking was observed within the three years of monitoring, however. The incidence of cracking may also have been affected to some extent by the fact that the layers stabilized with finely ground binding agents were thinner than those in which coarse binders were used. Also, the larger quantities of the coarse materials may have had the effect of reducing the incidence of cracks extending to the surface.
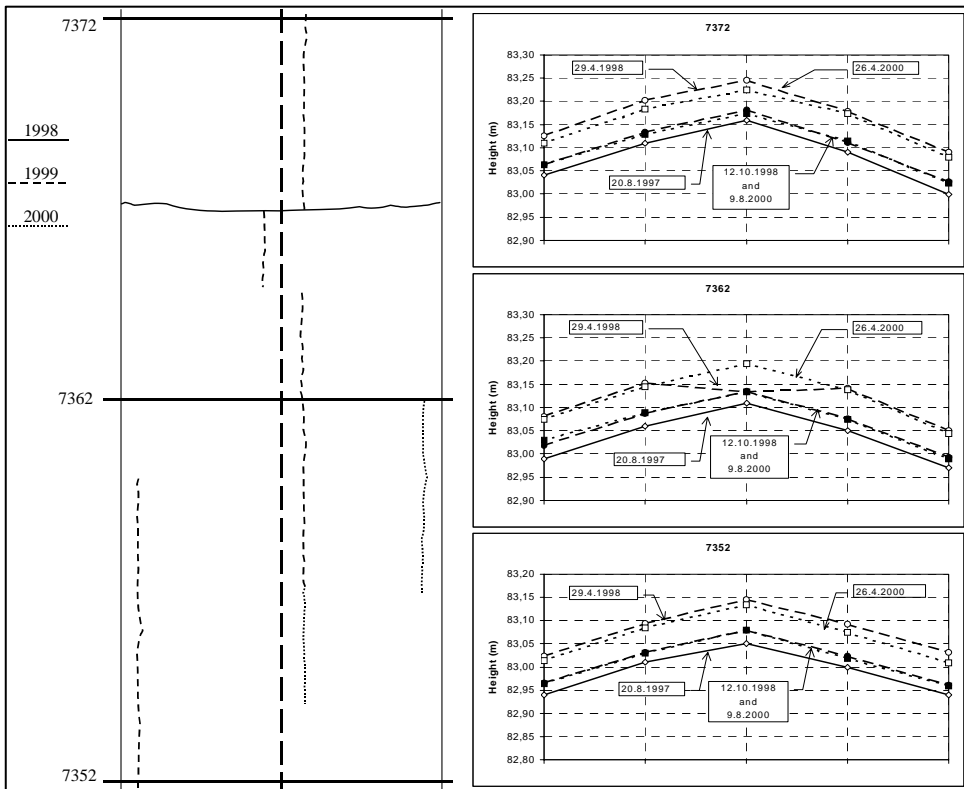

Fig. 49. Point in T4 at which the most serious cracking to be found in any of the test structures occurred, and frost heave at that point in the three years of monitoring.

6.6 Summary of the results

6.6.1 Verification of the stabilization classification

The following conclusions can be reached on the basis of the second stage in the research.

 Cement-activated granulated blast-furnace slag:

- The greatest strengths were achieved with a formula containing 10% slag and 2% cement (GBS10/CEM2), and the second best results on average with the formula GBS7.5/CEM1.5, although those gained with GBS5/CEM2 were almost as good. The formula GBS10/CEM1 gave markedly lower strength values.
- The compression strengths of those specimens that were successfully stabilized increased 1.2...1.7-fold between the ages of 28 and 91 days, but scarcely any improvement in strength was noted with the formula GBS10/CEM1.
- The amount of activator cement used has a decisive influence on stabilization, the correct amount probably being in the range 1...2% for all materials stabilized. If too little activator is used, even a large amount of granulated blast-furnace slag will not ensure stabilization.
- The effects of alterations in the amount of slag present are seen much more slowly than those in the amount of cement.

 Cement-activated partially ground granulated blast-furnace slag:

- The highest compression strengths were obtained with 5% slag and 2% cement in the mixture by dry weight (PGBS5/CEM2), and the second highest with the formula PGBS10/CEM1. The poorest performance in the uniaxial test was recorded for specimens stabilized with a PGBS7.5/CEM1 or PGBS5/CEM1 binder.
- The improvement in strength over the interval 28...91 days was roughly 1.7...1.8-fold, except that scarcely any improvement at all was noted with the PGBS5/CEM1 binder.
- The quantity of activator cement had the same general effect on the outcome as with granulated blast-furnace slag, although not so pronounced, as partially ground granulated blast-furnace slag is itself more reactive than the unground form.

 Cement-activated ground granulated blast-furnace slag:

- The highest compression strengths in the uniaxial test were obtained with formulae calculated in accordance with the Instructions for Cement Stabilization (Tielaitos & Tiehallitus 1992), higher values being attained with GGBS-CEM 3(50/50) than with GGBS-CEM 3(70/30).
- The increase in strength from 28 to 91 days was about 1.2-fold with the formula GGBS-CEM 3(70/30).

 Given the same amount of activator cement, the quantity of partially ground granulated blast-furnace slag required for a given stabilization strength was about a half of that of the unground equivalent, and still smaller amounts would be needed if the slag were to be still more finely ground, until eventually a point would be reached at which the proportion of activator cement could also be reduced.

 The large degree of scatter in strength results between the specimens, given the same binding agent formula, is a clear indication that the choice of formula is by no means a

straightforward one and that it is always necessary to carry out preliminary tests. The essential thing, however, is to reduce the number of these tests to the minimum needed in order to know the range of values in which to operate, so that the process will be technically and economically viable.

The system of strength classification for road materials was validated for nine samples in the light of earlier research. Stabilization of six of the specimens with cement-activated ground and unground granulated blast-furnace slag proceeded as predicted by the classification, but predictions based on organic content alone were successful in only five cases, whereas grain size provided accurate evaluations in seven. Some of the incorrectly predicted cases in fact came very close to the borderlines for the strength classes predicted.

6.6.2 Extension of the range of binding agents

The best stabilization results among the coarse-grained binding agent components were recorded with partially ground granulated blast-furnace slag (PGBS) and the poorest with granulated blast-furnace slag (GBS). The second best was LD steel slag (LD1).

The mean results obtained with binders composed of mixtures of granulated blast-furnace slag or partially ground granulated blast-furnace slag with LD steel slag allow the following observations to be made:
- The highest strength was attained with partially ground granulated blast-furnace slag and LD steel slag (PGBS-LD1)
- No increase in reactivity at all is achieved by pre-grinding the LD steel slag.
- LD steel slag definitely has a favourable effect on stabilization (especially in accelerating this at the early stages).
- The binding agents composed of granulated blast-furnace slag and LD steel slags (GBS-LD1 and GBS-LD2) were most effective in ratios of 50/50 and 30/70.
- The binding agents composed of partially ground granulated blast-furnace slag and LD steel slags (PGBS-LD1 and PGBS-LD2) were most effective in ratios of 70/30 and 50/50.

The various finely ground binding agents provided very similar results in terms of compression strength, only the combinations containing granulated blast-furnace slag or partially ground granulated blast-furnace slag standing out as slightly poorer than the others, as all the specimens concerned, with only one exception, had compression strengths of less than 2 MPa after 182 days. All the finely ground binding agents gave results of the order of 2...4 MPa with crushed rock aggregate at this point.

The mean results obtained with crushed rock stabilized with finely ground binding agents allowed the following observations to be made:
- In the case of the binding agents composed of granulated blast-furnace slag and ground LD slag (GBS-LD3) or partially ground granulated blast-furnace slag and ground LD slag (PGBS-LD3), any increase in the proportion of the granulated blast-furnace slag component tended to promote stabilization.
- In the case of the mixtures of LD steel slags with cement, the stabilization of crushed

rock aggregate improves almost linearly as the proportion of the LD steel slag component decreases.

- No clear, logical pattern could be detected in the initial compression strength achieved by stabilization with mixtures of ground granulated blast-furnace slag and LD steel slag, although with the exception of the mixing ratio 90/10, the behaviour of these binding agents was similar to that of LD slag - cement mixtures.

  The following conclusions can be reached from the comparison of binding agents:

- The crushed rock aggregate was stabilized best by cement-activated granulated blast-furnace slag products.
- The stabilization properties of a mixture of LD steel slag (LD1) and a granulated blast-furnace slag product are relatively good.
- There are substantial differences in stabilization potential between activated and unactivated granulated blast-furnace slag products.
- When stabilizing pure crushed rock aggregate, the strength achieved by stabilization increases almost linearly with binding agent content.

  The following general conclusions can be stated on the basis of this stage in the research:

- The best slag-based binding agents available are cement-activated granulated blast-furnace slag products.
- Binding agents comprising mixtures of LD steel slag (LD1) and granulated blast-furnace slag products are sufficiently promising as to warrant further investigation.
- The pre-grinding or grinding of LD steel slag does not provide sufficient benefit in terms of binding properties relative to the costs of the grinding processes that the continuation of research into this aspect would seem justified as far as the use of such products in road construction is concerned.
- By contrast, a substantial increase in the reactivity of granulated blast-furnace slag is achieved by pre-grinding.

  The results of the paste tests suggest that there is little point in continuing experimentation with partially ground LD steel slag, and the same may be said of the three poorest binding agents in the tests based on the stabilization of crushed rock aggregate, i.e. LD3 alone or as an activator and GGBS alone. Similarly, it was decided to abandon experimentation with the pre-grinding of granulated blast-furnace slag as the process raises the price of the product beyond the point at which it ceases to be a competitive alternative.

6.6.3 Definition of design parameters (laboratory tests)

The following observations may be made on the basis of the uniaxial compression strengths obtained in the laboratory tests:

- The best strengths in the stabilization of a "pure", well stabilizable material are obtained with cement-activated granulated blast-furnace slag.
- Very high strength values can be achieved in the long term with cement-activated granulated blast-furnace slag when used in the right amounts and mixed in the right

proportions.

