Mechanical Properties of Residues as Unbound Road Materials

– experimental tests on MSWI bottom ash, crushed concrete and blast furnace slag

MARIA ARM

KTH Land and Water Resources Engineering

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KTH Land and Water
Resources Engineering

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Maria Arm

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ABSTRACT

For recycled aggregates and industrial by-products to be used correctly in road construction, it is necessary to know their properties. Existing material specifications and test methods for aggregates used in Sweden and in many other countries are indirect and are based on empiricism. Over the years they have been adjusted to conventional aggregates, which makes the introduction of new materials difficult. Research of their properties is being conducted in many places although knowledge has been inadequately disseminated.

The objective of this thesis is to increase knowledge of the mechanical properties of certain selected residues for improved design of pavements using these residues.

The study has concentrated on residues in unbound road layers. The materials selected were processed municipal solid waste incinerator (MSWI) bottom ash, crushed concrete and air-cooled blast furnace slag (AcBFS). The deformation on loading, the possible strength development over time and the resistance to mechanical and climatic action were studied in the laboratory and in the field. The results were compared with those of the conventional aggregates they could possibly replace, such as sand, gravel and crushed rock. The methods used in the laboratory were cyclic load triaxial tests, Los Angeles tests, micro-Deval tests and freeze-thaw tests. In the field, test sections with residues and reference sections with conventional aggregates in the unbound layers were monitored by means of falling weight deflectometer (FWD) measurements.

The laboratory results showed that a high content of unburned material in MSWI bottom ash limits the resilient modulus but not the permanent deformation to the same extent. Both laboratory and field results showed several years' growth in stiffness for unbound layers with crushed concrete and AcBFS, which is not present for unbound layers with natural aggregates. This was thought to be caused by calcium dissolution and precipitation in the compacted material layer. A special investigation of the material in question, together with knowledge of the planned construction, could permit a higher value to be used in the design modulus than for crushed rock and thus benefit from the increased stiffness.

The Los Angeles test and other tests developed for single-sized aggregates did not really justify the performance of the materials studied. Recycled aggregates and other residues, as well as conventional unbound road materials, should be analysed using cyclic load triaxial tests in the laboratory and FWD measurements in the field, both of which take into account the whole composite material or layer. Consequently, a new methodology for material assessment and comparison is proposed, based on permanent deformations in cyclic load triaxial tests.

According to the laboratory and field tests, some bottom ash could be used, not only in embankments and capping layers but also to bear the stress levels expected in a sub-base. Recycled aggregates and other residues should be used near the source of production and not necessarily in roads with low traffic volumes. Their properties should be used to the greatest possible extent although their limitations must be taken into account.

Key Words: residues; unbound materials; MSWI bottom ash; crushed concrete; blast furnace slag; mechanical properties

PREFACE

This thesis is the second and concluding part of a PhD project started in 1997.

The first part resulted in a licentiate thesis in 2000, entitled "The properties of alternative aggregate materials – with a special reference to MSWI bottom ash, crushed concrete and blast furnace slag" (Arm, 2000a; in Swedish with an extensive summary in English). It was financed by the Swedish Transport and Communications Research Board (KFB), the Development Fund of the Swedish Construction Industry (SBUF) and the Swedish National Road and Transport Research Institute (VTI), Linköping.

This second part has been financed by the Swedish Agency for Innovation Systems (VINNOVA) and the Swedish Geotechnical Institute (SGI), Linköping. The work has been carried out at SGI in co-operation with the Road Material Laboratory at VTI and the Department of Land and Water Resources Engineering at The Royal Institute of Technology (KTH), Stockholm.

The Swedish Aggregates Producers Association (SBMI, formerly GMF) financed the investigations that formed the bases of Paper III. The European Commission, KFB and the Swedish National Road Administration (SNRA) provided funding for the study in Paper IV, which was written in co-operation with Krister Ydrevik and Hans G. Johansson, former researchers at VTI. The investigations that formed the bases of Paper V were carried out in co-operation with Krister Ydrevik and financed by SNRA.

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This thesis is based on the following papers, which are referred to in the text by their respective Roman numerals, and additional data. It is also to a large extent based on the licentiate thesis (Arm, 2000a).

Paper I

Arm, M. 2003. **Mechanical properties of processed MSWI bottom ash, evaluated from laboratory and field tests.**
Submitted to the Journal of Solid Waste Technology and Management

Paper II

Arm, M. 2003. **Variation in deformation properties of processed MSWI bottom ash**.
Submitted to Waste Management

Paper III

Arm, M. 2001. **Self-cementing properties of crushed demolished concrete in unbound layers: results from triaxial tests and field tests**.
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Paper IV

Arm, M., Johansson, H.G. & Ydrevik, K. 2003. **Performance-Related Tests on Air-Cooled Blast-Furnace Slag and Crushed Concrete.**
In: Eighmy (edited) Beneficial Use of Recycled Materials in Transportation Applications. Proceedings of the conference held in Arlington, Virginia, in November 13–15, 2001. Air & Waste Management Association (AWMA). pp 237–248.

Paper V

Arm, M. & Ydrevik, K. 2003. **Assessment of the deformation behaviour of alternative unbound road materials, by means of results from cyclic load triaxial tests**.
Manuscript

1 INTRODUCTION

Every year, about 75 million tonnes of aggregate material are produced in Sweden, which is approximately one lorry load of aggregate per inhabitant. The aggregate is used for the construction of roads, railways, bridges and embankments, but also buildings, streets, squares, car parks, noise barriers etc. The road construction industry is responsible for about half of the aggregate consumption.

As in other sectors, sustainable management of resources has commenced in road construction. This has resulted in the introduction of alternative aggregate materials, such as recycled aggregates or industrial residues of different kinds. The background to this is a number of political objectives and control instruments together with administrative and technical measures. The overall idea is that it should be a matter of course to use alternatives when possible and thus decrease landfill and reduce extraction from gravel pits and rock quarries. In this way, the use of alternative materials prolongs the life of existing landfills and reduces the need for new pits and quarries.

For residual products to be an interesting alternative to conventional aggregates, such as sand, gravel and crushed rock, certain qualifications must be fulfilled. The material must have suitable engineering properties, must have an acceptable environmental impact and its cost should be reasonable.

The present regulations for the utilisation of residues in unbound layers are set out in the Swedish National Road Administration’s technical specifications for roads, ATB VÄG (SNRA, 2003a). According to these regulations, evidence is required that the alternative aggregate is equivalent to the material it replaces in a standard construction. Another possibility is to propose an alternative design whose strength must also be demonstrated.

For residues to be utilised properly in road construction, it is necessary to know their properties. Research is in progress in many places and new experience is being gained from test sections, although knowledge is still inadequate and, most of all, it is inadequately disseminated, which was the incentive for this thesis.

1.1 Political goals and measures for increased recycling in Sweden

Political steps taken in Sweden to promote recycling in the road construction industry include:

- The Ecocycles Bill (Bill 1992/93:180): The Bill was adopted by Parliament in 1993 and stated: “It should be possible to use, reuse, recycle or finally take care of what is extracted from nature in a sustainable way, with less consumption of resources and without harming the natural environment”.
- The Tax on Natural Gravel: In 1996 a natural gravel tax of SEK 5 per tonne of gravel extracted was introduced. In 2003, the tax was doubled to SEK 10 per tonne. It is the gravel producer that pays the tax.
- Swedish Environmental Objectives (Bill 1997/98:145): In 1999, the Swedish Parliament adopted 15 environmental quality objectives, describing the state of the Swedish environment that would be necessary to achieve sustainable development within our generation. The fifteenth objective is ‘A good built environment’ and states: “Buildings and amenities must be located and designed in accordance with sound environmental principles and in such a way that they promote sustainable management of land, water and other resources”. Furthermore, the objective implies that natural gravel should only be used for construction purposes when there are no possible substitutes in specific applications. Moreover, waste and residues should be separated according to cate-

gory and recycled on a co-operative basis in urban areas and the surrounding rural areas.

- The Waste Tax (SFS 1999:673): After many years of preparation, a waste tax of SEK 250 per tonne of waste deposited on landfill sites was introduced in 2000. Since then it has gradually been increased, to SEK 288 per tonne in 2002 and to SEK 370 in 2003. For deposited material that is reused in some way, in road construction for example, the waste tax is repaid. The purpose is to gradually reduce the amount of waste reaching landfills.
- Swedish Environmental Objectives – Interim Targets and Action Strategies (Bill 2000/01:130) approved in Nov 2001: To guide efforts towards achieving the 15 objectives adopted in 1999, the Government proposed interim targets for each objective, indicating the directions and timescale of the actions to be taken. One of the interim targets for 'A good built environment' reads: "The quantity of landfill waste, excluding mining waste, will be reduced by at least 50% by 2005 compared with 1994, at the same time that the total quantity of waste generated does not increase". Two other targets state that by 2010 the extraction of natural gravel in the country will not exceed 12 million tonnes per year and the proportion of reused materials will represent at least 15% of the total aggregate used. In 2001, the corresponding figures were 23.4 million tonnes and 11% (SGU, 2002). The majority of this 11% consisted of excavated rock and scrap boulders.
- Ban on landfill (SFS 2001:512 and 2001:1063): To reduce the amount of waste sent for landfill, the Government introduced a ban on landfill using sorted combustible waste, effective from 2002 and a ban on the landfill of organic waste generally from 2005. As a result, expansion of recycling capacity, especially waste incineration with energy recovery, is planned for the whole country. In the meantime, exemptions to the ban need to be granted.

1.2 Conditions for recycling

For a recycled aggregate or an industrial residue to be an alternative to conventional aggregates certain qualifications must be fulfilled. According to what has been mentioned previously, use should result in suitable technical properties, acceptable environmental impact and reasonable costs.

What is meant by suitable technical properties for a road material depends of course on the use in road construction. If the material is used as a surfacing layer it must endure the load and the wear from the traffic. It must also endure the temperature changes, it must be dense and it must be able to protect underlying layers. If the material is placed further down in the construction, perhaps 40 cm below the road surface, the traffic load is not as important since the layers above have spread the load out. Furthermore, the temperature changes or the risk of being exposed to de-icing salt are no longer present. Instead, the material must be compactable into a stable platform for the layers above. It is also very important that the material is not frost-susceptible. It must not absorb water that freezes under volume expansion and results in heave.

Acceptable environmental impact is difficult to define. What is acceptable depends on limit values, which in turn depend on what is acceptable. The degree of environmental impact also depends on utilisation. There is a large difference between use of a material in the road surface, where it is exposed to all kinds of impact from climate and traffic, and use below a 15 cm thick asphalt layer.

When judging reasonable costs, the alternative, i.e. the use of a conventional material, must be considered. As part of this consideration, aggregate production costs, transport, the waste tax and landfill costs must be evaluated.

Around 30 million tonnes of residual products that could possibly be used as road materials are generated annually. The distribution among these is shown in Figure 1 (right). For comparison, the annual production of aggregates is also given (left). Production is divided into gravel, crushed rock, till and other production. "Other production" mostly includes crushed rock from mobile crushers and scrap boulders. However, the volume of aggregates produced varies from year to year depending on infrastructure investments and housing construction. In 1994, 83 million tonnes of aggregates were produced, from which 60% was used in the road sector (SNRA, 1996). In 2001, as illustrated in Figure 1, the corresponding figures were 71.5 million tonnes and 55% (SGU, 2002).

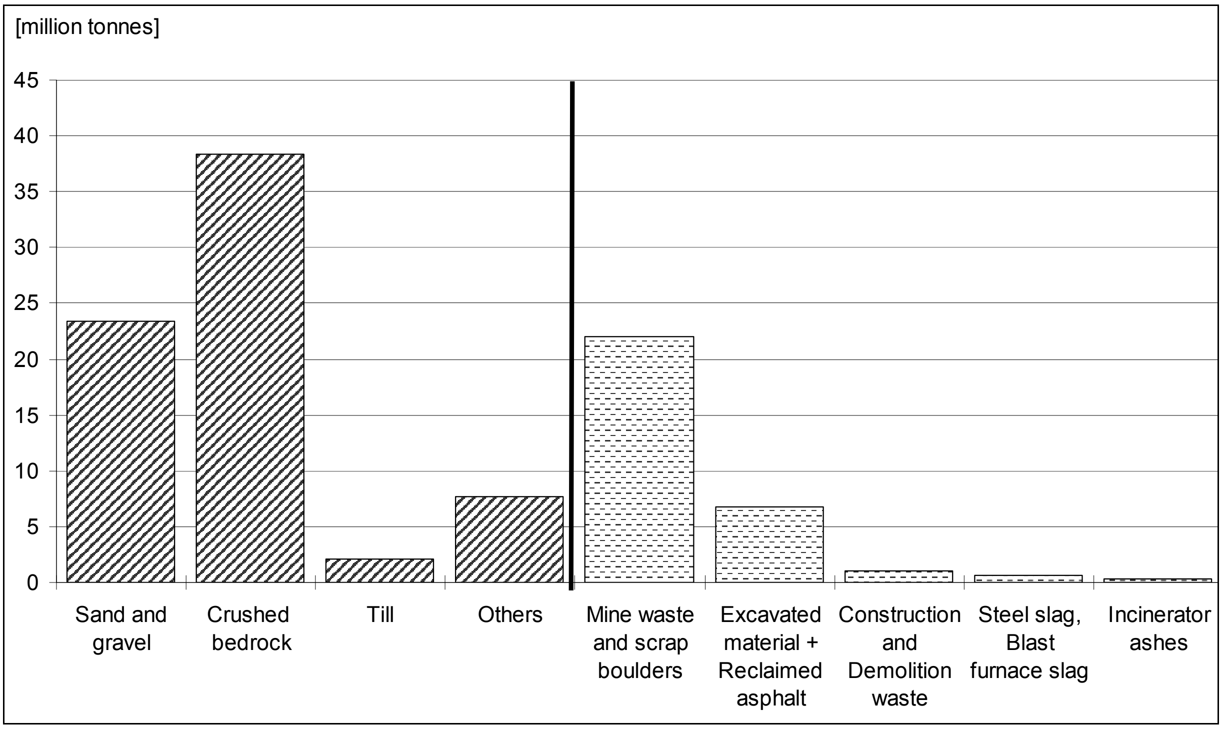


Figure 1. ***Annual aggregate 'production' in Sweden. Left: Annual production of conventional aggregates. Right: Distribution among possible alternative aggregates that are generated each year. (Data from Arell, 1997; SNRA, 2000; SGU, 2002; RVF, 2002).***

According to Figure 1, the possible alternative aggregates mainly take the form of mine waste, a residue that arises in remote areas, where consumption of aggregates for roads is generally low. In spite of this, the residues could be an interesting alternative, especially in areas with a shortage of extractable gravel and rock. It is quite possible, for instance, that mine waste is transported by train and boat from Kiruna in the North for use as aggregate in southern Sweden. The export of Swedish mine waste is also a possibility that is already working to some extent (Boverket, 1998).

The remaining residues in Figure 1 have different distributions throughout the country. Construction and demolition waste arise here and there depending on construction activities while residues from quarries, the steel industry and incineration plants are located at certain places. The conclusion is that although they correspond to a small 'production' volume in view of the needs of the whole country as a whole they could be an interesting alternative within the region in question.

A more complete inventory (volume and location) of residues that could possibly replace sand and gravel has been made by SGI within the framework of the ongoing environmental objectives and will be published later this year.

In this context it is interesting to see which of the possible alternative materials are affected by the waste tax. Firstly, the tax only covers waste sent to a landfill where more than 50 tonnes a year is finally disposed of or stored for longer than three years. Secondly, some waste categories are exempted, such as mine waste, steel slag and blast furnace slag. On the other hand, incinerator ash, reclaimed asphalt, construction and demolition waste are affected by the tax. Scrap boulders and excavated material are exempted if they are disposed of at a landfill site that does not receive taxable waste as well, such as construction and demolition waste.

1.3 Present assessment of residues as unbound road materials

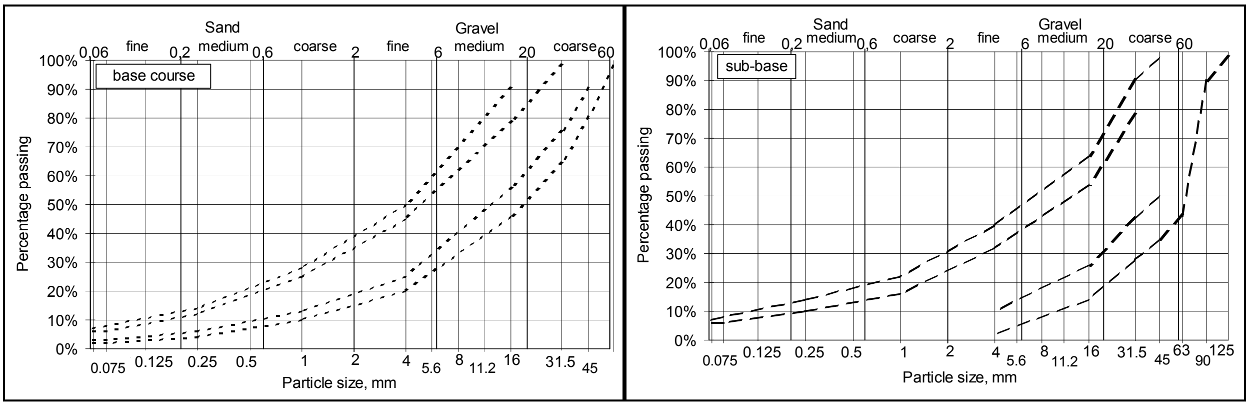
The present regulations for the utilisation of residues as road materials in unbound layers are set out in the Swedish National Road Administration's technical specifications for roads, ATB VÄG (SNRA, 2003a). According to these regulations, evidence is required that the function of the residue is equivalent to the function of the material it replaces. This is natural since the present design manual stipulates that the materials used have certain stiffness or deformation properties. ATB VÄG also offers the possibility of suggesting an alternative design but this must be approved in every single case. The strength, for instance, must be demonstrated, which requires knowledge of the E-modulus and 'permissible load' of the new material.

The fact that the alternative material should be equivalent to the conventional material means that the same test methods and limit values apply. By tradition, unbound materials are used to a large extent in Swedish roads. Typical Swedish pavements contain several layers of unbound material with different roles and subsequently different requirements regarding the properties. Table 1 and Figures 2a and 2b include a summary of the present requirements for unbound materials in paved roads with a flexible construction according to ATB VÄG.

Table 1. Present requirements for unbound materials in paved roads with a flexible construction according to ATB VÄG (after SNRA, 2003a)

Use	Property or method	Limit value
Base course	• micro-Deval value / Ball mill value[1]	− If used by construction traffic: max. 17 / max. 23, otherwise: max. 30 / max. 37.
	• Organic matter content[2,3]	− Max. 2 wt.-% of fraction <2 mm.
	• Particle size distribution	− According to Figure 2a.
	• Amount of uncrushed particles[2]	− <30 wt.-% of material >16 mm.
	• Maximum particle size	− Depends on layer thickness.
Sub-base of crushed material[4]	• micro-Deval value / Ball mill value[1]	− Max. 30/max. 37. Recommendation: If used by construction traffic, max. 17/max. 23.
	• Organic matter content[2,3]	− Max. 2 wt.-% of fraction <2 mm.
	• Particle size distribution	− According to Figure 2b.
	• Amount of uncrushed particles[2]	− <30 wt.-% of material >16 mm.
	• Maximum particle size	− Depends on layer thickness.
Capping layer	• Organic matter content[3]	− Max. 2 wt.-% of fraction <2 mm
	• Fines content / capillarity[1]	− Max. 11 wt.-% / max. 1 m
Unbound layer	• Thermal conductivity	− If 0–25 cm from road surface: >0.6 W/(m · K). If 26–50 cm below surface: >0.3 W/(m · K).

[1]:Alternative method. [2]:If material other than crushed rock. [3]:Through colorimetric measurement. [4]:For use as a sub-base of uncrushed material the requirements on the amount of uncrushed particles do not apply.



Figures 2a and 2b. Outer and inner limit curves for particle size distribution of base course and sub-base materials in paved roads (after SNRA, 2003a).

There are also additional requirements to Figures 2a and 2b regarding the particle size distribution of base course and sub-base material that prohibit the use of gap-graded material and material with a clay content that is too high. The organic matter requirement applies to all pavement materials and in fill situated within 1 m of the formation level. If cement-bound layers are used in the pavement, the limit applies within 2 m of the formation level.

The suitability of a residue as unbound road material can thus be demonstrated through different kinds of laboratory tests.

For material classification of residues, ATB VÄG requires a special investigation to be made to evaluate bearing capacity, stability, strength, resistance, frost susceptibility and environmental impact. However, some of these properties have by tradition not been regarded as problematic for unbound materials and therefore no methods or limit values have been specified. This applies to bearing capacity, stability, strength, resistance to climatic action, resistance to chemical action and environmental impact. It is true that there are indirect specifications for bearing capacity and stability. These are given in the form of limits on particle size distribution, the amount of uncrushed particles and the organic content of different pavement materials.

Field tests can also be used to prove equivalency. Test roads are then constructed and monitored using different measurements. In the evaluation, test sections with residues are compared to reference sections constructed using conventional materials. In this context it is important to continue monitoring over several years to study possible differences in the long-term properties.

In conclusion, the existing technical specifications do not give any proper guidance for evaluating the suitability of residues as road materials, which is something the National Road Administration has also pointed out (SNRA, 1996). The specifications are empirical and are founded firmly on long experience from conventional materials. However, the same limit values are not obvious for recycled aggregates and residues. Furthermore, new materials could possess other properties that are not measured properly using traditional methods.

Nor are there any methods or limit values specified for environmental impact from unbound materials. The lack of general guidelines means that the local environmental authority must examine individually all use of residues in roads. This is a system that leads to different results in different parts of the country.

However, quite recently, during spring 2003, the SNRA presented a proposal for separate guidelines for the use of recycled concrete road materials, which has been referred to different bodies for consideration (SNRA, 2003b). The proposal is based partly on the findings in the first part of this project (Arm, 2000a).

1.4 Lack of knowledge

Literature studies at the beginning of this project (Arm, 2000a) revealed quite a few areas with insufficient knowledge of the use of different residues in road construction:

According to an OECD report in 1997 on recycling strategies for road-works, there was a need for more research on *guidelines, test methods, characterisation of residues, design methods* and, most of all, *long-term properties of residues* (OECD, 1997).

Arell (1997) mentioned three areas for future research, namely *frost heave, permanent deformations in aggregate materials* and *test methods for the assessment of environmental impact.* In the matter of frost heave, it was questioned in particular whether the self-cementing properties of crushed concrete remain after several frost seasons. As regards permanent deformations, a general method was requested by which a few tests could determine whether a material is deformed more than is permitted, assuming a certain traffic load and use at a certain depth from the road surface. As regards environmental impact, a general relationship between laboratory results and field results was demanded. Furthermore, a general, simple and cheap method was requested on which a risk evaluation for different environmental conditions could be based.

In a summary by Johansson (1997) he stated that standard *specifications* and testing requirements for crushed concrete as well as other by-products, must be published. A modern approach, new and modified analysis, *performance-related tests*, *modified manuals*, new technical procedures, economic factors etc. were identified as areas that needed to be addressed to to promote the recycling strategy.

In the SNRA plan of action for sustainable road management a *national basis for the assessment of environmental impact* of different kinds of road materials was demanded (SNRA, 1996). The same report pointed out the lack of *criteria for evaluating* different secondary aggregates and residues and their suitability for utilisation in roads.

Nunes, who in his thesis (Nunes, 1997) reported results from triaxial tests on different residues, recommended continued research on the same residues, but with other origins, to study the *variability* of, for instance, coal ash. It was also recommended that the investigations should be extended to other alternative materials. Air-cooled blast furnace slag (AcBFS) was part of the study, but not crushed concrete or municipal solid waste incinerator (MSWI) bottom ash. It was pointed out that *performance-based specifications* should be drafted, incorporating laboratory characterisation using, among other things, stiffness modulus. Furthermore, it was recommended that research be done on the development of mathematical models describing permanent deformation behaviour in the laboratory and applying this to *in situ* behaviour.

The use of residues as road material in Sweden demands new *knowledge*. Firstly, it must be proved that the new material has properties equal to that of the conventional material it is to replace. This requires tests with certain standard methods. Secondly, in some way it must be proved that the residue is also equal in those areas where a test method and limit value are lacking, which demands considerable testing. If the tests then result in unequal properties it is advisable to suggest an alternative construction where the material properties are suitable. This also demands knowledge. According to SNRA (1996) the road construction industry regards this lack of knowledge as a problem.

Within the subject of environmental impact a great deal of knowledge has been gained on the leaching properties of different residues. Many leaching tests have been performed on ash materials and blast furnace slag and some on crushed concrete. The problem is that the methods for leaching tests have varied over the years. Furthermore, the results have seldom been related to limit values or to corresponding values for conventional materials.

1.5 Current research in Sweden

Several simultaneous research projects are in progress aimed at characterising different residues and establishing both design guidelines and environmental guidelines for use. This thesis is the result of one of these research projects. Other **examples** of current research in the field of alternative aggregates are:

- Modelling emissions from roads with residues (Lund Institute of Technology/ SGI/KTH/Luleå University of Technology)
- Use of different residues for landfill covering and as a low-quality road material (Chalmers University of Technology/Swedish National Testing and Research Institute/representatives from the construction industry, pulp and paper industry and foundry industry)
- Establishing environmental guidelines for the use of incinerator ash (SGI)
- Development of tools for assessment of environmental impact from aggregates (KTH)
- Stabilisation of processed MSWI bottom ash (SGI)

There are of course also broad-based activities outside Sweden. Only one project will be mentioned here. The project is SAMARIS and is financed by the EC 5th framework. The Swedish participant is VTI. Its objectives are to produce a general methodology for the assessment of road materials; to draft an environmental annex to CEN product standards and to define testing protocols for the investigation of hazardous components; to develop mechanical models and test methods in order to derive performance-based specifications related to functional properties; and, finally, to produce technical guidelines and recommendations for the correct use of recycling techniques in road construction. Recycling and alternative materials are also priority areas in the EC 6th framework.

Results from the first part of this project have been presented in a licentiate thesis published in 2000 (Arm, 2000a). In that thesis, results were presented separately for different areas. Firstly, it described important properties of unbound road materials and their importance to the functioning of an unbound layer. The existing test methods, both standardised and non-standardised, were also described for each of these properties. In some cases the suitability of the standardised test methods was discussed. Secondly, it described the regulations and experience related to the use of residues in Denmark, the Netherlands, Finland and USA. Former use in Sweden and the future impact of European harmonisation were also discussed. Furthermore, test results were set out for the three materials studied, both from investigations within this project and from investigations described in the literature. Finally, the licentiate thesis presented a proposal for assessing new and alternative materials, based on a comparison with well-known road materials. Design conditions, test methods and material characterisation were discussed. The concept of bearing capacity was described and exemplified.

In this doctoral thesis the method of comparison is described further and exemplified with test results. The deformation properties of the materials studied have also been evaluated further through new data from laboratory and field investigations.

2 PROBLEMS

Despite Sweden's relative richness in natural aggregate reserves, there is a political ambition to facilitate and increase the use of residues in Swedish road construction. When residues are used in road construction they need to comply with the structural engineering requirements and their environmental compatibility must be ensured. However, knowledge of residues in Sweden is inadequately disseminated and the existing specifications, test methods and pavement design are not always suitable for new, unknown material.

- There is a lack of knowledge of residues among public sector customers and the contractors.
- Existing test methods standardised for unbound road materials, are indirect and test particles instead of the whole composite material. The methods are developed for conventional material, such as gravel and crushed rock, and do not allow a fair comparison to be made between conventional materials and alternative materials, such as different kinds of residues.
- Performance-based design needs methods that can evaluate the performance, both in advance and after construction.

3 OBJECTIVES

The general objective of this thesis is to increase knowledge of the mechanical properties of certain selected residues for improved design of pavements using these residues (Figure 3). The particular objectives are

- to facilitate comparisons between conventional and alternative aggregate materials through a description of performance testing in the laboratory and in the field (Papers I, III and IV).
- to demonstrate possibilities and restrictions in the use of different residues through the presentation of results from laboratory tests and field tests on the three materials studied (Papers I, II, III and IV).
- to suggest a methodology for the assessment of the deformation behaviour of alternative unbound road materials (Paper V).

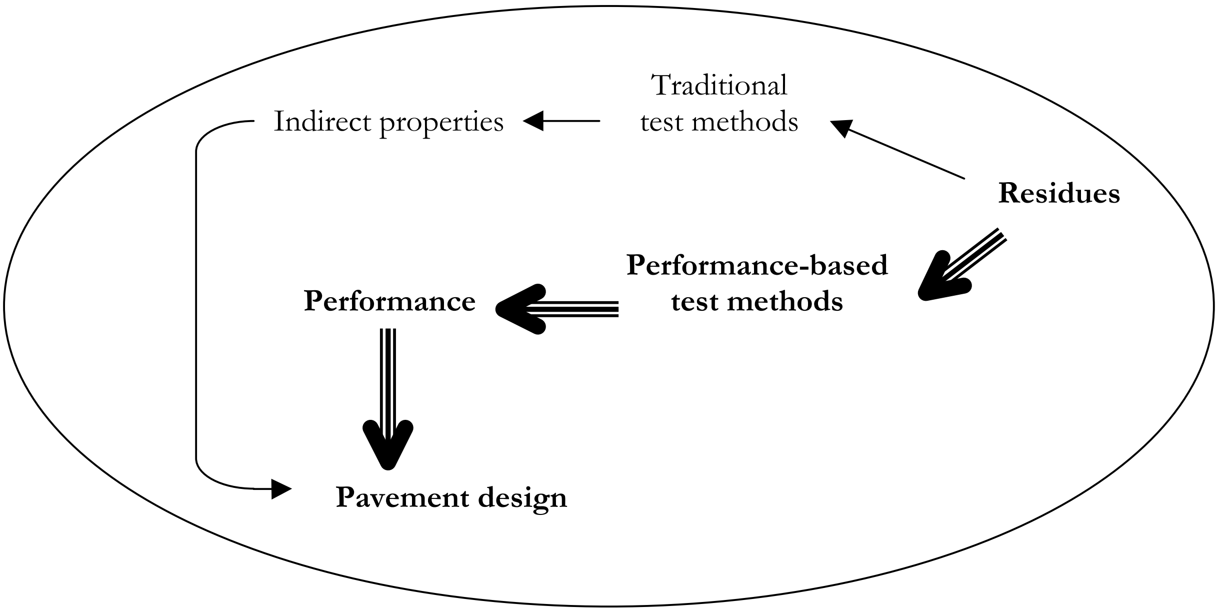


Figure 3. Research topics treated in this thesis. Bold text refers to a direct design method, whereas the other text refers to an indirect design method.

The focus of the study is on residues in unbound road layers. Since there is a tradition of thick, unbound layers in Swedish road construction, this implies a considerable savings potential with regard to in natural aggregates.

The materials selected for the detailed studies were processed municipal solid waste incinerator (MSWI) bottom ash, crushed concrete and air-cooled blast furnace slag (AcBFS). In Sweden '*slaggrus*' is the name for processed bottom ash from waste incinerator plants. The term *crushed concrete* refers to both concrete from the demolition of buildings and other structures and to residues from the production of concrete or concrete products. '*Hyttsten*' is the Swedish name for air-cooled slag from blast furnaces. These materials have been selected in view of their expected scope of application, available quantities and accessibility throughout the whole country, and the wishes of the users. All three residual products are used in road and civil engineering construction in other European countries.

The following properties, relevant for unbound aggregate material, have been studied:

- deformation on loading
- strength development
- resistance to mechanical and climatic action

In the previous work, reported in the licentiate thesis (Arm, 2000a), the susceptibility to frost heave, thermal conductivity and leaching behaviour were also discussed.

4 UNBOUND ROAD MATERIALS – IMPORTANT MECHANICAL PROPERTIES AND PRESENT TEST METHODS

This chapter describes important mechanical properties of unbound road materials and their significance in the performance of those materials, as well as existing standardised test methods. In the future, the national standards for aggregate and road materials of all member countries of the European Union will be replaced by European standards. These product standards (e.g. EN 13242 and EN 13285) and test method standards are presently being drawn up within the different technical committees of the European Committee for Standardisation (CEN, Comité Européen de Normalisation).

4.1 Unbound road materials

The role of an unbound layer within the pavement is to act as a stable platform on which the upper layers of the pavement can be compacted and constructed. The unbound layers should also be permeable and non-frost susceptible and they should operate as a frost protection layer, insulating the subgrade against frost. Finally, an unbound layer (as well as the bound layers) should spread the traffic load to reduce stress on the underlying pavement layer and the subgrade, thus preventing overstress and rutting in the subgrade.

The performance of a material depends on where it exists in the pavement structure. Traffic-induced stress is highest on the road surface and diminishes with depth according to the load-spreading capacity of the different materials (Figure 4). Bitumen-bound materials have a greater load-spreading ability than unbound materials. This applies even more to cement-bound materials.