- More "suitable" strengths were achieved with LD slag-activated granulated blast-furnace slag, although the amounts of binding agent required are quite large.
- The long-term stabilization result is somewhat problematical and difficult to control as far as the design of road structures is concerned, but this aspect has to be taken into account more carefully than ever when using relatively slow-acting slag-based binding agents.

Many of the specimens stabilized with GBS/LD1 mixtures contained cracks, presumably on account of the volume instability of the LD slag component. These cracks were not alarming in size, nor did they have any decisive effect on the results, but this tendency would have to be borne in mind if very substantial LD slag components were to be proposed. The amounts concerned here were no more than 15% of the total mass of material to be stabilized.

The results point to a minimum amount of activator that must evidently be present in order to achieve significant stabilization. A similar conclusion is reached by Kukko & Ruohomäki (1995) when studying the stabilization of clays with various binding agents, as they noted that even a small change in the amount of binder could have a major impact on the strengths attainable. The effects are in fact a lot less abrupt in the case of slag stabilization, and the general principle has been that the amount of binding agent and proportions of the mixture should be selected in such a way as to achieve strengths of the order of 3.0...3.5 MPa within three months with reasonable quantities of slag, i.e. 5...10% of the stabilized mixture by weight. This will ensure that the structure does not become too rigid in the course of time.

6.6.4 Definition of design parameters (case: test structures)

The choice of binding formulae for the test structures was highly successful, so that the differences in reactivity between the agents were well compensated for by adjustments in their amounts and in the thickness of the layer to be stabilized. This meant that similar bearing capacities for the whole road structure were obtained with the poorly binding coarse granulated blast-furnace slag and steelmaking slag to those obtained with the finer structures containing cement and ground blast-furnace slag, in spite of the great differences in the moduli for the stabilized layer. It should be remembered, of course, that the subgrade was less suitable on the stretches used for testing the fine-grained binding agents than on those used for the coarse-grained agents.

The advantage with coarse-grained binding agents may be regarded as being the fact that the larger amount of binder mixes into the stabilized material more "comprehensively", forming a more continuous mesh within the structure and giving a more even result. This means that the lower uniaxial compressive strengths are compensated for in the structure. A further advantage attained at the lowest strength levels is that there are less transverse cracks in the pavement of the kind that reflect shrinking cracks in a rigid structure.

6.6.5 Strength development and its prediction

In many cases it is not economically feasible to initiate stabilization in a material by means of an addition of slag alone, and an activator is required as well. However, the amount of activator is not so great, as the aim when selecting the relation between the amount and composition of the binding agent was to attain a compression strength of the order of 3.0...3.5 MPa within three months. The present research suggests, in fact, that lower strength levels, even of the order of around 1 MPa, may be adequate for stabilization with granulated blast-furnace slag activated with LD steel slag or cement, for if the binding agent has a high slag component, e.g. a total of over 10%, it can be assumed that a great deal of additional strength will develop after 91 days of age.

One thing that is certain, however, is that the use of slag-based binding agents calls for at least a relatively precise knowledge of the manner in which strength develops in the stabilized material. With this in mind, a regression model was constructed on the basis of the present data in order to enable equations to be put forward for predicting the development of strength in accordance with the binding agent used. The modelling proceeded in a stepwise manner by first constructing equations for the whole body of data relevant to each binding agent and then eliminating those cases in which the resulting strength level was well above the average, the aim being to improve the level of significance attached to the resulting exponents. This latter step was not applied to the cases of cement stabilization, however, as the data set was fairly limited in this respect anyway. The criteria for eliminating data items, expressed in terms of the increase in strength between 28 and 91 days, were the following:

- GBS/CEM: over 1.6-fold
- GGBS/CEM: over 2-fold
- GBS/LD1: over 3-fold

Exponents for the various binding agents were then calculated from this reduced body of data by means of the regression model, after which 91-day strength values calculated from the 28-day results by means of the exponents were compared with the actual strengths measured at 91 days and the mean deviations of the predictions obtained from the real values by means of the model were determined. The result was that the model gave on average lower strengths than the measured ones for all the binding agents. Thus a correction coefficient for the exponent was developed from the mean deviation, by which the model could be adjusted to conform more closely to the real trend in the strength of the stabilized material. Finally the corrected model was checked once again. The eventual exponents for use in predicting the trend in the strength of the stabilized material were:

GBS/CEM: $\quad UCS_{91} = 1.2980 \times (UCS_{28})^{1.0374}$ (14)

GBS/LD1: $\quad UCS_{91} = 2.2392 \times (UCS_{28})^{1.2554}$ (15)

GGBS/CEM: $\quad UCS_{91} = 1.3251 \times (UCS_{28})^{1.0682}$ (16)

CEM: $\quad UCS_{91} = 1.3934 \times (UCS_{28})^{0.9227}$ (17)

where $\quad UCS_{28}$ is the uniaxial compression strength at age 28 days and
UCS_{91} the uniaxial compression strength at age 91 days

A very rough estimate of the eventual trend in compression strength can also be obtained by examining the level attained by 28 days. If this figure is over 1.2-2.0 MPa with a GBS/CEM or GGBS/CEM binding agent combination, the amount of binding agent and its composition have been judged more or less correctly. The corresponding norm for a GBS/LD1 binding agent would be 0.5-0.6 MPa and that for cement approx. 1.8 MPa.

6.6.6 Dependences between test results

The results of the construction experiments conducted here are presented in the form of broad order-of-magnitude guidelines in Table 15. Looking at the overall figures for the road, it would seem that a sufficient bearing capacity can be achieved at relatively low compression strengths. The values obtained in the preliminary tests and the laboratory tests on the actual structures were relatively similar, and it is significant that those for the GBS/LD1 binding agent was only about 1.25 MPa and that for GBS/CEM about 2.2 MPa. This strength level would nevertheless appear to have been sufficient in this case. The results of the dynamic triaxial tests performed on material stabilized in the laboratory were not especially informative relative to the dynamic moduli of elasticity measured in the test structures.

Table 15. Results of the construction experiments, in terms of approximate orders of magnitude.

	UCS, 91 days (MPa)		M_R	E_{STAB}	E_{TOT}
	Stabilized mass	Preliminary investigation	182 days (MPa)	(MPa)	(MPa)
T1/T2 (GBS/LD1)	1.25	1.3	490-650	3150/1950	500/450
T3 (GBS/CEM)	2.2	2.85	710-880	3150	475
T4 (CEM)	3.0	2.8	630-780	5450	495
T5 (GBS/CEM)	2.85	3.45	730-900	9600	495

6.6.7 Effect of bitumen on strength

Although the *in situ* stabilization technique employed here involved the crushing of the old surfacing and mixing it with the old base course to obtain the material to be stabilized, the presence of the old bitumen in this mixture would not seem to have had sufficient effect on the outcome of stabilization that this needed to be taken into account in particular when designing the structure or selecting the binding agents. It did reduce the resulting compression strength of the material (Figure 50), but it evidently also improved the functionality and durability of the structure, giving it somewhat more

flexibility and reducing its susceptibility to cracking. The present results thus suggest that there is no need to take account of the presence of old surfacing material when performing preliminary tests for the designing of a stabilized layer, but that the tests can be carried out perfectly well using aggregate alone.
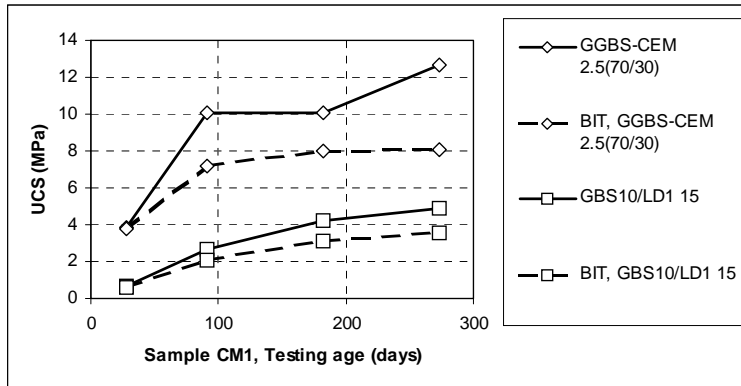

Fig. 50. Effect of material from an old bitumen surfacing on the compression strength of a stabilized aggregate (BIT = sample including old bituminous surfacing).

7 The commersialization procedure and products

7.1 The commersialization procedure and its initial values

The present work set out to follow the commercialization procedure depicted in Figure 17 in order to develop the blast-furnace and LD steel slags arising as by-products of the steel industry into binding agents to be used for stabilization purposes in road construction. In simplified terms, the procedure involved four stages (Figure 1). The point of departure was the *in situ* stabilization technique that is already in use with granulated blast-furnace slag, employing up to now almost exclusively cement as the activator. This method is able to make use of material from the old base course and bitumen surfacing, and is in this sense economical as it requires no deliveries of new aggregate to the site. The technique can, of course, be used to construct entirely new stretches of road, in which case the procedure is more straightforward, as the "pure" aggregate does not contain as many contaminants likely to interfere with stabilization.

The slags used here are by-products of actual iron and steel production that can be defined as raw materials as far as the processor is concerned. The problem in this definition is simply that the term "by-product" is not defined in law in any way, so that a definition has to be brought about by means of a separate statute or other interpretative procedure (Chapter 2).

The first stage of the commercialization procedure was devoted to defining the properties of the slags as raw materials for construction purposes, allowing conclusions to be reached regarding whether products should be developed for use in massive road structure layers or as admixtures or binding agents. As the properties of blast-furnace slag and LD steel slag had been investigated in numerous connections earlier (Chapter 4), it was possible to decide immediately upon embarking on this research that the products to be developed should be binding agents.

The second stage then involved investigations into their binding properties, in order to determine whether one or more of the raw materials available could be commercialized as such or after further processing and in what mixtures and what proportions they could best serve as binding agents. It was also necessary to recognize that the best binding agent would not necessary be the one with the highest reactivity, but that attention would

also have to be paid to the requirements laid down regarding the structures to be stabilized and to economic factors such as the price of the new product relative to existing ones. Initial data on the properties of the materials as binding agents were available from earlier research (sections 4.1, 4.2 and 4.3).