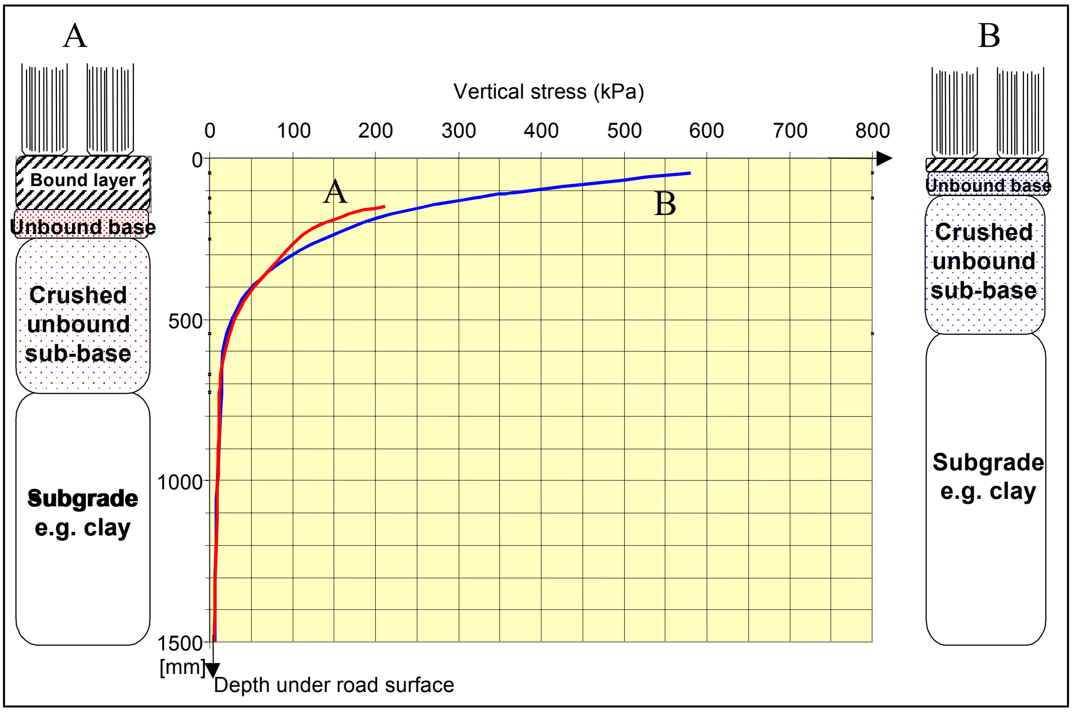


Figure 4. ***Comparison of vertical stress distribution in unbound road layers in two constructions, A and B, with different bound layer thickness.***

Figure 4 shows the traffic stress distribution in two constructions when loaded with a heavy vehicle (10 tonnes axle weight). It is only heavy vehicles that are important when discussing traffic load and bearing capacity. On the other hand, passenger cars contribute to surface wear and the annual average daily traffic level is therefore used when designing surfacing layers (type of surfacing e.g. asphalt concrete or surface dressing, aggregate size and quality, bitumen type etc).

4.2 Mechanical properties of unbound road materials

The amount by which an unbound aggregate material is deformed when loaded depends on its stiffness and stability. ***Stiffness***, or the ability to spread the load, is a measure of the resistance to resilient deformation. It is expressed in terms of a modulus of elasticity or resilience that is used in designing the pavement. ***Stability*** is a measure of the ability to resist permanent deformation. Another term is ***load-bearing capacity***, which could be defined as the load a layer of material can carry without being deformed more than the permissible amount. Determination of the bearing capacity thus requires a limiting deformation value.

These three properties are, among other things, dependent on the compaction result, which is in turn dependent on the particle size distribution and the particle shape. The *mineralogical composition* and the internal structure of the particles also have a considerable impact on deformation properties. The quality of fines (the type of minerals) is also an important parameter. Fines are defined as material with a particle size of up to 0.06 mm.

The *particle size distribution* is usually presented as a graph. In this graph, the maximum particle size, the fines content and the curve shape are important parameters. The curve shape can be characterised by a uniformity coefficient, c_u, which is the ratio of d_{60} to d_{10}. d_{60} means the mesh of the sieve through which 60% of the material passes. However, the c_u can be lacking in sensitivity as it does not indicate unstable curves with 'sand bumps'. In that case, the curvature index, c_c, $(= d_{30}^2/(d_{60} \cdot d_{10}))$ is more usable. A well-known equation used to describe the curve shape is Fuller's equation

$d = (P^2D)/10\,000$ (Fuller, 1905)

which could be written

$$P = (d/D)^n \qquad (1)$$

where

P = percentage smaller than particle diameter d
D = largest particle diameter in the material
n = parameter describing the shape of the curve, here n = 0.5

For natural aggregate materials, the size of the particles that form the material skeleton that transmits the load is most important for the stiffness. It is also well known that the less steep the particle size distribution curve, the more stable the material. To obtain the maximum number of contact points between particles, so-called optimal compaction, the distribution curve should have n-values of 0.35 to 0.45 in the equation (1) (Zheng et al, 1990 quoted in Kolisoja, 1997).

The *particle shape* can be rounded or be more or less angular. A more angular material requires greater compaction, which could create crushing and an increase in fines. Conversely, a material with rounded particles is generally easy to compact but is also more unstable than angular material.

Organic matter has a harmful effect on the stiffness of an unbound road material. Sweden and many other countries have therefore limited the permissible organic matter content of road materials. However, organic material is seldom problematic for Swedish natural aggregate road materials, such as sand, gravel, stone or crushed rock.

One factor that has been shown to be crucial to deformation properties, especially for fine-grained soils, is the *water content* (Arm, 1996, 1998; Kolisoja, 1997). In general, deformation in fine-grained soils exposed to repeated load increases with the increase in water content. This is due to low permeability in combination with the load that develops excess pore water pressure and subsequently decreases the effective stresses transmitted through the particle skeleton. The magnitude of the influence depends, except for the water content and the particle size distribution, on the electrochemical properties, which are based on the mineralogical composition of the aggregate (Kolisoja, op.cit.).

For deformation properties to remain the same over the life of the road, the particle size and particle shape must not change, i.e. the material must be resistant to both mechanical and climatic action. Unbound materials are exposed to mechanical action all the time they are handled (loading, unloading, spreading, compaction and construction traffic). Final traffic also has an impact, but this is very slight if the road is designed properly. The *resistance to mechanical action* depends on the particle strength, which depends on the geometrical shape, the mineral composition and cohesion and the structure and texture of the particle. In a layer of unbound materials, the contact between the particles, e.g. the degree of compaction of the material, which in turn depends on particle size distribution and so on, also exerts an influence.

Freeze–thaw resistance or resistance to temperature alternations is an important property of materials that may be expected to freeze and thaw repeatedly under totally or partly-saturated conditions. Degradation owing to poor freeze–thaw resistance occurs because the volume of water that has penetrated into the pores increases when it freezes, which gives rise to considerable forces, which in turn break up the aggregate particles. The risk of damage increases if salt is present, since salt reduces the surface tension of water and makes it easier for water to penetrate small pores. Freeze–thaw resistance is dependent on the strength of the particle, the number of pores and the size of pores inside the particles. Note, however, that it is only the pores accessible to water that are involved in this process. A porous material does not therefore automatically have a low freeze–thaw resistance.

4.3 Present standard test methods

The existing standardised test methods for the properties mentioned above are briefly described in this section, whereas the suitability of the methods is discussed in Section 8.5.

Test methods for deformation on loading

A large number of methods have been developed for measuring the deformation properties of a material under load.

A well-known laboratory method is the *California Bearing Ratio* method, CBR, which is widely used in other countries but is not applied in Sweden. Here, soil materials are by tradition classified indirectly according to particle size distribution. The California State Highway Department in the USA developed the CBR test method in the 1930s. It was intended for testing subsoil material comprising particles up to 19 mm in size. There are several standards for the method, one English (BS 1377), two American (AASHTO T193-72 equal to ASTM D1883) and also one draft for a European standard (draft prEN 13286-47). The differences between the methods are based mainly on the test cylinder diameter, the compaction method and the maximum particle size used. In a CBR test according to ASTM D1883, the material is compacted and loaded in a cylinder with a diameter of 150 mm and a height of 116 mm. The test can be performed at natural water content or under saturated conditions. Sometimes overload can be used (Figure 5).

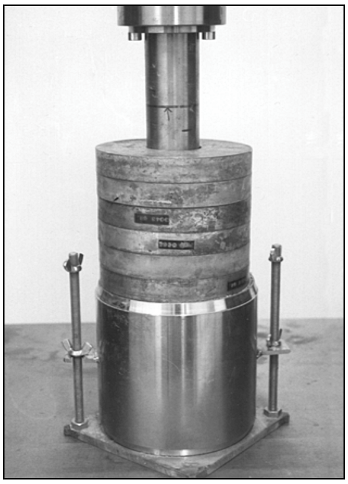


Figure 5. CBR test on a laboratory-compacted specimen with overload (Arm et al., 1995).

The CBR test value is obtained by relating the force that is required to obtain 2.54 mm and 5.08 mm depressions with a plunge in a standard compacted soil material, with the force required for the same depression in a reference material. The relationship is expressed as a percentage. A weak subgrade material can produce CBR values of 2–3% whereas well-graded gravel can produce values of between 30 and 80% (National Stone Association, 1991). It should be noted that when reporting a CBR test it is not only the CBR value that should be declared (which is very common). Since the procedure has a considerable impact on the result, the complete test report should contain both the standard and the procedure used. As an example, specimen compaction according to standard proctor differs greatly from compaction according to modified proctor. As mentioned, the CBR test is widely used outside Sweden – in the United Kingdom and the USA for instance. In some countries subgrade material is classified according to the CBR value. In line with the introduction of analytical design in those countries, empirical relationships between the CBR value and elasticity modulus (E-modulus) have been established. The relationships were necessary as the design systems require an E-modulus as input.

The *static plate bearing test* (VVMB 606:1993 based on DIN 18134) and test loading with a *falling weight deflectometer, FWD*, (VVMB 112:1998) are well-known, standardised field methods in Sweden and several other countries.

The FWD simulates the deflection of a road construction corresponding to the load produced by the wheel of a passing lorry. A falling weight impacts a circular plate with a specified diameter resting on the road surface. The deflection is measured by means of a number of seismometers, one placed in the centre of the loading plate and the others in a straight line radial from it. Data are collected on a disk for later calculation and estimation of the elastic modulus of the layers (= layer modulus) in the road construction. The moduli are estimated through back calculation, modelling the structure as a multilayer system. The back calculation procedure consists of calculating deflection values and comparing these with the measured deflections. The differences are minimised by adjusting the layer stiffnesses. A variety of back calculation software has been developed and finite element methods are being implemented more and more. (COST, 1997; VVMB 112:1998). It should be noted that layer moduli for materials in different test roads should not be compared with each other since the layer modulus is dependent on the actual stress, which is in turn dependent on the pavement construction. On the other hand, materials in test sections and reference sections with the same construction can be compared with each other. It is also very convenient to monitor the layer modulus of a specific section through repeated measurements at different times.

There is also other field equipment that can be used during construction to assess the bearing capacity achieved. These are the *dynamic cone penetrometer DCP,* as well as a number of *small falling weight deflectometers* of different makes (Sweere, 1990; Galjaard & Cools, 1995; Rogers et al., 1995; Henneveld, 1995, Arm et al., 1995).

Apart from these direct loading methods there are some *indirect methods*, such as the sand equivalent test (EN 933-8) and the methylene blue test (EN 933-9). These are used in other countries, in Denmark for example, to assess the quality of fine-grained materials, and are also covered by European standards.

Test methods for organic matter content

There are several methods for determining the organic matter content of a material, e.g. the *loss on ignition method*, LOI, *colorimetric measurement* (SS 02 71 07) and determination of *total organic carbon,* TOC, (prEN 13137). The most usual method for residues is the LOI method, which is performed at 550°C, 800°C or 950°C (SS 02 81 13, VVMB 34:1984, SS 02 71 05). The result is expressed in weight percent. There is also a European standard (EN 1744-1, Chapter 17), which describes ignition at 975±25ºC. NB. It is essential to record the temperature used.

Note also that the different test methods yield different results. Colorimetric measurement generally gives a lower content than the LOI method. (This is dealt with further in Section 8.5.1 of this thesis).

Test methods for resistance to mechanical action

The Swedish standard methods in this field produce different types of mechanical action. Tests in a *Ball mill* (FAS 259-98) and a *Los Angeles drum* (EN 1097-2) produce a combination of abrasion and crushing, while a *micro-Deval test* (EN 1097-1) causes only abrasion. In all three methods a certain fraction of the material is exposed to wear and the resulting increase in fines content is measured. An impact test (EN 1097-2) determines impact strength.

The Ball mill test was originally developed at VTI during the 1980s to test resistance to wear of aggregates for bitumen-bound surfacing layers (Höbeda & Chytla, 1985; Höbeda, 1988). It has later also been used for unbound materials for bases and sub-bases. One kilo of a certain fraction of the material is rotated in a steel drum together with 7 kg of steel balls and 2 litres of water for approximately one hour. Normally, the 11.2–16 mm fraction is tested. However, from 2004 onwards the Ball mill test will no longer be used for unbound aggregates, and will only be used to test resistance of aggregates for the surfacing layer to studded tyres according to EN 1097-9.

The European Los Angeles test is a modification of the original test method from the 1920s. Five kilos of the 10–14 mm fraction of the material is exposed to 500 rotations in a steel drum together with 11 steel balls.

The micro-Deval test was originally developed in France some 45 years ago. In this test, 0.5 kg of the 10–14 mm fraction of the material is rotated 12,000 times in a steel drum together with 5 kg of steel balls and 2.5 litres of water.

Other methods have been developed in different countries and are standardised there. In the USA, for example, there are five versions of the Los Angeles test depending on the fraction being tested (ASTM C131A–D, C535). In Britain, there is the Aggregate Abrasion Value, Aggregate Impact Value, Aggregate Crushing Value and Ten Per Cent Fines Value (BS 112). In Germany, there is the Schlagversuch (DIN 52 115), which forms the basis of the European standard for the Impact Test.

Test methods for resistance to climatic action

Direct tests for resistance to climatic action are *freeze–thaw tests with water* (EN 1367-1), with or without salt. The indirect methods used are *tests with magnesium sulphate* (EN 1367-2), *petrographic analysis* (EN 932-3) and *water absorption tests* (EN 1097-6).

In the freeze-thaw tests according to EN 1367-1 a certain fraction of the material is saturated in water and then exposed to repeated freezing and thawing cycles. Temperatures are –17.5˚C and +20˚C. The resulting degradation is measured. If, for instance, the 11–16 mm fraction is tested the wt.-% of particles <5.6 mm is checked after testing.

There are several variants of freeze-thaw tests standardised in their respective countries. The manner of saturating (duration, with or without vacuum), the manner of freezing (totally or partly saturated), the number of freeze-thaw cycles and the temperatures are varied.

5 PRODUCTION AND PROCESSING OF RESIDUES STUDIED

This chapter includes a description of how the three types of residues selected arise and how they are normally processed in order to be usable as unbound road material.

5.1 Municipal solid waste incinerator (MSWI) bottom ash

Municipal solid waste incinerator (MSWI) bottom ash is a residue from solid waste incineration. Before use it is generally refined through sieving and ageing. In Sweden, the term 'slaggrus' is used for processed bottom ash, where magnetic material and particles greater than 50 mm have been removed and the ash has been stored for at least six months. The storage enables some chemical reactions to take place, which improves the environmental and mechanical properties. It also reduces the water content and the alkalinity.

There are two broad categories of combustion systems for the incineration of MSW. The most common is mass-burning, where the waste is fed directly into the furnace and burned on a grate without any pretreatment. In the other system, refuse-derived fuel (RDF), a more homogenous fuel, is prepared through sieving, crushing or ferrous metal recovery. RDF fuel is generally fired in suspension, stoker or fluidised bed incinerators. The Swedish incinerators in this study are all of the mass-burn type with moving grates. Figures 6 and 7 describe the incinerator plant in Gothenburg.

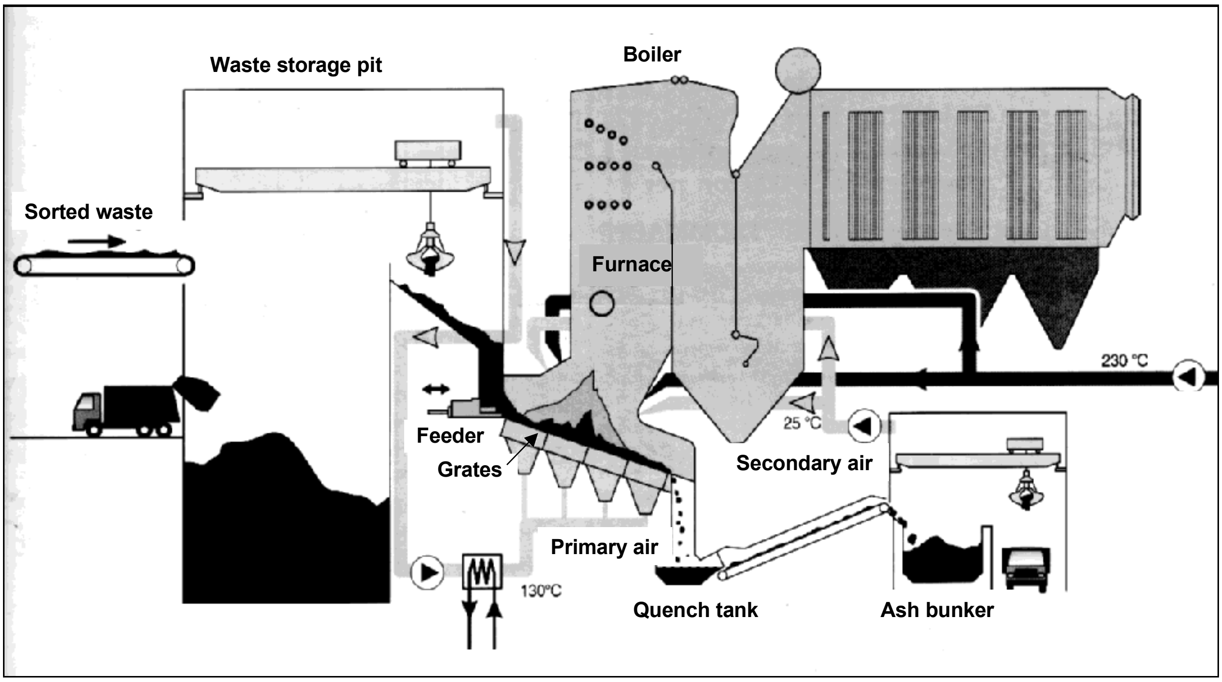


Figure 6. Schematic presentation of the mass-burn incinerator in Gothenburg (after GRAAB, 1996).

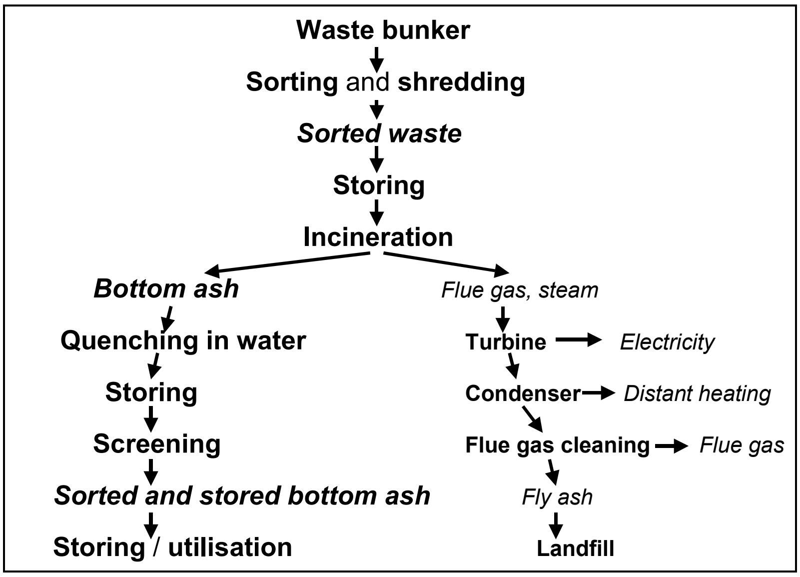


Figure 7. Principle for an MSWI plant (after GRAAB, 1996).

First, the waste is sorted and oversized material is shredded. The sorted waste is then stored in the waste bunker before it is incinerated. During incineration the waste is heated up to 800–1,000ºC. This is done gradually, which first makes the waste dry and at approximately 500ºC it starts to burn. The combustion process results in bottom ash and flue gas. After the incineration the bottom ash is quickly quenched in water and then stored. After screening, when ferrous metal and particles greater than 50 mm are removed, the end-product is obtained.

Heat from the flue gas is converted into electricity via turbines and is then transferred to the district heating system via condensers. Before discharge, the flue gas is cleaned in an air pollution control (APC) system. The residues resulting from this clean-up are in this context called fly ash and are usually disposed of at designated areas.

The most obvious effect of the incineration is the volume and mass reduction of the waste. According to Chandler et al. (1997) the volume is reduced by 90% and the mass by 60%. At Swedish plants a mass reduction level of 75% is reached, which can be concluded from Table 2. The table also shows the distribution between bottom ash and fly ash for certain incinerator plants.

The figures in Table 2 should be interpreted in such a way that incineration transforms one tonne of waste into around 200 kg of bottom ash, around 40 kg of fly ash and around 760 kg of cleaned flue gas. In

Table 2. Production statistics for municipal solid waste incinerator plants (data from RVF, 1997 and 2000).

Plant	Year	Incinerated waste (tonnes)	Energy production (MWh) Heat	Electricity	Residues (tonnes) Bottom ash	Fly ash
Stockholm (Högdalen)	1997	263 896	588 473	22 397	51 912	13 263
Stockholm	2000	401 621	1 130 000	111 000	55 180	18 691
Gothenburg	1997	381 500	1 038 040	129 418	79 302	13 457
Gothenburg	2000	380 827	1 023 413	124 684	76 874	16 112
Malmö	1997	202 166	573 749	0	36 850	4 410
Malmö	2000	198 294	577 299	3 793	37 000	5 300
Linköping	1997	225 585	655 691	0	49 500	9 501
Linköping	2000	225 800	642 690	0	48 396	5 179

addition, there is the production of heat and electricity.

Every year, around 400,000 tonnes of MSWI bottom ash can be produced in Sweden. It is important that only bottom ash is used, since the fly ash extracted from the flue gas is far more contaminated. Figure 8 shows the distribution of ash between the Swedish incinerator plants.

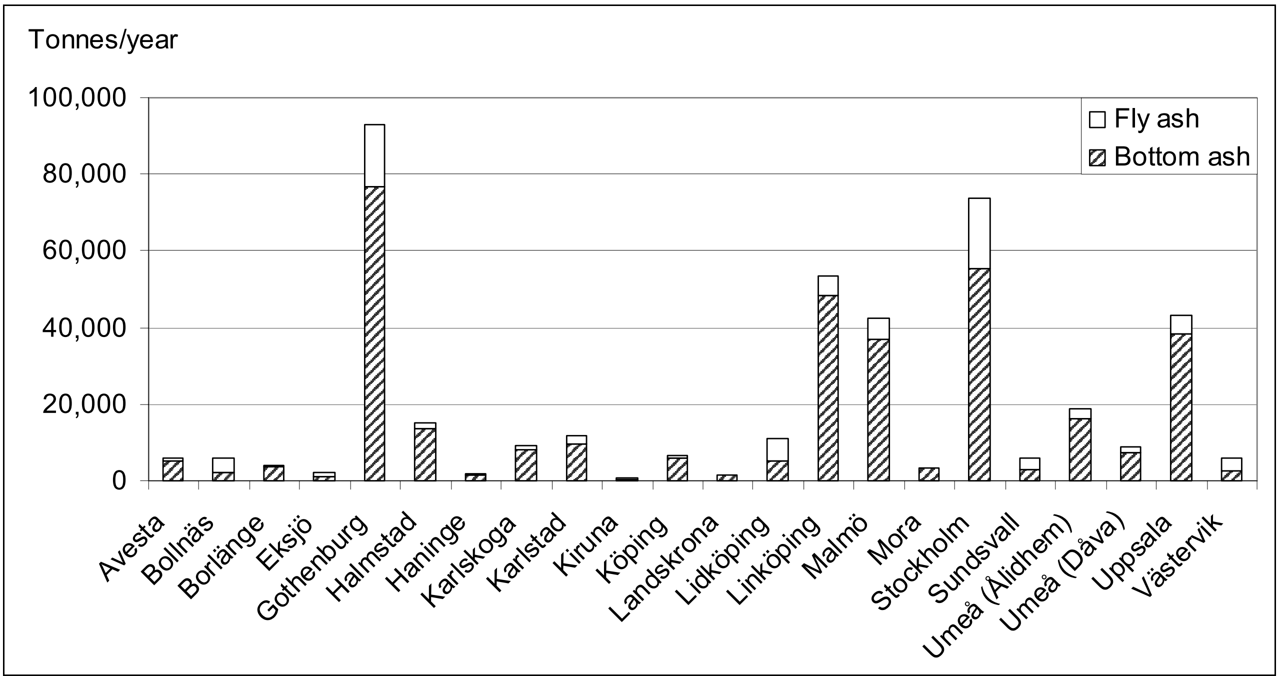


Figure 8. Residues from Swedish MSWI plants in 2000. Fly ash means APC products. (Data from RVF, 2000).

From the figure can also be seen where the plants are located. It can be concluded that the plants are spread all over the country and that most of them are small. For obvious reasons the top five are located close to large cities. Several new plants and expansions of the existing plants are planned as a result of the ban on landfill of combustible waste.

Chemical composition

The chemical composition and organic content of the bottom ash are affected by the incineration process. Important factors are temperature, redox conditions, chlorine content, content of reaction partners other than oxygen and chlorine, retention time of the waste in the furnace and mixing conditions in the furnace (Belevi, 1998). The particle size and the water content are also affected by the incineration and the subsequent quenching.

The chemical composition of MSWI bottom ash depends of course on what has been incinerated, although glass is usually a major component. Silicon dioxide, calcium oxide and aluminium oxide therefore dominate the chemical content. Other alkaline oxides are also included (Lundgren & Hartlén, 1991).

During storage, the ash undergoes alterations, such as hydration reactions, solidification reactions through the formation of calcite, sulphate reactions, salt formation reactions, corrosion reactions with iron as well as solution reactions (Pfrang-Stotz & Reichelt, 2000). The pH is also reduced during ageing due to the reaction with carbon dioxide originating either from the air or created in the biological destruction of organic material. The normal pH for MSWI bottom ash is 9.5–10 in a fresh condition and approximately 7 in an aged condition (Chandler et al., 1997).

5.2 Crushed concrete

The concrete relevant for crushing and use as unbound road material comes either from the demolition of concrete constructions or as residue from the production of concrete and concrete products. The term can therefore be divided into 'demolition concrete' and 'residue concrete'. In both cases the material can be crushed and sieved into desirable particle sizes.

'Demolition concrete' is by nature a heterogeneous material. To limit the negative impact of this inhomogeneity, modern processing includes removing light particles such as wood, plastic, paper etc. through sieving with air. Magnetic particles, such as reinforcements, are removed using magnets. A prerequisite is that the demolition is performed selectively. Figure 9 shows the crushing process used in a major construction project in 1999.

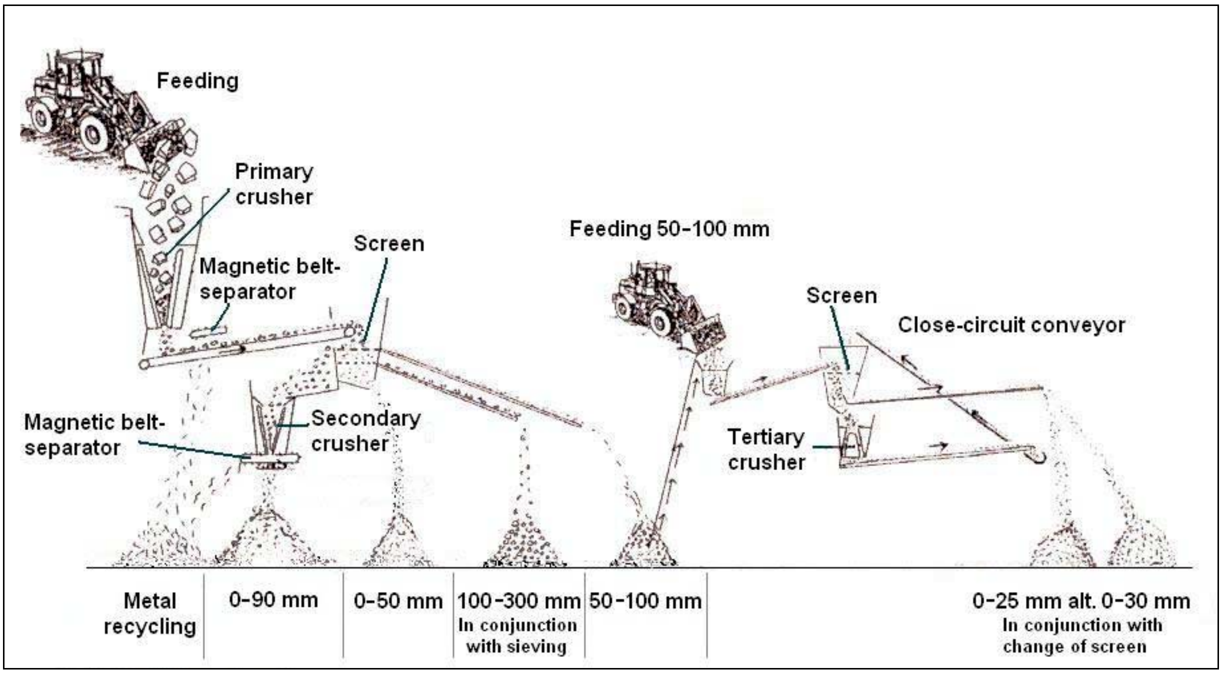


Figure 9. Flow diagram for crushing and processing of demolition concrete at Spillepeng, Stage 3 (after SYSAV, 2000).

Firstly, the demolition waste is sorted generally with an excavator, and blocks greater than 1 m are broken up with an axe-hammer or a concrete cutter. The material is then fed into the primary crusher, which is a modified rock crusher. The crushing releases a large part of the reinforcement steel. Around 90% of this is removed by means of a strong magnetic belt-separator. The crushed material is sieved into several fractions and the coarse fractions are fed into the secondary crusher, where it is crushed once again and where further reinforcement is released and removed by a second magnetic belt-separator. Finally, one more crusher has been installed to produce an even finer fraction. In the crushing plant described, seven different fractions could be produced: 0–8 mm, 0–25 mm, 8–25 mm, 0–50 mm, 0–90 mm, 50–100 mm and 100–300 mm. According to the project report, the capacity of the crushing plant was slightly lower than that of a conventional rock-crushing plant due to stoppages caused by reinforcement steel that got stuck in the transport system.

The annual volume of crushed concrete in Sweden is difficult to calculate. Arell (1997) estimated it at between 0.3 and 3 million tonnes a year, of which around 40,000 tonnes is in the form of residues from concrete plants. It is also difficult to forecast where the concrete from demolition activities will arise. On the contrary, the production locations for 'residue concrete' are well known, since they coincide with the location of concrete and concrete product production.

Mineralogy and chemical composition

Concrete consists of aggregates, cement and possible additives. 'Residue concrete' naturally has the same chemical composition as the original concrete, which is mainly silicon dioxide and calcium oxide. 'Demolition concrete', however, could contain large amounts of other materials, both on the macro- and micro-scale. These could either origin from the demolished building itself or from the surrounding filling. Examples of macro-impurities are gypsum, plastic, rubber, wood and plants that have been mixed with the concrete in the demolition process or in other forms of handling. Examples of micro-impurities are heavy metals, polyaromatic hydrocarbons (PAH) and oil that could originate from use of the original construction. Chlorides could have been added as accelerating additives or used in skid resistance treatment of the original construction.

5.3 Air-cooled blast furnace slag (AcBFS)

AcBFS is a residue from the production of pig iron in the steel industry. Pig iron (raw iron) is produced by mixing iron ore and coal in the blast furnace. Fluxing agents, often limestone, are added and combine with the silica and aluminium compounds of the ore to form the slag (Lindgren, 1992 cited in Lindgren, 1998). The slag is lighter than the melted iron and floats on top. At certain intervals the slag is tapped and taken away (Figure 10).