The binding agents and mixtures obtained as the results of the second stage in the work were then submitted to stabilization tests in the laboratory at the third stage (sections 6.2, 6.3 and 6.4), after which the fourth and last stage consisted of the construction and monitoring of a test road (section 6.5). It was also essential that the third and fourth stages should allow opinions to be put forward on what facts in particular should be borne in mind when using the new products (section 6.6).

The end result of the work was thus planned to be a set of products comprising 1) binding agents, 2) a road structure improved by stabilization with these products as binding agents, and 3) a procedure for designing the improvement of a road structure using these techniques. The products will be described in sections 7.2, 7.3 and 7.4 below.

7.2 Slag-based binding agents

The one entirely new binding agent developed in this work is LD steel slag-activated granulated blast-furnace slag, while further information was also acquired on the mixture of ground blast-furnace slag and cement, which has previously been used very little. Cement-activated granulated blast-furnace slag and cement alone were taken as reference materials, although a new basic formula for the use of the former was also developed here and used as a starting point in the design procedure. LD slag-activated granulated blast-furnace slag represents a type of binding agent which offers in principle an infinity of variations obtained by altering the proportions of the ingredients, and the same is also true of cement-activated granulated blast-furnace slag and the mixture of ground blast-furnace slag and cement. Boundary values can nevertheless be defined for all these combinations, providing a framework within which it is reasonable to attempt to design structures:

GBS/LD1:
- granulated blast-furnace slag 4-10%
- LD1 5-15%
- made up to 100% with aggregate
GGBS/CEM:
- binding agent content 1.5...~5% of dry weight of aggregate
- in the proportions GGBS/CEM 90%/10%...10%/90%
- preferable to maximize GGBS content in order to reduce binding agent costs
GBS/CEM:
- granulated blast-furnace slag 4-10%
- cement 1-2%
- preferable to maximize granulated blast-furnace slag content in order to reduce binding agent costs

The quantities stated above refer to the amounts of binding agent obtained in

calculations that take into account the thickness of the layer to be stabilized and the dry density of the aggregate. The basic formula for the binding agents for use in the designing of structures are:

- GBS10/CEM1
- GBS5/LD1 10
- GGBS-CEM 3(70/30)

These basic formula may be regarded as starting points upon which to base a stabilizability classification for use in adjusting the amount and composition of the binding agent to optimal levels for the material concerned.

7.3 A road structure improved by stabilization

In situ stabilization and simultaneous resurfacing, with drainage improvements where necessary, has proved a relatively cost-effective means of rehabilitating an existing road structure. In this case the lower courses are not directly affected, although their functioning is influenced by the provision of a new rigid or semi-rigid base course, which evens out the loading on the lower parts of the structure. This method is considerably more economical than renewing the whole road structure, even though it does not lengthen the service life of the road to the same extent. The problem with such a procedure is that nothing is done to lessen the frost susceptibility of the substructure, other than possibly by reducing the risk of frost damage through improved drainage. No opportunity remains to influence frost susceptibility through the choice of material in the substructure, although the stresses caused by frost heave are ameliorated somewhat by the effect of the rigid or semi-rigid stabilized layer in evening out traffic loads, and it is also possible to install a metal grid in the stabilized layer to reduce frost stress further.

One advantage of the use of granulated blast-furnace slag as a binding agent in stabilization relative to cement, for example, is that it is capable of re-binding at points where cracks occur, which will extend the service life of the stabilized layer somewhat relative to cement stabilization. Also, when activated with LD slag, the granulated blast-furnace slag is present in a large quantity in proportion to the aggregate, so that the resulting stabilization is more even than with finely ground binding agents, which are used in small quantities and give much higher compression strengths.

In spite of the fact that the binding agents tested here function and react in quite different ways in some cases, the improvement achieved in the bearing capacity of the experimental road was similar throughout (Table 16), the E_{TOT} -moduli describing this bearing capacity settling in the range 475...500 MPa in all the test stretches a year after stabilization. This was achieved by dint of structural decisions and choices of binding agents that took account of the characteristics of each agent separately. It is also significant that although the bearing capacities were of the same order of magnitude throughout, the E-moduli of the stabilized layers themselves varied greatly, within the range 1950...9600 MPa, and the same was true of the compression strengths of the stabilized material at an age of 91 days, which varied in the range 1.25-3.0 MPa depending on the binding agent used.

Table 16. Mean strengths and bearing capacities of the road structures improved by stabilization.

	UCS, 91 days (MPa) Stabilized mass	E_{STAB} -modulus (MPa)	E_{TOT} -modulus (MPa)
T1/T2 (GBS/LD1)	1.25	3150/1950	500/450
T3 (GBS/CEM)	2.2	3150	475
T4 (CEM)	3.0	5450	495
T5 (GBS/CEM)	2.85	9600	495

7.4 The design procedure

The design of a road structure stabilized with slag-based binding agents is assumed to proceed according to the stages set out in Figure 51.
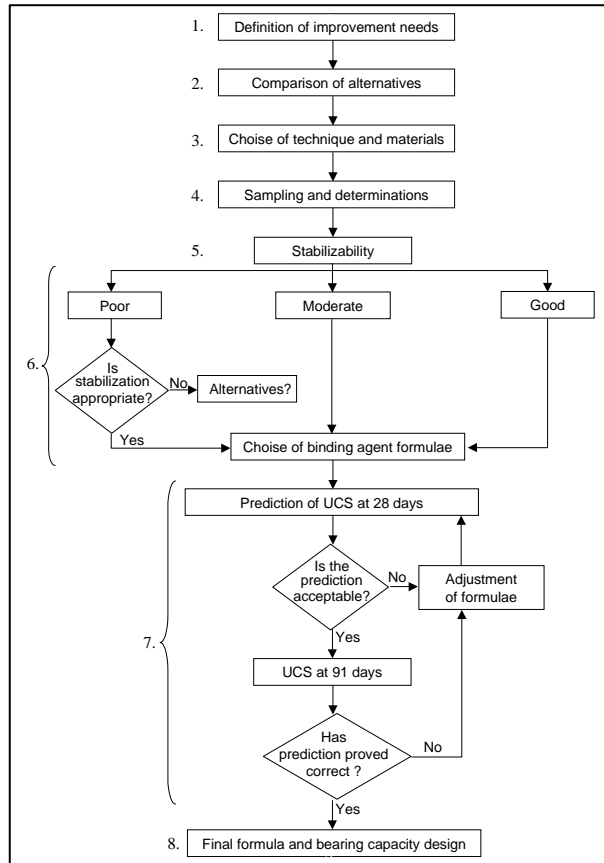

Fig. 51. Procedure for the design of a road structure stabilized with slag-based binding agents.

1. Definition of improvement needs

An attempt should be made to produce calculations that take into account the traffic load on the road structure for the whole of its service life. It is also important at this stage to obtain information on the materials contained in the existing structure, including the thicknesses of the layers and the grain sizes and moisture content of their materials and the material composition, grain size distribution and moisture content of the subgrade. FWD measurements will have to be carried out for design purposes and a survey will have to be made of the availability of any materials to be added to the structure. It is worthwhile when taking samples at this stage to bear in mind what samples will in any case be needed for the determinations to be made at stage 4.

2. Comparison of alternatives

The second stage involves comparing the alternative means available for improving the road structure. This will include definition of the targets to be set for the improvement work (improvement of bearing capacity and/or reduction of frost susceptibility) and also an assessment of the materials available, construction conditions, direct costs of the work and service level envisaged for the road for the next 20 years, for instance, in order to define the maintenance costs. Possible boundary conditions affecting the work, such as land use designations and environmental protection should similarly be looked into at this stage in the procedure.

3. Choice of technique and materials

It is decided at this stage what technique is to be adopted for improving the road structure, i.e. whether stabilization will take place *in situ* or at a mixing plant. Preliminary decisions should also be made regarding the choice of binding agents.

4. Sampling and determinations

The grain-size distribution of the material to be stabilized should be determined by washing and sieving, the maximum dry density and optimum water content by Proctor tests, the pH of the <2 mm fraction in a water suspension, the humus classification with the NaOH test and the humus content of the fines by loss on ignition. It is recommended that the optimum water content and maximum dry density of the whole mixture to be stabilized should be determined in the actual stabilizability tests.

5. Stabilizability classification

The indices required for the stabilizability classification of the material, as set out in Table 17, should be defined at this stage. Each stabilization classification should be aimed at achieving the compression strengths listed in Table 18 when using the basic formulae.

Table 17. Stabilizability classification required for determination of the binding agent formula (Mäkikyrö 1995).

Contributory factor	Boundary values		
	Well stabilizable	Moderately stabilizable	Poorly stabilizable
d_{40} (mm)	> 1.0	0.5 – 1.0	< 0.5
d_{50} (mm)	> 1.75	0.75 – 1.75	< 0.75
Humus content of fines (%)	< 1.8	1.8 – 3.0	> 3.0
Improved humus classification*	0 – IV	III – V	V
pH of <2 mm fraction	> 7.25	6.25 – 7.25	< 6.25
* Class V = black solutions			

Table 18. Results achieved in the uniaxial compression strength tests for the basic binding agent formulae when used with the given stabilizability classes (Mäkikyrö 1995). Figures for the formula GBS5/LD1 10 have been added on the basis of the present tests.