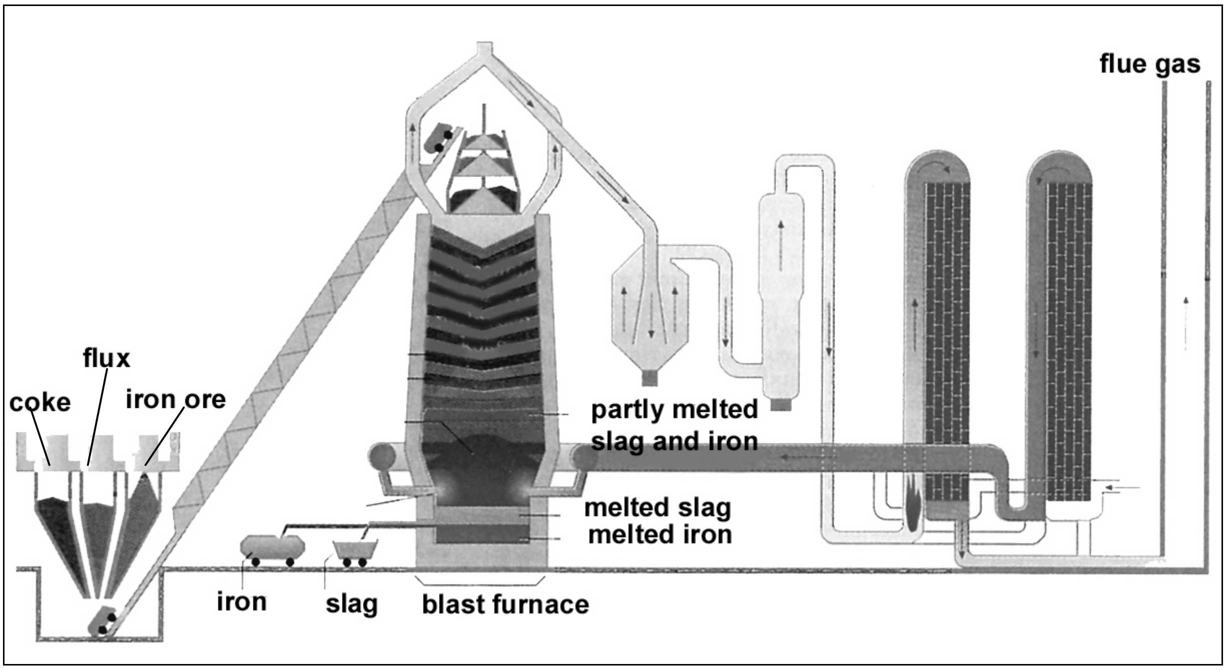


Figure 10. Blast furnace (after NE, 1994).

After air-cooling, the blast furnace slag can be crushed into a desirable particle size. If the slag is cooled in water it results in granulated blast furnace slag, which is more fine-grained and is not dealt with in this thesis.

Blast furnaces are situated at two locations in Sweden – Luleå in northern Sweden and Oxelösund in south-east Sweden. In Luleå, around 230,000 tonnes of AcBFS are generated each year, in Oxelösund around 170,000 tonnes.

Mineralogy and chemical composition

The mineral composition of AcBFS could be described as 1/3 silicon dioxide, 1/3 calcium oxide and 1/3 magnesium and aluminium oxides. The slag contains very small amounts of iron, which is quite natural, since the slag is a residue from iron and steel production. The sulphur content in fresh slag is about 1.3 wt.-%. According to Lindgren (1998), half of this is released after six months.

6 MATERIALS

The specific materials used in the study are presented here. Further details are given in Chapter 8.

6.1 Processed MSWI bottom ash

In the laboratory tests, processed bottom ash from incinerators in Stockholm, Gothenburg, Malmö, Linköping and Umeå were studied. All incinerators are of the mass-burn type, but of different ages. The bottom ash was sieved and particles greater than 50 mm and magnetic material were removed. All materials were stored outdoors during different periods.

Sampling took place in Stockholm, Gothenburg, Malmö and Linköping during four, two-week periods spread over the year (Paper II). Further sampling took place in Malmö a few years later. These samples were taken from stockpiled material on one occasion by the producer. The Umeå material was also sampled by the producer on one occasion. Since the Umeå incinerator was newly constructed this ash was stored for a shorter period than the others. The two latter materials correspond to the material used in the test roads described in Paper I.

The particle size distribution for all ash materials was similar to that of sandy gravel and the composition was dominated by slag and glass particles and the content of organic matter varied between the incinerators (Papers I and II).

In the licentiate thesis (Arm, 2000a) an older type of bottom ash from Linköping was also studied. This was produced in the 1980s and is used in a test road constructed back in 1987, but which is also investigated within the framework of this project.

6.2 Crushed concrete

Crushed concrete materials from different sources have been studied. Most of them were residues from demolished buildings. The term 'crushed concrete' is used here for material containing no more than around 10% contaminants (e.g. lightweight concrete, brick, wood, paper, plastic, bitumen etc), even if its origin is building and demolition waste.

Paper IV contains reports on investigations of crushed concrete from two sources. One, used in Road 597, with the 0–60 mm fraction, originated from a specific building that was demolished and was due to be reused as road material. The other, used in Road 109, with the 0–100 mm fraction, came from a landfill of mixed building and demolition concrete.

Paper III describes the investigations of four different concrete materials. Two were the same as in Paper IV; the other two came from specific building demolition objects in Västerås and Grums. Of the Västerås material, several different fractions were used whereas only laboratory tests on the 0–32 mm fraction were performed on the Grums material.

In Arm (2000a) results are also reported from investigations of six other concrete materials, such as crushed railway sleepers, concrete slabs and four additional demolished buildings.

All concrete materials tested in the laboratory were sampled by VTI staff and crushed in commercial crushing plants, except the Grums concrete, which was crushed in a laboratory crusher. Before the triaxial tests, all materials, except the one used in Road 597, were proportioned to the same particle size distribution, namely a curve in the centre of the approved zone for Swedish base course material. This curve was called the 'Normal curve'. The Road 597 concrete was tested with a similar curve but with a maximum particle size of 22 mm (Paper IV).

Furthermore, all materials, except the railway sleepers, were processed to remove reinforcements using magnetic separators. The railway sleepers had been crushed and stockpiled in advance without any magnetic separation. This material was used in the Stenstorp test road mentioned in Section 8.2. The material used in Road 109 was processed even further by means of wind-sieving and thus contained very few impurities.

This thesis also discusses additional data on crushed concrete from another building demolition object. This material (SU) was crushed and sampled by the producer. It did not fulfil the requirements for base course or sub-base material and was tested with its original particle size distribution but with particles greater than 32 mm removed.

6.3 AcBFS

The AcBFS, whose properties are described in Paper IV, originated from the blast furnace in Oxelösund. In the field test, crushed slag with a particle size of between 0 and 125 mm was used, whereas the laboratory-tested material was proportioned to the 'Normal curve' mentioned in Section 6.2.

6.4 Conventional materials

To relate the properties of the materials studied to something well-known, the mechanical properties of some conventional materials were used. Materials that the residues could possibly replace were chosen. These were sand, gravel and crushed rock. They had all been tested in a similar way within other projects at VTI and belong to the 'unbound material database' of VTI. The material used for comparison is described separately in each paper. In most cases material with the same particle size distribution as the residue was chosen.

7 METHODOLOGY

The methodology employed in the project can be described as follows: Information was collected through studies of the literature and study visits. Previous Swedish experience was evaluated and foreign experiences translated, thus identifying areas in which there was a lack of knowledge. Experimental work was carried out by investigating an old test road, planning and evaluating new field tests, and planning and evaluating new laboratory experiments.

Field tests

Several test roads and test sections were constructed with great optimism 15–20 years ago. One of these test roads has been studied in particular as part of this project. It was constructed in Linköping in 1987 (Jacobson & Viman, 1989; Lundgren & Hartlén, 1991) and has one section where **MSWI bottom ash** has been used in the sub-base. This road has been monitored in many ways and the results have been compared to results from measurements made when the road was new. The frost depth has been followed for two winters and FWD measurements have been performed. The measurements were performed to demonstrate possible time-dependent changes in frost depth or in the stiffness of the construction. Results from these investigations are reported in Arm (2000a). The test section and the reference section are described in Paper I and the results from the FWD measurements are reported.

Two new test roads have been planned and studied in particular within this project. Reference sections with conventional road material have been constructed on both test roads. The purpose of the investigations is to study the long-term performance of crushed concrete and MSWI bottom ash and to compare the test sections with the reference sections.

One of the roads is situated in Törringe, outside Malmö in southern Sweden. It was built in 1998, and has several test sections and two reference sections (Figure 11).

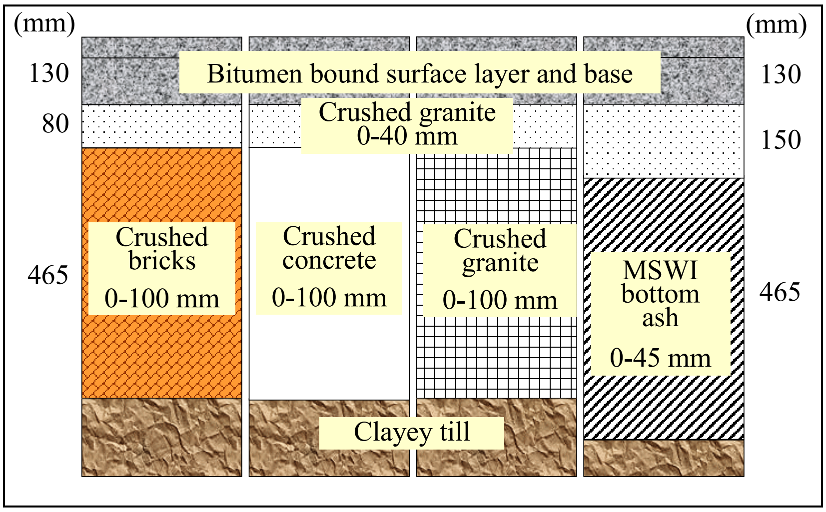


Figure 11. Cross-section of the test road in Törringe.

As can be seen in the figure, different residues are used in the sub-base. In some other sections there is also crushed concrete in the base course. In this project the sections constructed with **MSWI bottom ash** and **crushed concrete** in the sub-base have been studied. FWD measurements have been performed on several occasions, frost depth has been registered and environmental characterisation of the actual sub-base materials has been made. The FWD measurements on the MSWI bottom ash section are reported in Paper I and the other investigations in Arm (2000a).

The other test road is situated in Umeå in northern Sweden and was built in 2001. This road also has **MSWI bottom ash** in the sub-base and is the subject of several environmental studies performed by SGI, KTH, Geological Survey of Sweden (SGU), and Luleå University of Technology. Within this project, FWD measurements and frost depth measurements have been carried out. The build-up of the test sections and the results of the FWD measurements are reported in Paper I.

In addition to these test roads, it has been possible to analyse data from VTI's other field tests on **crushed concrete** and **AcBFS** within the framework of the project (Papers III and IV). In Paper III, the results are reported from repeated FWD measurements on three test sections with crushed concrete in the sub-base. Paper IV describes investigations conducted on two test roads with AcBFS in the sub-base – one old and one new. FWD measurements were carried out and material from the test pits was analysed.

On four of the tests roads with crushed concrete mentioned above, additional FWD measurements were performed in 2000.

Laboratory tests

Within this project an extensive study of **MSWI bottom ash** from different Swedish incinerator stations has been carried out. The investigations included analysis of particle size distribution, analysis of composition and evaluation of resilient modulus, M_r, and bearing capacity by means of cyclic load triaxial tests (Papers I and II). In addition, the durability properties were studied through two standard tests, namely the micro-Deval test and the freeze-thaw test (Paper I).

Variations in deformation properties of the ash were expected due to seasonal variations in the waste. Different incinerators were also believed to affect the mechanical properties of the resulting bottom ash. The investigations therefore comprised bottom ash from five incinerators and different sampling periods during the year. Statistical evaluation for four of the incinerators was performed by means of analysis of variance ANOVA (Paper II). The impact of stress and water content on resilient and permanent deformation was analysed and is discussed in Paper I.

Crushed concrete has been studied in several projects at VTI, where composition analysis, compaction tests, cyclic load triaxial tests and FWD measurements have been carried out (Johansson et al., 1996; Ydrevik et al., 1996; Ydrevik, 1999a–d). Results in the form of 'raw data' from these investigations have been compiled and analysed within this project and are reported in Arm (2000a) and in Papers III and IV. In Arm (2000a) laboratory results and field results in the form of M_r, permanent deformation and layer modulus for concrete materials from different sources were reported. Strength increase by time and the effects of source and processing were discussed and a comparison was made with natural aggregates. In Paper III the effect of storage time on laboratory-evaluated resilient modulus and field-measured layer modulus is compared and described. Paper IV compares durability according to standardised methods, such as the Los Angeles, Ball mill and freeze-thaw tests, with performance evaluated by means of cyclic load triaxial tests and FWD measurements.

In this thesis laboratory results from visual classification, chemical composition and cyclic load triaxial tests from two new concrete sources completed the study.

AcBFS has not been investigated solely for this project. Results from the Swedish part of the European ALT-MAT project formed the basis of Paper IV (TRL, 2000). The paper reports results from cyclic load triaxial tests on fresh and aged slag and from standard durability tests on fresh slag.

7.1 Cyclic load triaxial tests

The cyclic load triaxial test is a laboratory method that is being used increasingly, especially in the context of performance testing of unbound aggregates. It is based on the principle that the entire material, up to a certain particle size, e.g. 64 mm, is exposed to a simulated traffic load, whereupon the deformation is recorded. Cylindrical specimens are compacted and exposed to cyclic loads of different magnitudes. The stiffness or load-spreading ability can be evaluated from the resilient strain and the result is expressed as a resilient modulus in different stress states. The permanent deformation appearing in the material at different loads can also be evaluated and used as a stability measurement. A CEN standard is under development (prEN 13286-7) and the method will be used for evaluating unbound granular materials in the future. The draft pre-standard describes different loading procedures. In this study, the testing equipment at VTI was used and tests using constant confining pressure (CCP) were performed. The procedure used at VTI is most similar to the so-called multi-stage procedure, where the same specimen is exposed to several stress levels (Ydrevik, 1996).

The procedure is based on applying a cyclic axial (vertical) load to a cylindrical specimen under static radial (horizontal) pressure. Vertical and horizontal loads represent the imposed traffic load and overburden pressures from overlying pavement layers. In the tests, the specimen diameter was 150 mm and the height was 300 mm (Figure 12).

The target testing conditions for crushed concrete and AcBFS were water content around 60% of optimum and density around 90% of maximum density obtained in a modified proctor test. For MSWI bottom ash the corresponding values were optimum water content and around 90% of maximum density. The specimens were compacted with simultaneous vibration and compression in a Vibrocompressor (EN 13286-52).

After compaction the specimens were pressed out of the compaction cylinder and equipped with top plates and a thin rubber membrane around the skin. After storage for around 24 hours the specimen was placed in a triaxial cell, where air was used as a medium to create the horizontal stress that simulates the surrounding soil pressure. The cyclic load was obtained from a hydraulic cylinder and simulated the impact from the traffic load. The cyclic load was varied according to a sine-shaped wave at a frequency of 10 Hz without rest periods. This gave a loading time of 0.1 second, which should correspond to passage by a truck travelling at around 70 km/h.

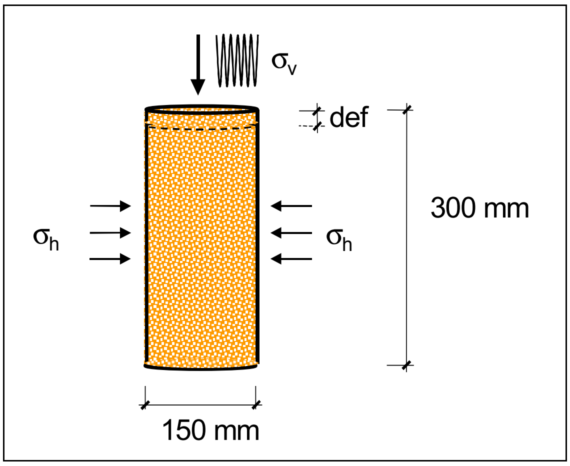


Figure 12. Principle of cyclic load triaxial tests used in the study.

Every specimen was exposed to several consequential loading sequences. Two types of tests were used, the 'capping-layer test' and the 'base course test'. In the first test, the vertical cyclic stress, $\sigma_{v,c}$, was varied between 10 and 150 kPa and the horizontal stress, σ_h, was kept constant at 10 or 20 kPa, resulting in a deviator stress, q, of between 30 and 170 kPa. In the second test, $\sigma_{v,c}$ was varied between 120 and 1,220 kPa and σ_h was kept constant at 60 or 120 kPa, resulting in a q between 140 and 1,240 kPa (Ydrevik, 1996).

Each test was continued until all the sequences were completed or until the permanent deformation registered at one single sequence exceeded 20 mm. In such a case the specimen was considered as failed and the test was stopped automatically.

During the test, the vertical deformations (both resilient and permanent) were measured and stored. The vertical deformation was registered by means of an external LVDT (= Linear Variable Differential Transducer). This means that the deformation was measured over the total specimen height. Horizontal deformation was not measured. The strains were calculated from the deformations and the resilient modulus, M_r, was calculated from the resilient strain, $\varepsilon_{v,r}$, using the equation (2)

$$M_r = \frac{\sigma_{v,c}}{\varepsilon_{v,r}} \qquad (2)$$

where
M_r = resilient modulus
$\sigma_{v,c}$ = vertical cyclic stress
$\varepsilon_{v,r}$ = vertical resilient strain

The values discussed in this thesis are the averages of two and sometimes three specimens of each material.