Binding receipt	Uniaxial compression strength at 91 days, MPa		
	Well stabilizable	Moderately stabilizable	Poorly stabilizable
GBS10/CEM1	> 2.8	0.7-2.8	< 0.7
GGBS-CEM 3(70/30)	> 3.2	0.4-3.2	< 0.4
GBS5/LD1 10	> 2.8	0.7-2.8	< 0.7

6. Choice of binding agent formulae

The following points should be borne in mind when choosing the binding agent:

- Two preliminary formulae should always be chosen for each binding agent, in order to be able to consider the amount of binder and the proportions of its components objectively.
- The highest strengths will probably be obtained with a GGBS/CEM binding agent, but if the subgrade is of poor quality and susceptible to frost, the coarse binding agents GBS/CEM and GBS/LD1 may be better, as they provide better resistance to cracking, largely on account of the re-binding properties of granulated blast-furnace slag. One requirement in the case of the coarser binding agents, however, is that they should give the material the necessary strength.
- In spite of the larger quantities required, the coarser-grained binding agents are frequently cheaper to use than GGBS/CEM, especially at sites close to the location of their production.
- If the organic matter content of the material to be stabilized means that the stabilizability indices are close to the lower limit for the moderately stabilizable class or the minimum values for the well stabilizable class, an increase in strength can probably be brought about by increasing the quantity of activator while maintaining that of the blast-furnace slag component constant.

If the aggregate that is intended to be stabilized proves to be poorly stabilizable it is necessary to reassess whether it is technically or economically advisable to attempt stabilization. If it can be shown by means of appropriate calculations that stabilization is not a viable proposition, some other means of improvement must be considered, but if

the calculations lead to the conclusion that stabilization can still be used, there remains two alternatives: 1) to raise the quality of the material to be stabilized by mixing it with "pure" aggregate imported from elsewhere, or 2) to exchange the whole mass of old aggregate material for new. In both cases the determinations performed at stage 4 will have to be repeated for the new material or mixture. It frequently transpires in such cases, however, that the whole advisability of stabilization has to be reconsidered, for although it might be possible to produce a good bearing course by means of stabilization, this would not be sufficient to compensate for other weaknesses in the road structure.

The preliminary binding agent formulae chosen for a poorly stabilizable aggregate should aim at maximum compression strength (see section 7.2) or close to this:
-   GBS10/CEM2 and GBS10/CEM1.5
-   GBS10/LD1 15 and GBS10/LD1 10
-   GGBS-CEM 5(70/30) and GGBS-CEM 5 (50/50)

If the stabilizability classification indices for a material are close to the lower end of the moderately stabilizable class, the choice should rest on the more active of the coarse-grained binding agents:
-   GBS10/CEM1.5 and GBS5/CEM2
-   GBS7.5/LD1 15 and GBS7.5/LD1 10

If the stabilizability classification indices for a material are close to the upper end of the moderately stabilizable class, the choice should be between the following coarse-grained binding agents:
-   GBS7.5/CEM1.5 and GBS5/CEM1.5
-   GBS7.5/LD1 10 and GBS5/LD1 10

If fine-grained binding agents are preferred, the choice in both of the above instances should fall on:
-   GGBS-CEM 3(70/30) and GGBS-CEM 3(50/50)

If the aggregate is well stabilizable, the basic formula for any of the binding agents can be accepted, together with an alternative that shows lower reactivity:
-   GBS10/CEM1 and GBS5/CEM1
-   GBS5/LD1 10 and GBS5/LD1 5
-   GGBS-CEM 3(70/30) and GGBS-CEM 2(70/30)

7. Prediction of UCS at 28 days

The probable compression strength attainable with each preliminarily chosen binding agent can be calculated from the value at 28 days by means of the formulae listed in section 6.6.5 above:

GBS/CEM: $UCS_{91} = 1.2980 \times (UCS_{28})^{1.0374}$
GBS/LD1: $UCS_{91} = 2.2392 \times (UCS_{28})^{1.2554}$
GGBS/CEM: $UCS_{91} = 1.3251 \times (UCS_{28})^{1.0682}$

In this way a decision can be reached on the basis of the 28-day strengths achieved with the preliminary choices of binding agent as to whether the eventual choice will be an agent of the GBS/CEM, GBS/LD1 or GGBS-CEM type. If a poorly stabilizable material has shown no signs of an increase in strength by 28 days, it may be concluded that stabilization with a slag-based binding agent is not technically or economically feasible.

If, on the other hand, the predicted strength at 91 days is too high, it will be necessary to test new specimens selected on the following principles:
- It may be sufficient to reduce the amount of activator used with a coarse-grained binding agent by 0.2-0.3%-units for CEM and 2-3%-units for LD1. In the case of a GGBS-CEM formula, the proportion of ground blast-furnace slag can be reduced to <70%.
- Alternatively, the amount of activator used with a coarse-grained binding agent can be reduced by 0.1-0.2 %-units for CEM and 1-2 %-units for LD1, with a simultaneous reduction of 1-2%-units in GBS. Similarly, the amount of GGBS-CEM binding agent can be reduced by 0.3-0.5%-units and the proportion of ground blast-furnace slag to <70%.

If the predicted strengths at 91 days are suitable, one should wait to see how the actual strengths turn out, while if they are too low, additions should be made in the case of the coarse-grained binding agents that are of the same magnitude as the reductions made for excessively high compression strengths, while for the GGBS-CEM formulae the proportions of the components should be kept the same but the amount reduced by 0.3-0.5%-units.

The actual 91-day compression strengths of the preliminary specimens and any additional specimens required should be within the following ranges:
- GBS/CEM: 2.5-3.0 MPa
- GBS/LD1:   1.2-2.5 MPa
- GGBS-CEM: 3.0-3.5 MPa

If these strengths are not produced, it will be necessary to adjust the formulae still further until the desired result is obtained. As observed in the course of this research, the compression strength does not have to be the highest achievable, as a satisfactory bearing capacity can be produced with only a moderate strength result for the stabilized layer. This is particularly true when using a GBS/LD1 binding agent.

8. Final formula and bearing capacity design

When the final binding agent formula has been selected for stabilization purposes, the bearing capacity design can be carried out by reference to the graphs in Figures 52-54, which are constructed on the basis of the mean steady-state E-moduli obtained in the experimental structures after an interval of one year, with a deduction of 0.5×standard deviation.
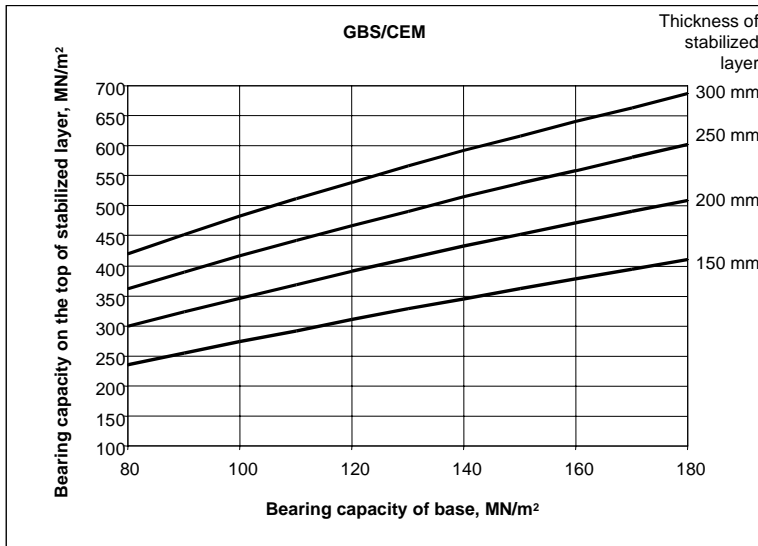
Fig. 52. Bearing capacity design values for road structures stabilized with a GBS/CEM binding agent.
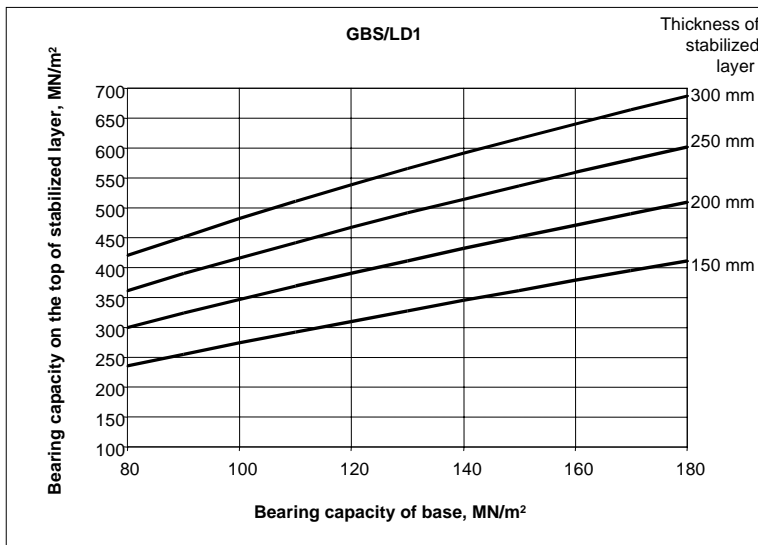

Fig. 53. Bearing capacity design values for road structures stabilized with a GBS/LD1 binding agent.
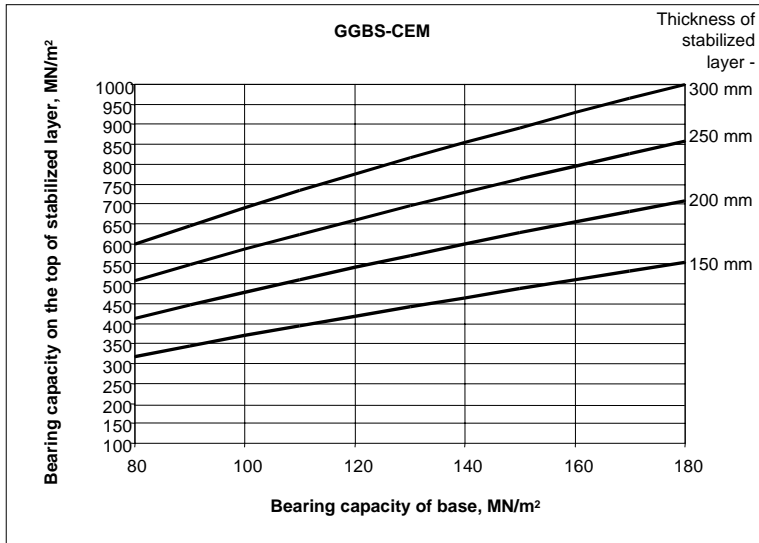
Fig. 54. Bearing capacity design values for road structures stabilized with a GGBS/CEM binding agent.

8 Summary

The aim of this work was to define a procedure for the commercialization of blast-furnace and LD steel slags arising as by-products of the steelmaking industry and to develop products in accordance with this procedure. In simplified form, the procedure consists of four stages: 1) determination of the properties of the materials, 2) determination of the binding properties of the products to be commercialized, 3) determination of the interactions between these binding agents and the aggregate to be stabilized, and 4) construction of a test road. The binding properties were investigated primarily by engineering means, i.e. on the basis of strength measurements, without any detailed inquiry into the mechanisms of the binding reactions involved. The point of departure for this was the existing technique of *in situ* stabilization with granulated blast-furnace slag, in which the activator has up to now almost exclusively been cement. The principal point of interest in the present work as far as the binding agents were concerned lay in the possibilities for commercializing LD steel slag as an alternative activator. A wide range of proportions of both binding agents and activators were tested here, although the total number of mixtures had to be restricted, for two principal reasons. Firstly, the testing of a large range of materials would have led to an unreasonable expansion in the research programme, and secondly, the range adopted was adequate for determining the optimum proportions for each mixture of agents. In practical stabilization and design projects it is necessary to be content with a small number of basic formulae in terms of which the eventual binding agent and its quantity and proportions must be determined.

The principal problem to emerge as far as commercialization is concerned involves the official category of the initial material, on which a variety of opinions have been put forward depending on whether the matter is viewed from the perspective of the beneficiary or the licensing authority. In other words, it is impossible to define precisely whether the material to be commercialized here should be classified as waste, a by-product, a raw material or something else. In this study slags are classified as industrial by-products (raw materials) which have been developed into products. The present discussion ended up with three products or groups of products: 1) slag-based binding agents, 2) an improved road structure achieved by stabilization with a binding agent, and 3) a dimensioning procedure for the design of a stabilized structure.

Attention was also paid in the course of implementing the research to the practical applications of the results, i.e. efforts were made to simplify the methods. This was achieved in practice by restricting the laboratory testing mainly to uniaxial tests of compression strength, the results of which were employed directly to design a sequence of five test road structures for the analysis of bearing capacity behaviour. One drawback with the compression test that deserves mentioning is that it tells us scarcely anything about the actual functioning of the structure. The method was nevertheless chosen because the test is simple and quick to perform, it is easily repeatable and it does not require any complex equipment or arrangements. In addition, the results are unambiguous and easy to interpret by comparison with the dynamic triaxial test, for instance. Another reason for the choice of this test was that when defining design parameters for a road structure to be stabilized, especially with new constructions, the monitoring of compression strengths and test structures almost without exception allows results to be extracted that are of a standard that they can form a basis for dimensioning.

The research was carried out in stages, setting out from the results of the author's licentiate thesis. Care was also taken to examine these results in such a way as to ensure that they are internationally as applicable as possible. The design procedure is admittedly based on existing Finnish practices, but it is also capable of being used equally well elsewhere.

There are numerous factors that affect the outcome of stabilization, the most important of which are the grain size and organic content of the aggregate to be stabilized. Determinations of these two factors and certain indices calculated from them go a long way towards enabling the order of magnitude of the eventual stabilization result to be estimated. It would be unreasonable to attempt to assess all the relevant factors in the context of a single project, for reasons of expense, and in any case the central problem affecting stabilization emerged very clearly in the present work: that the results achieved cannot be related directly to clearly definable contributory factors associated with the material to be stabilized. We are probably moving here in a borderline area in which it may be necessary to consider such factors as the sulphates, sulphites and other aspects of the composition of the material that are more difficult to assess. It is necessary to be acquainted with the "character" of the material in order to achieve a good final result, as it is exceedingly difficult to construct sets of competent graphs and tables that would enable design work to proceed reliably. Generalization of the results at an international level will also be affected by the fact that all the aggregates employed in this work were of Finnish origin, and that these may differ markedly from the materials available elsewhere. This does not detract from the significance of the work, however, as the same regularities hold good everywhere and a stabilization project will always call for preliminary tests in order to determine the best approach.

The binding agent components considered in the early stages of this work were granulated blast-furnace slag as such and in partially ground and ground form and LD steel slag in crushed (0-3 mm), partially ground and ground form, but as the work progressed the partially ground LD slag and the ground LD slag were dropped out, as the technical advantages gained from them were not such as would warrant the continuation of research into their use. In other words, the costs of pre-grinding and grinding were too high in relation to the benefit gained. It was observed at an early stage, on the basis of the paste tests, that there was little point in experimenting any further with partially ground

LD steel slag, and the results of the stabilization of pure crushed rock samples with ground LD slag as an activator and ground blast-furnace slag as such showed that these functioned sufficiently poorly that there was little need to pursue the investigations any further. The results suggested that ground blast-furnace slag always requires an activator in order to function as a binding agent. The pre-grinding of granulated blast-furnace slag, on the other hand, produced a considerable increase in its reactivity while at the same time preserving the good properties of the starting material, namely its coarse-grained texture and its re-binding capacity. Further tests were nevertheless abandoned because pre-grinding would have raised the price of the product to a level at which it would have ceased to be competitive.

A number of potential products that could have had applications in binding technology had to be rejected in the course of this work, in many cases precisely for reasons connected with their commercialization. This was particularly true of partially ground granulated blast-furnace slag, for example, which would have been a much better binding agent component than unground granulated blast-furnace slag, as it is more reactive and only half the quantity would have been required to fulfil the same purpose, but its production costs had too great an effect on its price to make it competitive. It should be remembered, however, that the products rejected here could well emerge as economically viable at some time in the future, e.g. as a consequence of changes in environmental taxation. There was no intention in the present work of examining products purely because of theoretical prospects applying to some point in the distant future, but the potential has been created here for their commercialization, so that the threshold for taking up their case later on will not be so high.

The main new slag-based binding agent to be developed here was LD steel slag-activated granulated blast-furnace slag, which is a quite distinct material from cement or cement-activated ground blast-furnace slag. Its behaviour most closely resembles that of cement-activated granulated blast-furnace slag, and for this reason many of the same strength criteria can be applied to its use for activating granulated blast-furnace slag as with cement as an activator. The resulting binding agent nevertheless reacts more slowly and is required to be present in larger quantities. It is significant, in fact, that the amount of LD slag needed as an activator is greater than that of the granulated blast-furnace slag itself as a binding agent. Although LD slag may reach proportions of as much as 15% of the material to be stabilized, its volume instability does not constitute a problem, because its amount is still not so large relative to the material as a whole and the binding itself reduces the expansion effect. In addition, part of the free lime contained in it has time to react during the stabilization work, before surfacing of the road. Some cracking can occur in laboratory specimens, however, as observed here in the preliminary tests and when examining their results.

The compression strengths achieved with LD steel slag-activated granulated blast-furnace slag are not especially high, but this may be compensated for by greater structural depths and larger amounts of binding agent, which is beneficial as far as the functioning of the structure is concerned, as the stabilization result is more even than with other binding agents, on account of the large quantities used. Also, the detrimental effects of organic matter in the aggregate on the result of stabilization are reduced, as this organic matter makes up a smaller proportion of the total material. On the other hand, the larger amount of binding agent required naturally increases transport costs, although this

is offset to some extent by the lower prices of the materials.

The coarse binding agents, i.e. cement-activated and LD slag-activated granulated blast-furnace slag, differ considerably in their mode of action from the fine-ground materials, i.e. cement and cement-activated ground blast-furnace slag, the latter being far more reactive, so that even small discrepancies in the amounts present can lead to major differences in the compression strength achieved. The coarse binding agents are more problematic as far as the bearing capacity design of a road structure is concerned, as the eventual strength level only begins to become apparent after about three months, but the results suggest that they may be "safer" alternatives, as provided that stabilization is successful, the resulting structure is not so susceptible to damage from frost or traffic loading. In addition, the re-binding property of granulated blast-furnace slag is a significant advantage, especially in areas affected by seasonal frosts. These materials also entail certain advantages during construction work, as they allow a longer working time and a layer stabilized with them will withstand traffic loading, even in an unbound state, better than will the equivalent structure produced with finely ground binding agents. The coarse binding agents, and above all granulated blast-furnace slag, will improve the grain-size distribution of the aggregate as they are, even if no binding occurs, which means that if for some reason no increment in strength is achieved immediately, the fault is not so disastrous as with a finely ground binding agent.

It would seem from the present research that, at least in some cases, there is a threshold value in the amount of activator to be added, so that if this amount remains below the threshold no stabilization will take place, while if the threshold is exceeded stabilization will begin and very little further activator will need to be added in order to achieve a greater final strength. It is thus important to be able to optimize the proportions of the binding agent components, especially with cement activation, when an additional 0.1-0.2%-units is usually sufficient. When LD slag is used, the addition naturally has to be very much larger, about 1-2%-units.

The road construction experiment was highly successful, the only weak point being test structure 2, in which the actual amount of binding agent used was considerably lower than planned. The thicknesses of the stabilized layers produced with the coarse binding agents were greater than those produced with the finely ground agents, so that it may be claimed that their thickness made up for their poorer strength values. In general, however, it may be said that in spite of the fact that the binding agents used differed markedly in their reactivity and mode of action, the improvements in bearing capacity achieved in the various stretches of road were very similar, the E-modulus throughout having settled within the range 475...500 MPa a year after the improvement work. This result was achieved by taking the "character" of each binding agent into account in the selection procedure and when deciding on the structure of the road. On the other hand, although all the stretches of road had a similar bearing capacity when defined for their whole structure, the E-moduli for the stabilized layers themselves varied greatly, over the range 1950...9600 MPa. This means that the overall situation must have been influenced by the subgrade and untreated lower structural layers. This reminds us that the initial strength and the bearing capacity achievable on the basis of it may assume a decisive role from the point of view of the design and construction of a road improvement, and that in this respect the strengths achievable with finely-ground binding agents are superior to those achievable with coarse-grained ones. The slow reactions of the latter materials may

prove a limitation on their use in projects with a tight timetable.