7.2 Falling weight deflectometer (FWD)

Most, but not all, FWD measurements reported in Papers I, III and IV were carried out by VTI according to VVMB 112:1998. Some measurements were performed by different consultants. The VTI measurements were carried out with a load of approximately 50 kN applied on a plate with a diameter of 300 mm. Six seismometers were used (Figure 13).

The moduli were estimated through back-calculation using ITERCHEV and CLEVERCALC software.

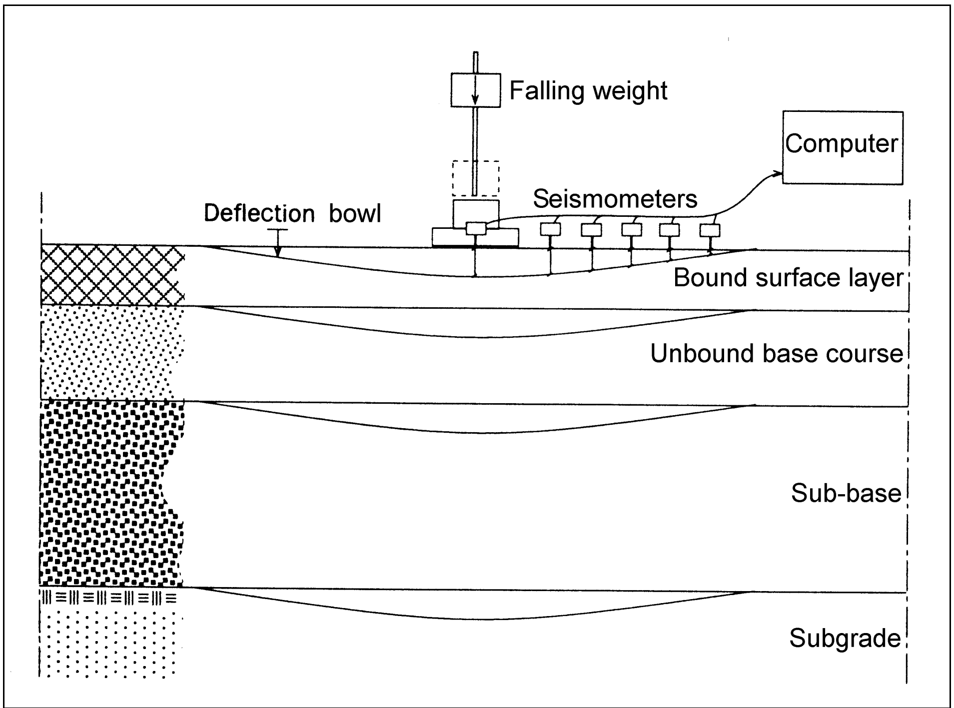


Figure 13. Principle figure of the VTI FWD measurement (after Carlsson, personal communication, 2000).

7.3 Content of organic matter

In conjunction with other investigations in the project the content of organic matter was investigated for the materials studied. This was performed by means of colorimetric measurement (SS 02 71 07), as prescribed in the predecessor to ATB VÄG, VÄG 94 (SNRA, 1994). However, in the literature it is almost solely the results from ignition that are presented when the content of the organic matter in the residue is discussed. A minor study was therefore performed to compare the results of different methods.

The content of organic matter in three materials was measured using three different methods, colorimetric measurement and ignition at two different temperatures, 550˚C and 950˚C (SS 02 81 13, SS 02 71 05). The materials were processed MSWI bottom ash, crushed concrete and crushed granite. All three were used in the test road in Törringe. The results are presented and discussed in Section 8.5.1.

7.4 Composition of materials

In order to characterise the residue, simplify the comparison with natural aggregate and to explain possible differences in the properties, the composition of MSWI bottom ash and crushed concrete was investigated. The AcBFS was regarded as homogenous and was not analysed in this project.

The main constituents of MSWI bottom ash were analysed through visual classification of the 4–32 mm fraction. The classification method and the categories were partly chosen according to earlier studies (Höbeda et al., 1985). All particles in a 600–900 g sample were classified into slag, glass, ceramics, metal or other. (Höbeda et al. used "non-magnetic" instead of slag). Each class was weighed and the weight percent of the total was calculated. The term 'slag' means slag particles with no visible glass, ceramics or metal. The 'Other' category includes non-combusted matter, such as paper, plastic, textile and wood.

The crushed demolition concrete was also analysed through visual classification of the 4–32 mm fraction. The sample size was around 1 kg and in this context the categories were chosen to reflect the 'purity' of the material:

- Aggregate + mortar (aggregate particles with remaining mortar)
- Mortar
- Clean aggregate particles
- Bricks, tiles
- Lightweight concrete (density <1.6 tonnes/m^3)
- Other (wood, plastic, paper, glass, metal, bitumen)

This method and the categories chosen differ from the method suggested in the draft for a European standard (prEN 933-11), where particles greater than 8 mm are sorted into asphalt, building materials, concrete and concrete products, lightweight materials with a density of <1 tonnes/m^3, unbound aggregates or other. However, the prEN is still a draft and subject to national member comment.

8 RESULTS AND DISCUSSION

In this chapter results from the different investigations performed in the project are presented and discussed. In Sections 8.1 to 8.3 results from laboratory and field tests on the three materials studied are discussed. Section 8.4 contains a suggestion for material assessment based on deformation properties. Section 8.5 deals with results regarding test methods and discusses the suitability of the present standard test methods, especially for residues.

8.1 Results from tests on processed MSWI bottom ash

The MSWI bottom ash materials were well-graded with a particle size distribution similar to that of sandy gravel. The main part consisted of slag and glass particles.

According to cyclic load triaxial tests the ash materials showed resilient deformation properties similar to that of natural gravel but showed less permanent deformation due to more angular particles. The stress-dependency of M_r was slightly positive, as is the case for sand and natural gravel. It was obvious that a high content of unburned material limits the stiffness. The water susceptibility of deformation properties showed significant differences between the two ashes studied and thus need to be investigated further. The deformation properties expressed as calculated resilient modulus and measured permanent deformation from triaxial tests were reasonably uniform for each MSWI station. There was a significant difference between the stations but not within a station. The difference in deformation properties between the stations was most likely caused by different organic content.

All ash materials showed greater fines formation and disintegration than natural aggregates when tested in micro-Deval tests and freeze-thaw tests. This must be taken into account in construction when choosing the compaction tool and when planning for construction traffic, whereas the final traffic only has a very slight impact if the road is designed properly.

According to the FWD measurements, initially and one year after construction, the mean stiffness of the bottom ash sections was lower than for the reference sections with crushed granite. Slow stiffness increase with time was observed in a 12-year-old bottom ash layer but not yet in the one-year-old test sections. This means that the increase is too slow to be taken advantage of in the design.

Finally, the technical usability of MSWI bottom ash increases significantly if the organic content can be kept low. The ash can thus be used not only in embankments and capping layers but also in a sub-base, if the road is designed properly.

8.1.1 Laboratory tests

This section presents the results on the particle size distribution, the material composition, the organic matter content, the deformation properties and the resistance to wear and disintegration.

Particle size distribution

All MSWI bottom ash studied, screened to a 0–50 mm fraction, was by nature well-graded materials (Papers I and II) (Figure 14).

The thick lines in Figure 14 mark the limits of a zone in which the particle size distribution of all the bottom ash studied fell. The particle size distribution is similar to that of sandy gravel with a fines content of between two and ten percent.

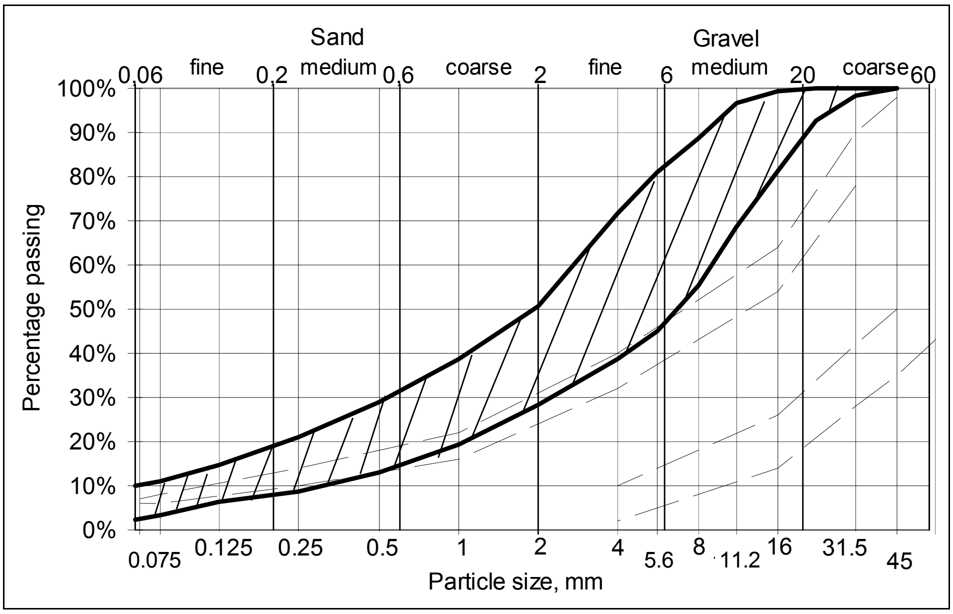


Figure 14. Particle size distribution for the MSWI bottom ash studied. A zone for 18 samples from five incinerator plants. The thin line means the outer and inner zone for sub-base material according to ATB VÄG.

Material composition and organic matter content

The sieved and stored ash material consisted mainly of slag and glass particles (Paper I). Approximately 80 wt.-% of the particles in the 4–32 mm fraction was characterised as slag or glass. The rest was ceramics, metal, paper, plastic, textiles and wood. The organic matter content varied between 2.3 and 9.2 wt.-%, measured as LOI at 550°C depending on the incinerator and the sample period (Paper I). When comparing the material composition of bottom ash arising today with bottom ash from 1987, as reported in Lundgren & Hartlén (1991), two main differences were noticed (Arm, 2000a). The proportion of glass in the Malmö bottom ash had decreased from 62 to 34 wt.-%. The organic matter content had also decreased. It was hard, however, to quantify the decrease since different ignition temperatures were used.

Chemical analysis showed that the principle elements were silicon, calcium, aluminium and iron (Figure 15). In addition sodium and potassium were present. Chlorides and sulphates were also detected.

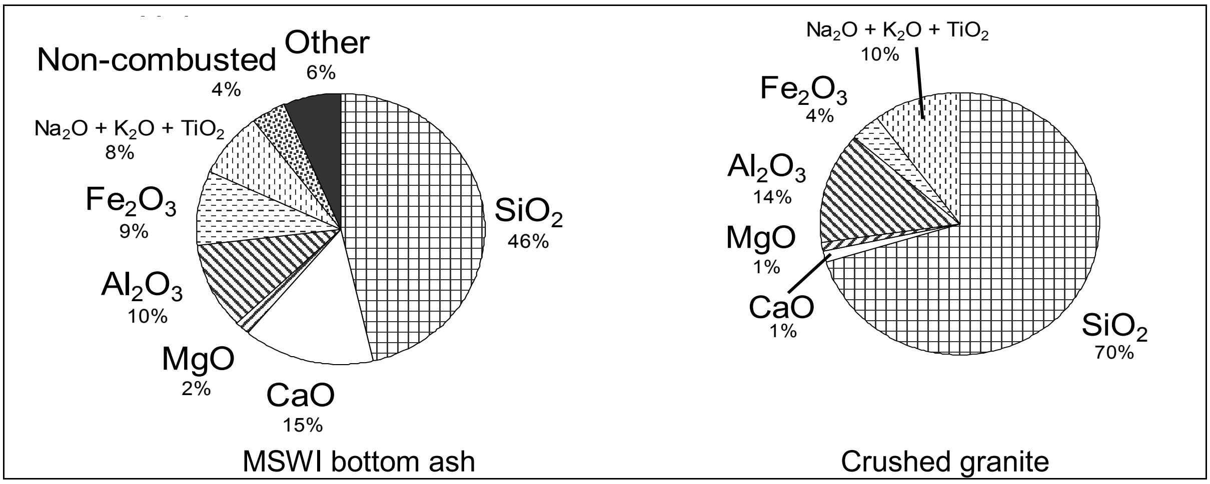


Figure 15. Chemical composition of MSWI bottom ash and crushed granite from the test road in Törringe/Malmö (after Arm, 2000a).

Deformation properties

When tested in cyclic load triaxial tests at low stress levels (p <75 kPa) the bottom ash showed resilient deformation properties similar to that of sand (Paper I). This was valid both for the value of resilient modulus, M_r, which ranged from 75 to 150 MPa, and the stress-dependency, which was slightly positive. One single ash obtained an M_r of between 175 and 225 MPa, which was explained by a longer storage time and subsequent ageing before testing. Two ash materials that were tested at higher stress levels (p =100–150 kPa and q/p =1.7) performed like crushed granite, whereas at stress levels above that, they performed similar to natural gravel and obtained an M_r of 200–300 MPa. These two ash materials did not fail until the stress reached stress levels beyond what was expected in a Swedish sub-base.

The load-bearing capacity expressed as the maximum load that could be carried without excessive permanent deformation was much better than that of sand and better than that of natural gravel. This was attributed to the angular particles of bottom ash.

The impact of water content on deformation properties was not unequivocal (Paper I). The stiffness of the 'M –99' ash, was increased and reached a maximum at optimum water content (evaluated from modified proctor compaction), whereas the 'U – 01' ash had a lower stiffness when the water content was increased. The permanent deformation result revealed the same difference between materials, with the U ash deforming more and more with increasing water content and the 'M –99' ash being more insensitive to both water content and stress level.

The investigation reported in Paper II showed that the resilient deformation properties were influenced considerably by organic matter content. A high proportion of non-combusted material was detrimental to the resilient modulus. The M_r increased by 50% (from approximately 80 MPa to 120 MPa) when the organic matter content was decreased from 9% to 4% LOI at 550°C. These findings were confirmed in the study in Paper I. The permanent deformation, however, was not influenced to the same extent.

Resilient modulus varied significantly between incinerator plants, but no significant variation was found between different sampling periods within the plants (Paper II). Neither were there any significant variations in permanent deformation within different sampling periods or between incinerator plants. The variation in resilient modulus was explained by the variation in the non-combusted material content.

Resistance to wear and disintegration

The MSWI bottom ash materials were more sensitive to wear and disintegration than crushed rock and sandy gravel (Paper I) (Arm, 2000a). This was obvious when tested using standardised laboratory methods such as the micro-Deval, Los Angeles and freeze-thaw tests. Lower resistance was shown in greater fine-material formation and disintegration. The ash obtained the following results: Los Angeles value 45% (one material), micro-Deval value 26–39% (seven materials), freeze–thaw test with distilled water 0.8–7.7% disintegration (seven materials). Furthermore, compaction according to modified proctor produced significant disintegration. The low resistance to wear and freeze-thaw changes was attributed to the porous particles. This conclusion was confirmed by the good relationship found between the disintegration and the content of porous slag particles (Paper I).

8.1.2 Field tests

According to the field tests described in Paper I, processed MSWI bottom ash initially had lower stiffness than crushed rock. In the field tests in Linköping and Malmö the stiffness was expressed as the layer modulus determined by FWD measurements. The ratio between layer moduli in test sections with bottom ash in the sub-base and layer moduli in reference sections with natural aggregates was 68–79% during the first year (Table 3). In the field tests in Umeå the ratio between the calculated surface moduli for each deflection point (E0, E200, E300 etc) was used instead.

Table 3. ***Results from FWD measurements on the test roads in Linköping, Malmö and Umeå. Ratio between layer moduli in the bottom ash section and layer moduli in the reference section. B = base, SB = sub-base, SG = subgrade. (Data from Jacobson & Viman, 1989; Jansson, personal communication, 1999 and 2000; VV-konsult, 2002).***

Linköping Constructed in 1987		**Malmö** Constructed in 1998		**Umeå** Constructed in 2001	
Date	Ratio between layer moduli	Date	Ratio between layer moduli	Date	Ratio between surface moduli
1987-11	55% (SB+SG)	1999-06	68% (B+SB) 90% (SG)	2002-09	77% (SB) 88% (SG)
1988-05	79% “	2000-06	64% (B+SB) 82% (SG)		
1988-10	92% “				
1999-08	101% (SB) 143% (SG				

An increase in stiffness over time was observed in MSWI bottom ash produced in the 1980s (Table 3). Stiffness expressed in terms of layer modulus from FWD measurements had doubled after about one year, which meant that the initial difference in relation to natural aggregate had vanished. A similar increase has not been noted in the two materials with a later incineration date. It should be noted that the development for the combined layer of base and sub-base in the Malmö bottom ash section should not be regarded as a decrease since the relationship between subgrades decreased even more.