The compression strengths of the stabilized material at 91 days also varied, in the range 1.25...3.0 MPa depending on the binding agent used. The uniaxial compression strength required of the stabilized material need not necessarily be especially high when working with coarse-grained binding agents, and even the target of 3.0...3.5 MPa would seem disproportionately high, as a suitable bearing capacity was obtained in the test road constructed here with considerably lower strength values.

Comparison of the test structures in which cement and LD slag were used for activating granulated blast-furnace slag suggests that the outcome was very much the same in both cases. The main difference was that the structure had to be of greater thickness in the case of LD slag activation in order to make up for the poorer strength increment. The structures were nevertheless technically very similar, in other words the main aim, of replacing cement with a slag product, was fulfilled successfully.

The planning of the work of improving a road structure by means of stabilization, and likewise the design procedure, did not differ from those employed in connection with traditional binding agents. The definition of the need for improvement, comparison of alternative methods, choice of technique and materials and sampling for the necessary preliminary determinations all proceeded according to existing methods and routines. One clear departure from the usual design procedure was caused by the long reaction times of the slag-based binding agents, which are therefore matters of relevance for the choice of binding agent and proportion of activator. In order to reduce the time required, the number of preliminary tests and the costs of the design stage, models were constructed here for predicting the trend in the strength of the bound course. The bearing capacity design stage that followed the final choice of binding agent did not depart significantly from the accustomed procedure, and although the design procedure outlined here was used in connection with a project to improve the bearing capacity of an existing road structure, it can be applied as such to a new construction project as well, the only significant differences being that the initial values are easier to define and it is easier to choose the binding agent, as the material to be stabilized is of better quality than that obtainable from an old road structure and the outcome of stabilization more predictable.

9 Conclusions

The following conclusions can be reached on the basis of the research reported here:

Slag-based binding agents
1. The cement used as an activator in granulated blast-furnace slag stabilization can be replaced with LD slag, a by-product of steel manufacturing, but the following points have to be noted:
   - Although the granulated blast-furnace slag/LD slag binding agent reacts more slowly than granulated blast-furnace slag/cement, the compressive strength at 91 days can be used as a formula selection criterion in both cases.
   - A greater amount of LD slag is generally used as an activator than of granulated blast-furnace slag. A suitable amount is 5-15% of the dry weight of the material to be stabilized, and usually 5-10% will be sufficient.
   - The compressive strength at 91 days required with a granulated blast-furnace slag/LD slag binding agent is only about a half of that for granulated blast-furnace slag/cement.
   - A larger amount of granulated blast-furnace slag/LD slag binding agent is required than of granulated blast-furnace slag /cement, because LD slag is less efficient and is not in itself bound to anything like the same extent as cement.
2. Partially ground granulated blast-furnace slag (PGBS) is a better binding agent than granulated blast-furnace slag (GBS), because it is more reactive, so that only a half of the quantity is needed. PGBS is nevertheless so coarse after the first crushing that its properties at this stage are similar to those of granulated blast-furnace slag.
3. All slag-based binding agents have good strength properties in stabilization. The special characteristics of each binding agent have to be borne in mind at the time of selection.

An improved road structure achieved by stabilization with a binding agent (case)
1. The poorer strength achievable with a granulated blast-furnace slag/LD slag binding agent can be compensated for by increasing the thickness of the stabilized layer.
2. An equally good improvement in bearing capacity can be achieved with a structure stabilized with a granulated blast-furnace slag /LD slag binding agent as with

granulated blast-furnace slag /cement, by virtue of the thicker stabilization layer and greater quantity of binding agent used.

<u>A dimensioning procedure for the design of a stabilized structure</u>
1. The road structure dimensioning procedure to be followed when using slag-based binding agents does not depart in any essential manner from that required for "traditional" natural materials. The only clear difference is the slower reaction rate of slag-based binding agents, which can be estimated with reasonable accuracy on the basis of 28-day compressive strength tests. Dimensioning with all slag-based binding agents is possible on the basis of the compressive strength at 91 days.
2. The uniaxial compression test is a sufficient indicator of the best binding agent for use in a particular case and of the optimum quantity of this agent.

10 Prospects for further research

One case was examined here, that of an old road structure in which the bearing capacity was to be improved by *in situ* stabilization, and if one were interested in drawing up comprehensive methodological instructions and determining accurate boundary values, it would naturally be necessary in principle to carry out tests with all possible types of aggregate and all possible combinations of binding agents. Since such a task would be never-ending, we are obliged to accept broad generalizations in order to gain useful results in an economic manner. The obtaining of reliable bearing capacity dimensioning values would also call for the construction and monitoring of a number of test roads, the sites for which should be chosen in order to examine as wide a variety of conditions as possible. This would mean in practice, of course, that they should be located in different parts of the country, so that allowance could be made for meteorological effects, mainly frost conditions.

In order to develop the range of products and the design procedure further, it would be necessary to determine how large the LD slag component may be in the stabilized mixture before a risk emerges of the volume instability characteristic of this material causing damage to the structure. It is possible, for instance, that the proportion of slag could be so large that it would no longer be a matter of stabilization with a binding agent but of a mixture of materials, including slag, in which the crushed natural rock was just one relatively passive component. Although the LD slag serves as an activator in the binding agent used here, an attempt should be made to determine its binding action, i.e. how much of the resulting strength increment is attributable to the activator reaction, and the same should also be done for cement as an activator. Further development of the procedure would also call for a substantial number of additional experimental constructions and laboratory tests in order to formulate the strength prediction model more precisely.

As use can be made of existing materials at many stabilization sites, including the old bitumen surfacing, the influence of this bitumen material on the eventual structure should also be studied and its impacts on the strength of the bound layer, the rate of stabilization and the functioning of the road structure should be assessed. At least in theory, it would seem that bitumen from the old surfacing should increase the "flexibility" of the stabilized structure and thereby reduce the incidence of shrinking cracks.

As a large number of test structures would be needed to investigate the aspects mentioned above, it would be most reasonable to carry out such tests in connection with actual construction projects. The results presented here could then be applied in practice and the monitoring would provide a means of adjusting and updating the binding agent formulae and design procedure. Future projects could also be used for testing partially ground granulated blast-furnace slag in an experimental context with a view to possible future commercialization. Attention should also be paid to the use of LD slag, e.g. of grain size 0-10 mm, as an activator. If the coarser LD slag proved to function in the same manner as the 0-3 mm material and finer fractions tested here, it could mean savings in material costs, since the coarser grade is cheaper to produce. Similarly, the mixing of blast-furnace slag and LD slag in the factory prior to delivery would also entail savings in binder agent distribution costs at the stabilization site.

The effects of the slow reaction rate of slags and the expansion of LD slag on the experimental results should also be investigated. These are important matters if one wishes to use preliminary laboratory tests to predict the behaviour of the stabilized material in the structure. This would also improve the comparability of tests performed in different laboratories, although some attention has already been paid to this problem (Heaton & Cao 1998, Stock *et al.* 1996). The main emphasis in the present laboratory tests was on the uniaxial compressive tests, and it would be useful in the future to elucidate what other tests could be used to help define the stabilization formulae. Further work should also be done on the relations of the laboratory findings to the results obtained in actual structures produced in practise.

If it is really intended to make full use of Life Cycle Assessment and Life Cycle Cost Analysis in future comparisons of alternative means of road construction, clearly expressed methods for developing these should be expounded. This is not a research problem as such, however, but above all a matter of taking the necessary decisions. In other words, it has to be decided how to put prices on environmental effects. These are already been spoken about and attemps are being made to use them as criteria of a kind when comparing alternatives, but in practice the lack of any concrete pricing means that eventually they scarcely affect the resulting decisions at all. Then effects of harmonization of EU and national standards will also pose a challenge for future research, on account of the differences between countries in the conditions under which roads are constructed.

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Material information

OM1: Old road material with clay-gravel surfacing material, Mt 858, Pudasjärvi
(2. phase)
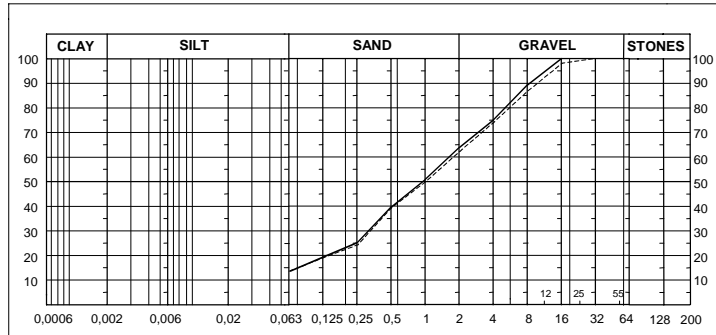
			Amount of fraction			Stabilization Class	
γ_{dmax} =	22.34	kN/m^3					
w_{opt} =	5.8	%	< 0.063 mm =	13.1	%	Grain size distrib.:	moder
d_{40} =	0.58	mm	0.063 – 2.0 mm =	49.7	%	Organic matter:	moder
d_{50} =	0.95	mm	2.0 – 16.0 mm =	37.2	%	As a whole:	moder
$pH_{<2,0\ mm}$ =	6.0						
MHC =	V		Loss on ignition =	1.80	%		

OM2: Old road material with clay-gravel surfacing material, Pt 18960, Pudasjärvi
(2. phase)
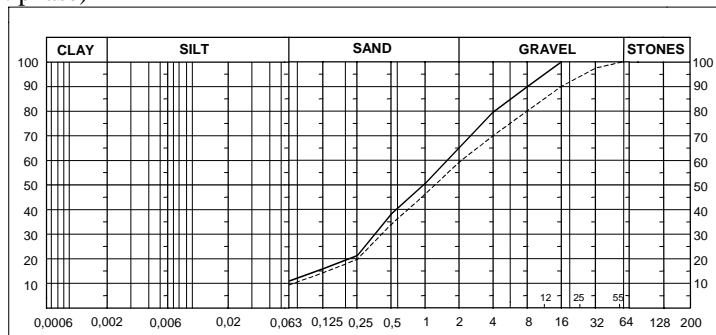
			Amount of fraction			Stabilization Class	
γ_{dmax} =	22.03	kN/m^3					
w_{opt} =	6.8	%	< 0.063 mm =	10.7	%	Grain size distrib.:	moder
d_{40} =	0.6	mm	0.063 – 2.0 mm =	54.9	%	Organic matter:	moder
d_{50} =	0.95	mm	2.0 – 16.0 mm =	34.4	%	As a whole:	moder
$pH_{<2,0\ mm}$ =	6.46						
MHC =	III		Loss on ignition =	1.79	%		

OM3: Old road material, Pt 18799, Ii (2. phase)
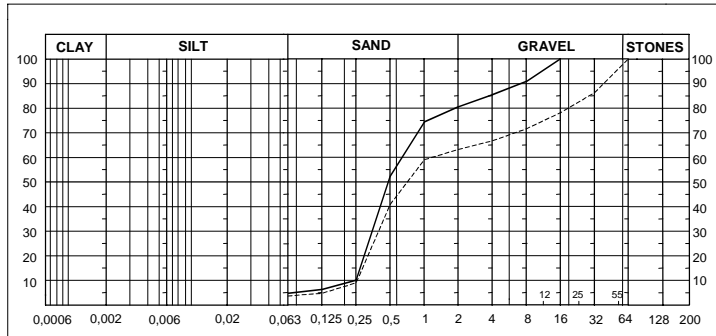
$\gamma_{dmax} =$ 19.72 kN/m^3 <u>Amount   of fraction</u>   <u>Stabilization Class</u>

$w_{opt} =$	9.0	%	< 0.063 mm =	4.7	%	Grain size distrib.: poor
$d_{40} =$	0.43	mm	0.063 – 2.0 mm =	75.3	%	Organic matter: poor
$d_{50} =$	0.49	mm	2.0 – 16.0 mm =	20.0	%	As a whole: poor
$pH_{<2,0\ mm} =$	5.04					
MHC =	V		Loss on ignition =	2.33	%	

OM4: Old road material, Pt 18772, Ii (2. phase)
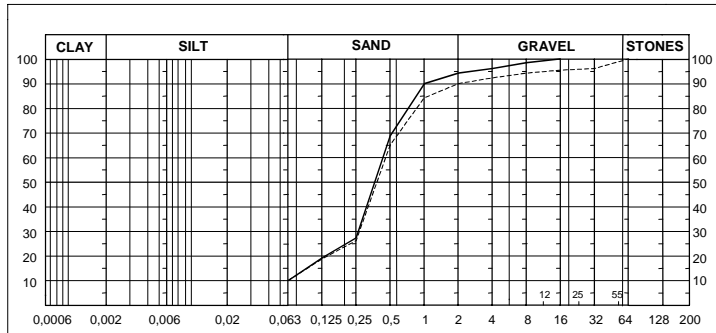
$\gamma_{dmax} =$ 19.89 kN/m^3 <u>Amount   of fraction</u>   <u>Stabilization Class</u>

$w_{opt} =$	6.4	%	< 0.063 mm =	9.9	%	Grain size distrib.: poor
$d_{40} =$	0.33	mm	0.063 – 2.0 mm =	84.1	%	Organic matter: poor
$d_{50} =$	0.38	mm	2.0 – 16.0 mm =	6.0	%	As a whole: poor
$pH_{<2,0\ mm} =$	6.04					
MHC =	V		Loss on ignition =	3.26	%	

OM5: Old road material, city road, Oulu (2. phase)

| | CLAY | SILT | SAND | GRAVEL | STONES |
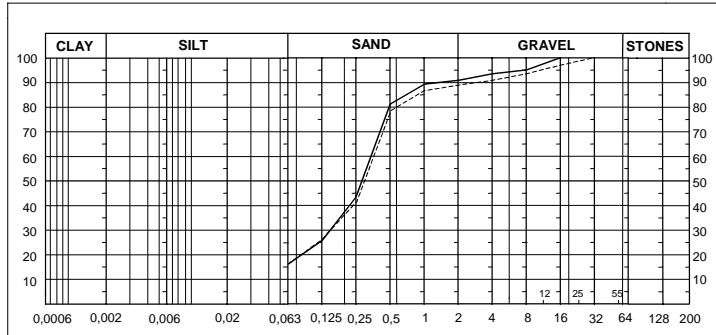
γ_{dmax} = 19.15 kN/m^3

<u>Amount of fraction</u>

Stabilization Class

w_{opt} =	10.1	%	< 0.063 mm =	16.2	%
d_{40} =	0.23	mm	0.063 – 2.0 mm =	74.6	%
d_{50} =	0.29	mm	2.0 – 16.0 mm =	9.2	%
$pH_{<2,0\ mm}$ =	4.69				
MHC =	V		Loss on ignition =	3.98	%

Grain size distrib.: poor
Organic matter: poor
As a whole: poor

OM6: Old road material with clay-gravel surfacing material, Pt 18658, Siikajoki (2. phase)

| | CLAY | SILT | SAND | GRAVEL | STONES |

γ_{dmax} = 21.53 kN/m^3

<u>Amount of fraction</u>

Stabilization Class

w_{opt} =	4.9	%	< 0.063 mm =	9.6	%
d_{40} =	0.49	mm	0.063 – 2.0 mm =	61.4	%
d_{50} =	0.85	mm	2.0 – 16.0 mm =	29.0	%
$pH_{<2,0\ mm}$ =	4.71				
MHC =	IV		Loss on ignition =	1.86	%

Grain size distrib.: moder
Organic matter: poor
As a whole: poor

OM7: Old road material with clay-gravel surfacing material, Pt 18564, Rantsila
(2. phase)

| | CLAY | SILT | SAND | GRAVEL | STONES | |
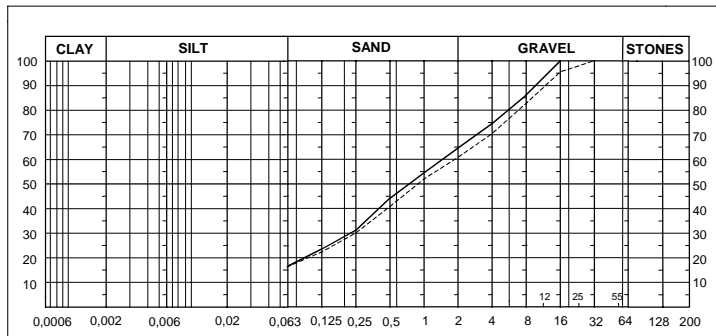
$\gamma_{dmax} = $ 22.29 kN/m^3 Amount of fraction   Stabilization Class

$w_{opt} = $	5.4	%	< 0.063 mm =	17.1	%	Grain size distrib.: moder
$d_{40} = $	0.43	mm	0.063 – 2.0 mm =	46.8	%	Organic matter: moder
$d_{50} = $	0.8	mm	2.0 – 16.0 mm =	36.1	%	As a whole: moder
$pH_{<2,0\ mm} = $	6.60					
MHC =	IV		Loss on ignition =	2.47	%	

OM8: Old road material, Mt 798, Haapavesi (2. phase)

| | CLAY | SILT | SAND | GRAVEL | STONES | |
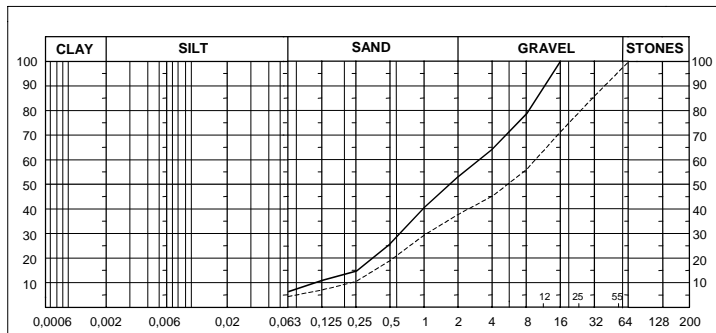
$\gamma_{dmax} = $ 21.27 kN/m^3 Amount of fraction   Stabilization Class

$w_{opt} = $	6.4	%	< 0.063 mm =	6.4	%	Grain size distrib.: moder
$d_{40} = $	0.98	mm	0.063 – 2.0 mm =	46.2	%	Organic matter: moder
$d_{50} = $	1.85	mm	2.0 – 16.0 mm =	47.4	%	As a whole: moder
$pH_{<2,0\ mm} = $	6.66					
MHC =	V		Loss on ignition =	2.64	%	

CG1: Crushed gravel from oil gravel manufacturing plant, Pudasjärvi (2. phase)
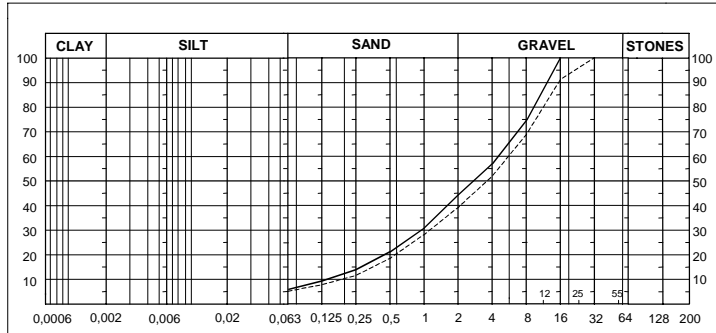
$\gamma_{dmax} =$	21.53	kN/m^3	Amount of fraction			Stabilization Class	
$w_{opt} =$	4.9	%	< 0.063 mm =	5.3	%	Grain size distrib.:	well
$d_{40} =$	1.75	mm	0.063 – 2.0 mm =	38.0	%	Organic matter:	moder
$d_{50} =$	3.0	mm	2.0 – 16.0 mm =	56.7	%	As a whole:	well
$pH_{<2,0\ mm} =$	6.23						
MHC =	II		Loss on ignition =	1.40	%		