8.1.3 Discussion

According to the field results, the MSWI bottom ash studied, which by definition is of 0–50 mm particle size, had lower stiffness than crushed rock with 0–100 mm particle size. This is partly due to the smaller particle size of the bottom ash. On the other hand, MSWI bottom ash performs well during spreading and compaction due to the well-graded curve and the angular particles, both being parameters that indicate a stable unbound material. However, even if the bottom ash materials were well-graded and might be compacted to a stable layer they did not meet the present Swedish specifications for sub-base materials. They were all too fine-grained (Figure 14) and the micro-Deval value was too high for three of seven materials tested if used in a sub-base that is not exposed to construction traffic.

It has been suggested that bottom ash should be *processed even more* in such a way that the fine fraction or even the fraction up to 2 mm should be removed in order to improve the environmental impact properties. The background is that 90% of the pollutants are found in the finest particles (Barbery & Ghodsi, 2000). However, it should be observed that this action changes the bottom ash into a more single-sized material that is less stable and less resistant to permanent deformation and also more sensitive to degradation through crushing. A solution could be to remove the fine fraction <2 mm and replace it with a fine fraction that is less polluted. Perhaps other residues, such as foundry sand or the 0–2 mm fraction from crushing plants, are usable alternatives.

The *water susceptibility* of deformation properties of the materials studied was not clearly analysed and must be investigated further and in other ash materials. The preliminary explanation of the results was that the target water content was not obtained in the most fine-grained material and that this ash was drier than expected during the test.

Material composition and inhomogeneity

The *material composition* of MSWI bottom ash depends on the waste incinerated, but glass is often a main constituent. It was therefore not surprising that Swedish MSWI bottom ash is also a calcium-aluminium silicate, as well as the German bottom ash (Pfrang-Stotz & Reichelt, 2000) and bottom ash from the USA, Canada, Switzerland, Denmark and the Netherlands (Chandler et al., 1997). When compared with natural aggregate the main compounds are the same but the proportions differ (Figure 15). This conclusion is also drawn by Chandler et al. (op.cit.), who report that calcium, aluminium, silicon and iron are represented to the same extent in bottom ash as in the lithosphere and in the crust. However, chlorine, zinc, copper, lead and chromium are often present in higher contents in bottom ash since they are widely used in manufactured products that finally become waste.

One of the main characteristics of waste and MSWI bottom ash is said to be its *inhomogeneity* (Chandler et al., op.cit.). This was the reason for the study in Paper II. The result showed that with a thorough incineration process, resulting in a low organic matter content, together with careful and standardised sorting and sieving, the parameters that have a major impact on the deformation properties could be controlled and the variation in stiffness and stability could thus be kept low. Even when bottom ash from four different incinerator plants was included in the study in Paper II, the only significant variation between ash materials that was found, was the variation in resilient modulus in cyclic load triaxial tests between incinerator plants.

Organic matter content

The variation in resilient modulus, M_r, between incinerator plants was explained mainly by different *organic matter content.* A clear relationship was thus found between the organic matter content and M_r. Reduction of the organic matter content from 9% to 4% (LOI at 550ºC) might double the M_r. This was not surprising, since it is well known that organic matter has a detrimental effect on the resilient deformation properties of conventional material. Most countries have therefore limited the permissible organic matter content in road materials. The requirement in ATB VÄG is expressed as not more than 2% organic matter measured using the colorimetric method. A further description of the requirements is found in Section 1.3 of this thesis.

On the other hand no clear relationship was found between organic content and permanent deformation. This could be explained by the fact that permanent deformation depends mainly on the particle size distribution. All materials tested were well-graded and therefore stable.

All countries that already have guidelines for the use of MSWI bottom ash, have limits for permissible organic matter content. The limit is mostly expressed as a maximum value from loss on ignition, e.g. 5.5% for certified MSWI bottom ash in the Netherlands (CROW, 1999). Denmark has two limits, a maximum of 1.5 wt.-% non-combusted material in the particle fraction 16–50 mm (Pihl & Milvang-Jensen, 1996) and a maximum of 3% TOC (Miljø- og Energiministeriet, 2000).

The impact of organic matter should be borne in mind when regulating the incineration process. At present, there are no general Swedish limits for organic content in the bottom ash. There must be such a limit in 2005, when the EG directive for landfill of waste is introduced. However, these limits will be set up with another purpose, namely to determine a suitable landfill category.

Resistance to wear and disintegration

The Los Angeles value of 45% was in agreement with what has been found in the literature. LA values for MSWI bottom ash between 10 and 90%, with typical values around 40–45% have been observed (Chandler et al., 1997; Pihl & Milvang-Jensen, 1996). The durability results give a hint with regard to handling in conjunction with use. They could be interpreted as sensitivity to heavy compaction tools. On the other hand, the low *durability* is of no importance for the completed road if the pavement is properly designed.

The low *resistance to freeze-thaw* changes can be attributable to high porosity, specific mineral composition and relatively weak carbonate bond between the constituents (Pfrang-Stotz & Reichelt, 2000).

Field tests

In the field tests reported in Paper I, *stiffness increase* over time has so far only been observed in the bottom ash produced in the 1980s. The difference between old and new bottom ash could nowadays possibly depend on more careful removal of fly ash and grate siftings. The calcium content in those products is supposed to be the reason for the increase in stiffness. Future field measurements will show if the difference remains.

One attempt to determine *disintegration* of old bottom ash in the field was made in the test road in Linköping (Paper I) within another project (SNRA, 2001). Sampling took place in the sub-base materials (bottom ash and reference material of gravel) and possible changes regarding particle size distribution were investigated. However, the sampling was done by digging up the bottom ash and this produced results that were difficult to interpret, since disintegration may have been caused by the actual digging through the layer, which had become partly bound over time (Arm, 2000a).

Rut depth measurements and static plate bearing tests have been performed on the test road in Malmö and will be reported later.

Environmental characterisation

Environmental characterisation of the ash materials used in this study was also performed. Fällman et al. (1999) and Fällman (2000) reports the results for ash from the four incinerators described in Paper II. In the *leaching test* at L/S 10, 87–91% of the leachate consisted of calcium, sulphur, chlorine and sodium. Copper dominated among the trace elements. The results from the last incinerator (U) are reported in Nilsson & Larsson (2002). Leaching results on the old bottom ash used in the test road in Linköping described in Paper I are reported in Arm (2000a) with references to both the initial investigation in 1987 (Lundgren & Hartlén, 1991) and the monitoring investigation in 1998 (SNRA, 2001).

MSWI bottom ash in the future

It is likely that there will be an increase in waste incineration, which means that more bottom ash will need to be driven to landfill. This is all the more reason to find fields of application for MSWI bottom ash, but in order to avoid the expensive waterproofing measures that are required for environmental reasons, the product must be *refined.* Such refinement will also be favourable to its engineering properties.

A lot of research has been devoted to investigation of the MSWI bottom ash currently being produced. When attempts are made to find different techniques for using bottom ash, it is sometimes found that excessive quantities of one or more substances are leached out. The solution tried then is to insulate or encapsulate the material in some way, but these measures are *passive* and do not tackle the actual problem. Where do the noxious substances come from? What does the waste contain? What happens during incineration? To what extent are the substances concentrated in the bottom ash? How can the composition of the waste be changed? How can the incineration process be altered so as to enhance the quality of the bottom ash?

Considerable progress has been made during the last decade regarding the knowledge of, and the technology for, *flue gas cleaning* at waste incineration plants, owing to the extensive requirements regarding the emissions into the air in the form of flue gases. Technology for the improvement of the MSWI bottom ash has not had the same attention so far. Only a few studies of the incineration process have been made on the laboratory scale and through theoretical reasoning (Belevi, 1998 referring to Fernandez et al., 1992 and Verhulst et al., 1996). Nor are there any requirements for this residual product, presumably because it is, after all, only going to be driven to landfill.

In view of the demands being made by an ecocyclic society, the incineration plants of the future will not be able to regard themselves as pure energy-producers. Fundamental practical studies of the incineration process are required in order to *increase the usability* of MSWI bottom ash and thus reduce the need for landfill.

8.2 Results from tests on crushed concrete

The origin and handling of the concrete influences the mechanical properties of the crushed material. Porous cement as well as foreign weak particles, such as lightweight concrete and brick, but also wood, plaster and reinforcement, reduces quality. Particle size distribution with a large maximum particle size as well as a well-graded curve is positive, as is the case for natural aggregates.

According to cyclic load triaxial tests and FWD measurements, crushed concrete initially had the same resilient modulus as crushed rock (granite, gneiss and limestone). The M_r was less stress-dependent than was the case for crushed rock.

Both laboratory and field results showed an increase in stiffness for unbound layers with crushed concrete, which is not present for unbound layers with natural aggregates. This increase was considerably larger in the field tests than in the laboratory test. The increase was greatest during the first months and then diminished. This meant that the layer modulus two years after construction was about twice as high as the level after one month.

A low degree of carbonation in the original concrete yields faster carbonation and resulting stiffness increase in the compacted layer of crushed concrete. A lot of masonry and natural aggregate is limiting since these materials do not carbonate themselves. A long contact period between water and concrete particles, together with a large particle surface, which means a dense grading rich on fines, is favourable.

When tested with standardised laboratory methods, crushed concrete had lower resistance in the form of a greater proportion of fines formed and greater disintegration than gravel and crushed granite. The resistance to mechanical impact depends on particle shape and indirectly on the way of crushing. The more flaky, the lower the resistance. The resistance is also affected by the amount of foreign material in that a lot of masonry and lightweight concrete decreases the resistance. High strength in the original concrete yields better resistance to wear. If the concrete material is clean, the fines produced are not plastic as in natural aggregates, but contribute to the stiffness increase mentioned above.

For demolition concrete to be used for high-quality purposes, selective demolition must be carried out. Chemically-contaminated concrete must not be re-used. When crushed concrete with very few impurities is to be used as a Swedish sub-base material, at least the same design modulus could be used as for natural aggregates. However, a special investigation of the specific concrete material, together with knowledge of the pavement planned, could allow a higher value of the modulus to be used and thus benefit from the increasing stiffness caused by self-cementing.

8.2.1 Laboratory tests and field tests

This section presents the results on the particle size distribution, the material composition, the deformation properties, the stiffness increase and the resistance to wear and disintegration.

Particle size distribution and material composition

Both ‘demolition concrete’ and ‘residue concrete’ can be crushed into a desirable particle size suitable for the application in question. The particle size distribution used for the majority of the concrete materials exposed to cyclic load triaxial tests evaluated in this study was close to the so-called ‘Normal curve’. This means a curve in the centre of the approved zone for Swedish base course material (Figure 16).

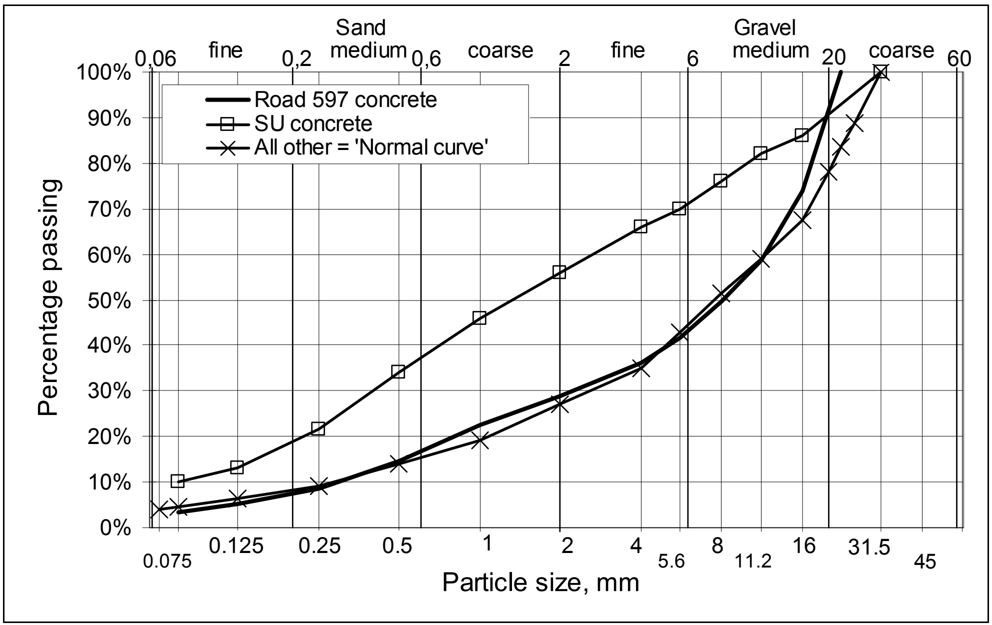


Figure 16. Particle size distribution for all concrete materials studied in the laboratory.

One material, the SU concrete in Figure 16, was tested with a particle size distribution that did not fulfil the requirements for base course or sub-base material.

According to visual classification of the 4–32 mm fraction, none of the concrete materials contained more than 11 wt.-% contaminants (Figure 17).

The concrete material in the Törringe test road described in Paper I consisted mainly of silicon, calcium, potassium and sodium (Figure 18).

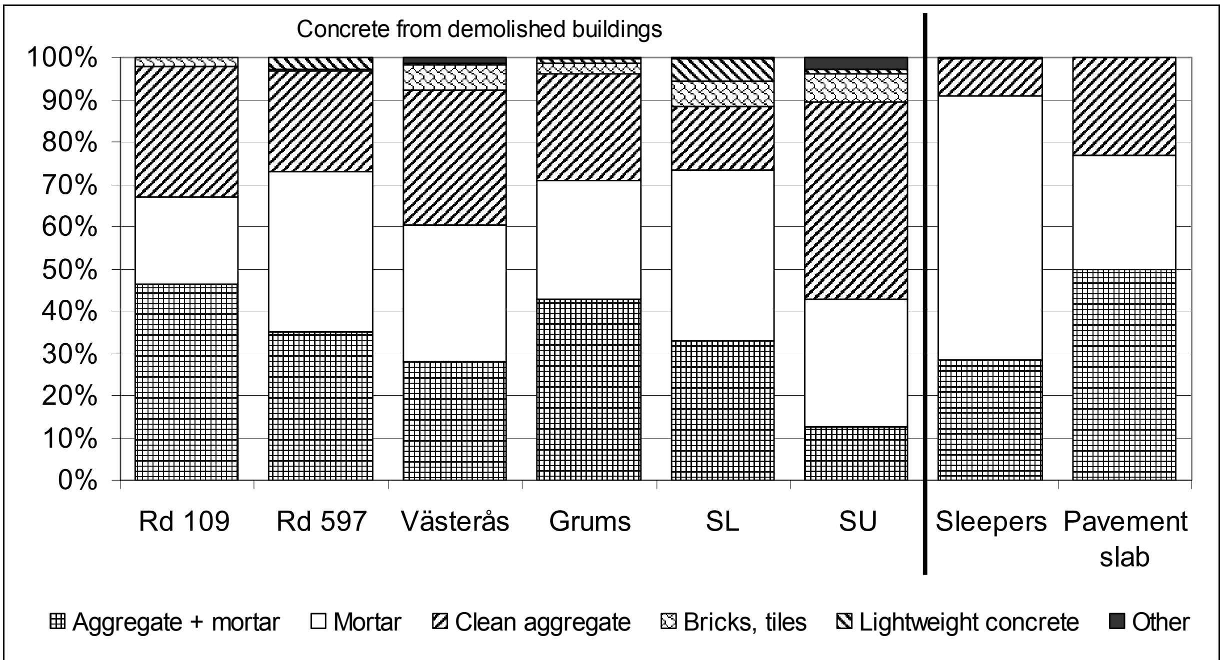


Figure 17. Result of visual classification of concrete materials (wt.-%).

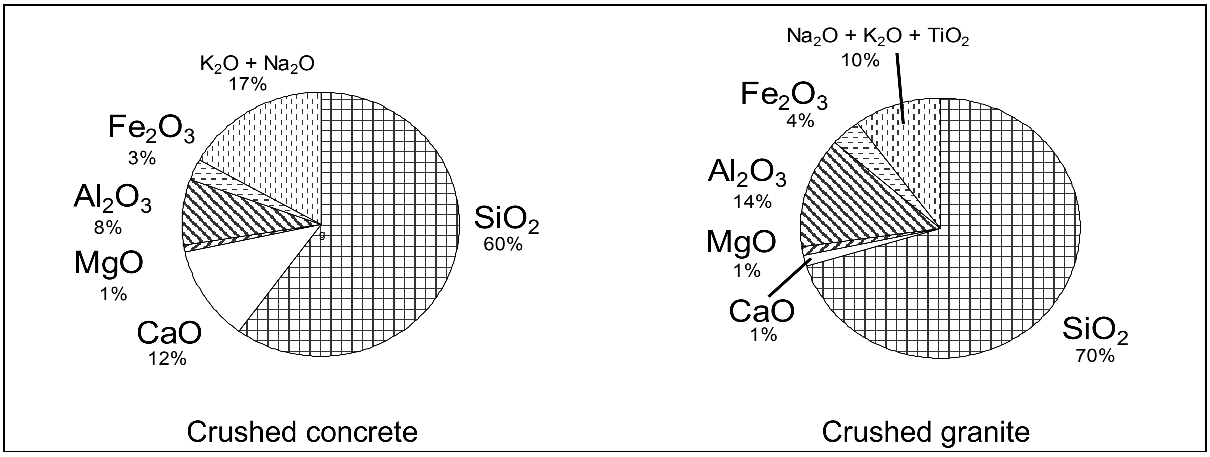


Figure 18. Chemical composition of crushed concrete and crushed granite used in the test road in Törringe/Malmö (wt.-%).