CG2: Crushed gravel from oil gravel manufacturing plant, Siikajoki (4. phase)
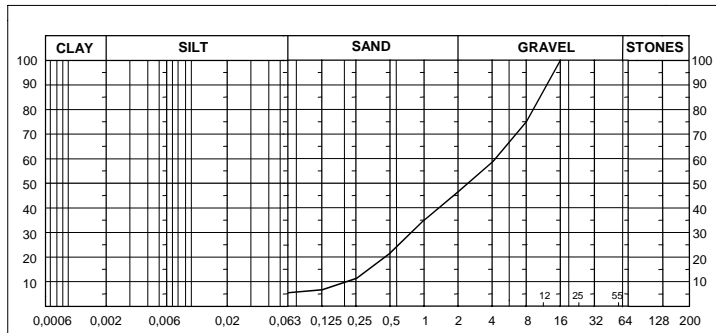
$\gamma_{dmax} =$	22.11	kN/m^3	Amount of fraction			Stabilization Class	
$w_{opt} =$	6.66	%	< 0.063 mm =	5.2	%	Grain size distrib.:	well
$d_{40} =$	1.4	mm	0.063 – 2.0 mm =	42.1	%	Organic matter:	well
$d_{50} =$	2.6	mm	2.0 – 16.0 mm =	52.7	%	As a whole:	well
$pH_{<2,0\ mm} =$	7.31						
MHC =	I		Loss on ignition =	1.50	%		

CR1: Crushed rock, Oulu (3 batches, 3. phase)
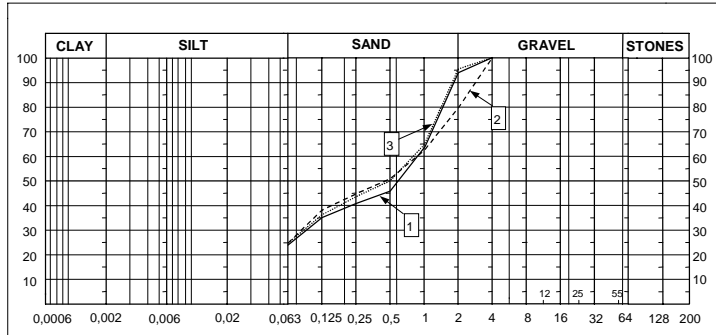
Batch 1	Batch 2	Batch 3
γ_{dmax} = 20.37 kN/m^3	γ_{dmax} = 20.52 kN/m^3	γ_{dmax} = 20.16 kN/m^3
w_{opt} = 8.03 %	w_{opt} = 7.06 %	w_{opt} = 7.84 %

CR2: Crushed rock, Oulu (3. phase)
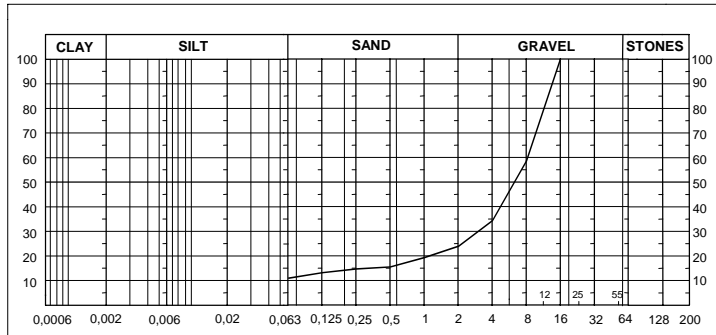
γ_{dmax} =   22.84   kN/m^3

w_{opt} =	6.51	%	Amount of fraction		Stabilization Class
			< 0.063 mm = 10.5 %		Grain size distrib.: well
d_{40} =	5.2	mm	0.063 – 2.0 mm = 13.2 %		Organic matter: well
d_{50} =	6.4	mm	2.0 – 16.0 mm = 76.3 %		As a whole: well
$pH_{<2,0\ mm}$ =	-				
MHC =	-		Loss on ignition = - %		

CR3: Crushed rock, Oulu (3. phase)
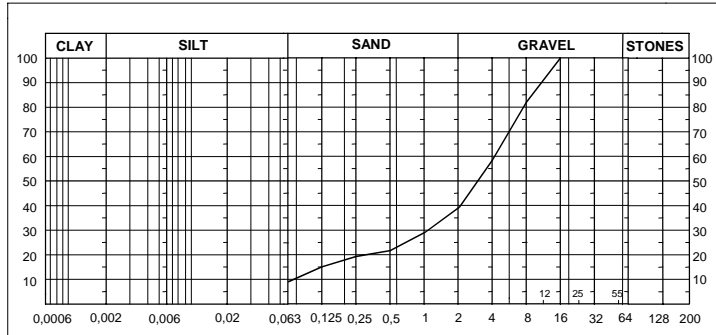
$\gamma_{dmax} =$	21.98	kN/m³	Amount of fraction			Stabilization Class	
$w_{opt} =$	6.91	%	< 0.063 mm =	9.0	%	Grain size distrib.:	well
$d_{40} =$	2.0	mm	0.063 – 2.0 mm =	30.1	%	Organic matter:	well
$d_{50} =$	3.2	mm	2.0 – 16.0 mm =	60.9	%	As a whole:	well
$pH_{<2,0\ mm} =$	-						
MHC =	-		Loss on ignition =	-	%		

CR4: Crushed rock, Oulu (4. phase)
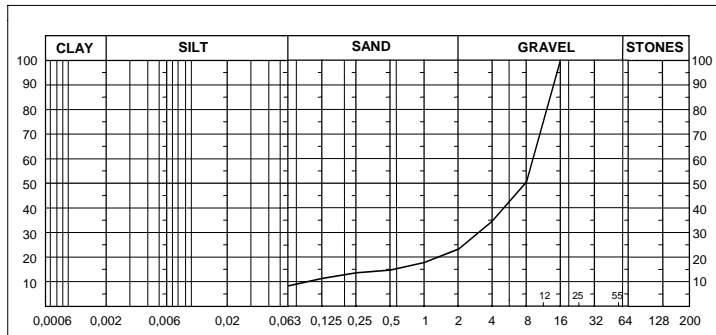
$\gamma_{dmax} =$	22.87	kN/m³	Amount of fraction			Stabilization Class	
$w_{opt} =$	6.92	%	< 0.063 mm =	8.0	%	Grain size distrib.:	well
$d_{40} =$	4.8	mm	0.063 – 2.0 mm =	14.9	%	Organic matter:	well
$d_{50} =$	6.0	mm	2.0 – 16.0 mm =	77.1	%	As a whole:	well
$pH_{<2,0\ mm} =$	7.26						
MHC =	-		Loss on ignition =	0.74	%		

CR5: Crushed rock, Kajaani (4. phase)
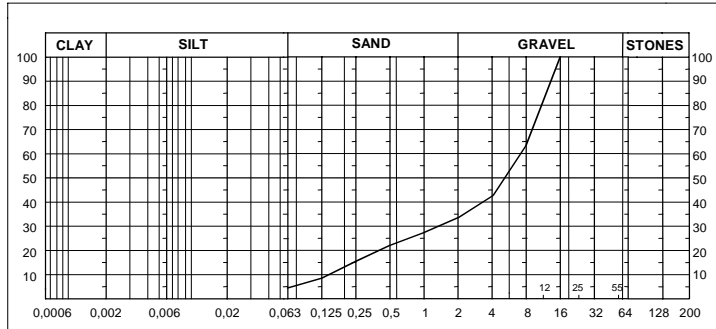
γ_{dmax} =	23.10	kN/m³	<u>Amount of</u>				<u>Stabilization Class</u>		
			<u>fraction</u>						
w_{opt} =	7.22	%	< 0.063 mm =	4.7	%		Grain size distrib.:	well	
d_{40} =	3.6	mm	0.063 – 2.0 mm =	28.3	%		Organic matter:	well	
d_{50} =	5.6	mm	2.0 – 16.0 mm =	67.0	%		As a whole:	well	
$pH_{<2,0\ mm}$ =	6.95								
MHC =	-		Loss on ignition =	0.54	%				

CT1: Crushed till, Pihtipudas (4. phase)
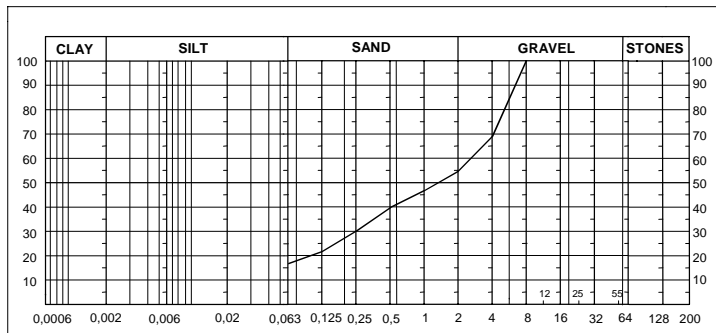
γ_{dmax} =	22.04	kN/m³	<u>Amount of</u>				<u>Stabilization Class</u>		
			<u>fraction</u>						
w_{opt} =	5.78	%	< 0.063 mm =	16.4	%		Grain size distrib.:	moder	
d_{40} =	0.6	mm	0.063 – 2.0 mm =	38.2	%		Organic matter:	well	
d_{50} =	1.5	mm	2.0 – 16.0 mm =	45.4	%		As a whole:	well	
$pH_{<2,0\ mm}$ =	6.81								
MHC =	II		Loss on ignition =	1.59	%				

Deflection basins, Test structure 5

Trial5 24.6.97

Trial5 20.8.97

Trial5 7.5.98

Trial5 20.8.98

Trial5 1.6.99

Trial5 30.6.99

Trial5 23.9.99

Trial5 22.5.2000