Modified proctor tests on all triaxial-tested concrete materials reported in Arm (2000a) resulted in dry density values between 1.83 and 1.98 tonnes/m^3 and an optimum water content of between 7.7 and 13.2%.

Deformation properties

According to cyclic load triaxial tests, crushed concrete initially had the same stiffness as crushed rock and was much stiffer than sand (Arm, 2000a). It was also stiffer than natural gravel. Stiffness expressed as resilient modulus, M_r, increased when the load was increased (Figure 19), but it was not as stress-dependent as the stiffness of crushed rock.

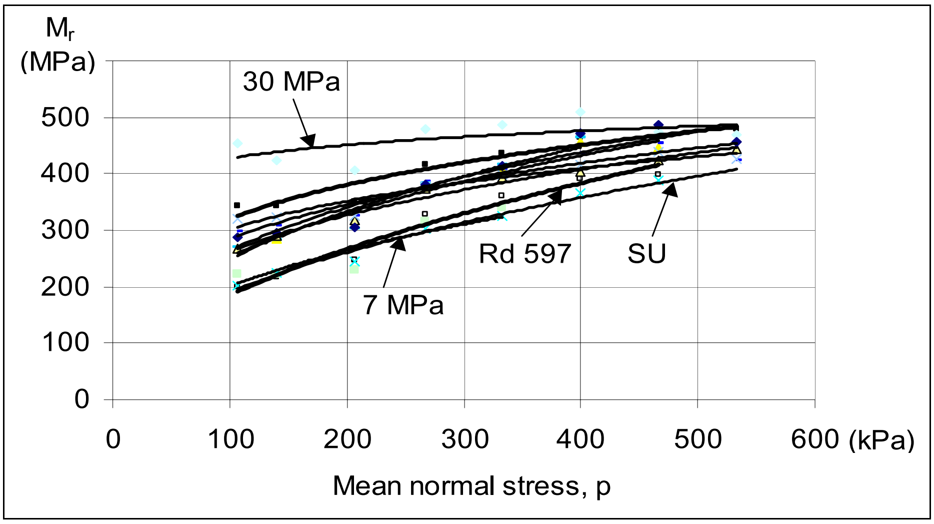


Figure 19. Resilient modulus, M_r, from cyclic load triaxial tests on eleven crushed concrete materials of different origin. (Arm, 2000a and data on one additional concrete material).

The largest M_r and also the less stress-dependent M_r was obtained by a clean building concrete with a compressive strength of 30 MPa. A lower M_r than the others was obtained by a clean building concrete with a compressive strength of 7 MPa and also by the demolition concrete used in Rd 597, which had smaller maximum particle size and a lower density than the others. Lower M_r was also obtained by the SU concrete, which contained more fines, more contaminants and more clean aggregate particles than the other.

Load-bearing capacity varied with origin in the same way as the M_r, and was as good as, or inferior to, that of crushed rock.

According to field tests described in Arm (2000a) and Papers III and IV, crushed concrete initially had about the same stiffness as crushed rock, expressed in terms of layer modulus from FWD measurements.

Stiffness increase

Arm (2000a) and Paper III contain reports of how stiffness in layers of crushed concrete increased over time in both laboratory and field tests. A similar increase was not present for unbound layers with natural aggregates. In the laboratory, 11% of the increase in the resilient modulus was obtained during the first month. In the field, FWD measurements showed that stiffness, expressed as the layer modulus, doubled or trebled during the first three months.

Recently, stiffness increase evaluated from field tests reported in Arm (2000a) and in Papers III and IV has been completed and analysed further (Figure 20). This analysis revealed a difference between the four test roads. In two of the roads, Road 109 and Stenstorp, the stiffness increase began immediately, whereas in the other two no stiffness increase was obtained during the first months.

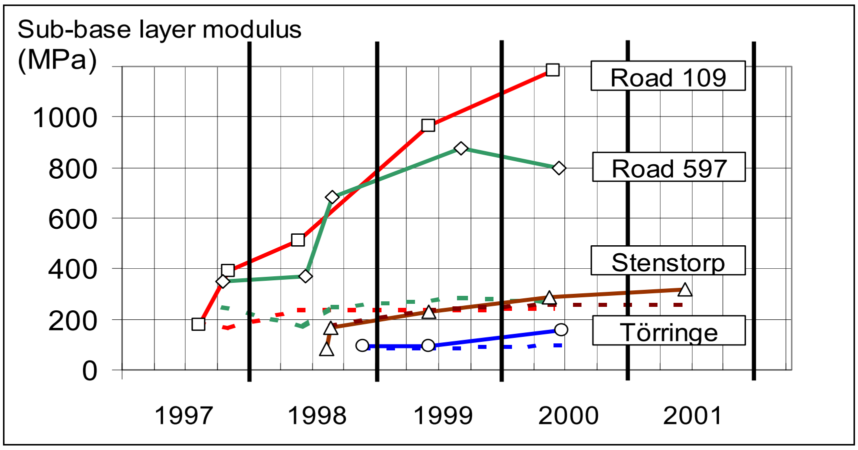


Figure 20. Results from FWD measurements on the test roads monitored. The dotted lines represent reference sections with natural aggregates on each test road (after Arm, 2000a, with additional unpublished data and with Stenstorp data from Carlsson, 2001).

Resistance to wear and disintegration

When tested using standardised laboratory methods, the crushed concrete materials had lower resistance in the form of a greater proportion of fines formed and greater disintegration than natural gravel and crushed rock (Arm, 2000a). Examples of indices under *mechanical impact* were a Ball mill value of 23–53% and a Los Angeles value of 33–39% depending on the origin. Compaction according to modified proctor produced significant crushing.

Examples of values in *freeze–thaw tests* with water were 8–17% disintegration depending on the origin, and in tests with saltwater, 15–59%.

8.2.2 Discussion

The properties of crushed concrete vary according to a number of parameters. Consequently, it is not possible to specify properties that apply to all types of crushed concrete, classified as one single material group.

Deformation properties

Both resilient and permanent deformation varied among the concrete materials. However, it was not clear whether it was the *origin* or the *handling*, e.g. crushing and cleaning, that affected the properties most. It was noticed that porous cement paste and rounded particles, as well as foreign materials and clean aggregate particles, without any remaining mortar, had a detrimental effect.

The term *origin* refers to the compressive strength and the type of aggregate and cement in the original concrete. In a 'demolition concrete' there are also foreign materials in the raw material, which are not found in a clean concrete and which influence the crushed product (Arm, 2000a). There was a difference between high- and low-strength concrete, which was explained by the fact that in high-strength concrete crushing yields broken particles, whereas in low-strength concrete the cement is the weakest part. This results in angular aggregate particles in the high-strength concrete and also in better stiffness and stability. However, if the aggregate particles are clean and the particles are rounded, the stiffness and stability are reduced. (Arm, 2000a)

Important parameters with regard to *handling* are the method of demolition, crushing and cleaning. The proportion of foreign material in the crushed product is of great significance to the properties. Weak particles, such as lightweight concrete and brick, but also wood, plaster and reinforcement, reduce the quality. For demolition concrete to be used for high-quality purposes, selective demolition must be carried out. Crushing has a significant influence on the properties, as is the case for natural aggregates since the resulting particle size distribution has an impact on stiffness, stability, frost susceptibility etc.

Rut depth measurements and static plate bearing tests have been performed on the test road in Törringe and will be reported later.

Stiffness increase

Stiffness increase in layers of crushed concrete has been observed by several researchers (Sweere & Henny, 1987; Yoshikane, 1988; Sweere, 1990; Kivekäs, 1997).

The conclusion drawn from the literature studies was that the mechanism underlying this stiffness increase is a leaching of calcium hydroxide formed during original hydration, followed by precipitation of calcium carbonate (Bruinsma et al., 1997). In the light of this knowledge, it should be possible to determine the appropriate handling and method of construction that will best utilise this property. It is expected that a low degree of carbonation in the original concrete produces more rapid carbonation and resultant increase in stiffness in the road. A lot of brick and natural material should have a limiting effect, since these materials do not themselves become carbonated. Concrete should be stored outdoors and in moist conditions to avoid early carbonation. However, when the material is placed in a construction, carbonation is desirable. This is obtained by thorough compaction and wetting, resulting in a long period of contact between water and the concrete material. A large particle surface area, i.e. dense grading with a lot of fines, would be favourable. This is in agreement with the Finnish specification for layers of crushed concrete (FINNRA, 2000), which recommends watering of the concrete for one month, or until the bitumen surfacing has been applied.

Within this project, the extent and process of strength development was evaluated in both laboratory and field tests. It was found that strength development was considerably less in the laboratory than in the field. One explanation may be that the laboratory samples were wrapped in plastic foil during storage and this prevented the desired carbonation. When the Swedish results were compared with Dutch laboratory and field results and Finnish field results, it was found that the results of field tests were largely in agreement in these countries. The Dutch field tests gave an increase in stiffness that was slightly quicker. On the other hand, the laboratory results were different inasmuch as the Swedish results showed a much lower increase in stiffness than the Dutch ones. Apart from the explanation above regarding the method of storage, the difference may also be explained by differences in test procedure and loading history, water ratio and size of the test specimen. (Arm, 2000a)

The further analysis of the FWD measurements (Figure 20) revealed an interesting difference between the four test roads. In two of the roads, Törringe and Road 597, no stiffness increase was obtained during the first months. Unlike the other two roads, these roads were constructed during late autumn/early winter and the concrete layers were laid in temperatures below freezing. It is therefore most probable that the chemical reactions causing the increase in stiffness were delayed until the springtime, with more advantageous temperatures. These findings were in conformity with Finnish experience. According to FINNRA (2000) the concrete layer must be spread out and compacted some weeks before the frost period begins to take advantage of the stiffness increase. After the frost period is finished the increase continues.

In both Finland and the Netherlands, the stiffness increase in layers of crushed concrete is taken into account in pavement design. In Finland, a higher design modulus is used for crushed concrete than for crushed rock if stiffness increase properties can be proved. This is proved through a compressive strength test of the same kind used for concrete. The values used, 700 MPa for 'residue concrete' and 500 MPa for 'demolition concrete', are described as conservative, the explanation being that this is a precautionary measure due to the short experience of this material (FINNRA, 2000). For crushed rock a modulus of 280 MPa is used.

In the Netherlands, the modulus used for unbound layers of crushed concrete is 600 MPa (CROW, 1999). For demolition rubble with 50% concrete and 50% other aggregates, a modulus value of 400 MPa is used. The modulus for 'ordinary' unbound aggregates, such as lava stone and crushed masonry, is 150 MPa (DWW, 1998).

Durability

Resistance to mechanical impact depends on particle shape and indirectly on the method of crushing, the type of crusher and the number of crushing stages. The flakier the particles, the lower the resistance. Resistance is also influenced by purity, so that a lot of brick and lightweight concrete have a detrimental effect on the recycled product. Original concrete of lower strength produces poorer abrasion resistance due to more porous cement paste (Ydrevik et al., 1996).

In view of the results of abrasion tests, crushing in several stages may give rise to unnecessarily large quantities of fines (Arm, 2000a). However, if the concrete material is pure, the fines formed will not be plastic but will instead participate in the strength development referred to previously.

Layers with crushed demolition debris with 45% concrete result in a stiffness decrease when exposed to repeated freeze-thaw periods with loading during thaw periods. Layers with 100% crushed concrete are not affected. (Kalisch, 1998).

In order to inspect the crushed concrete after two years in the road, material from the sub-base in Road 597 was dug up. An increase in particle size was noted, probably as a result of carbonation (Arm, 2000a). Yoshikane (1988) observed the same phenomenon.

The freeze-thaw tests in saltwater caused much more disintegration than tests in clean water. This sensitivity could be taken into consideration by using a denser surfacing layer on the road, as recommended in the Finnish specifications (FINNRA, 2000).

Swedish and foreign experience has demonstrated that long-term properties other than strength development have not yet been shown to differ from those of conventional materials. The Swedish objects being monitored are no more than three years old. (Arm, 2000a)

Environmental impact

Concrete consists of aggregates, cement and possible additives. The main difference in content between crushed concrete and crushed rock is therefore the cement, possible additives and possible impurities.

It is generally agreed that the amount of foreign materials in the concrete influences the environmental impact. The age, the field of application and the environment of the original construction affect the content of environmentally hazardous elements in the material. Chemically-contaminated concrete must naturally not be re-used.

At Road 109 and 597, leachate was collected from the test section and the reference section and was analysed in the laboratory. The results are reported in Arm (2000a) with references to Boverket (1999).

8.3 Results from tests on AcBFS

In cyclic load triaxial tests performed in this study, the unaged AcBFS had about the same resilient modulus – but less stress-dependent – as crushed granite of the same particle size (Paper IV). The stability of the slag, expressed in terms of accumulated permanent deformation, was initially considerably inferior to that of crushed granite but better than that of natural gravel (Arm, 2000a). However, after accelerated ageing in the laboratory both the resilient and the permanent deformation were decreased (Paper IV). An increase in strength with time was also observed in trial pits and it was measured by FWD (Paper IV).

The slag showed medium abrasion values, 22–26%, when tested in the Ball mill test. These values represented blast furnace slag of different origins and from different times (Paper IV, Arm 2000a).

Discussion

Blast furnace slag is a material that has for a long time been regarded, both in other European countries and the USA, as being largely the same as conventional natural materials.

Since the material is a residual product from the manufacture of iron, the raw materials, such as the iron ore and the added flux, are significant to the properties of the final product. In somewhat simple terms, it may be said that the better the ore, the lower the strength of the blast furnace slag (Höbeda, 1976). The manufacturing process, primarily cooling, is also very significant. The raw materials affect the chemical composition and the strength development. The cooling process during manufacturing affects particle size, porosity and strength development. The grading chosen influences the susceptibility to disintegration of the finished blast furnace slag layer.

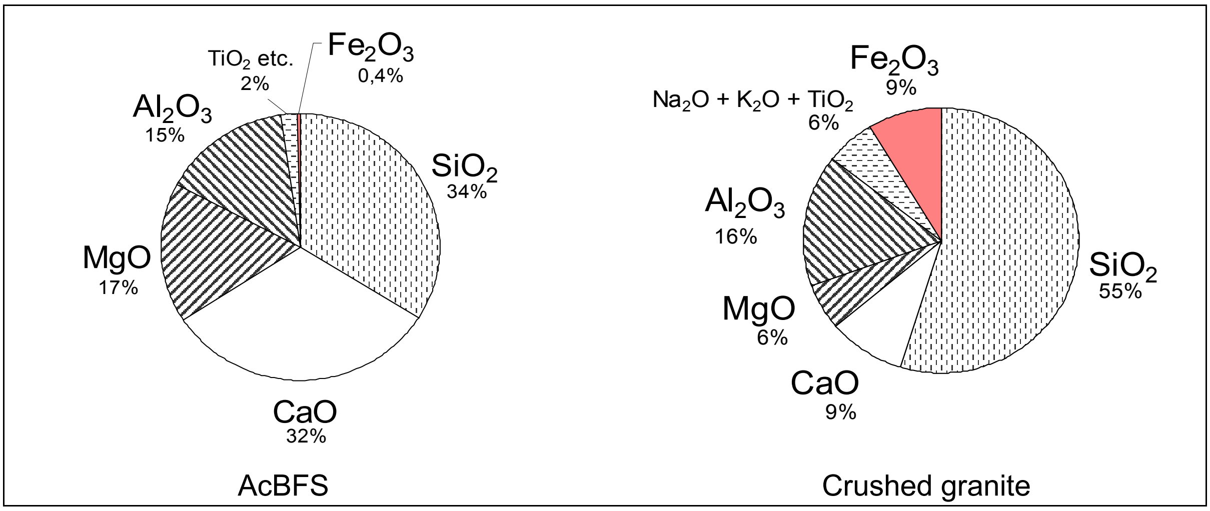


Figure 21. Chemical composition of AcBFS and crushed granite from Luleå (data from Lindgren, 1998).

Due to the blast furnace process the mineral composition of the slag is similar to other geological materials, such as basalt and diabase (Höbeda, op.cit.). The main differences are that the slag is more porous and the sulphur content in fresh slag is higher. In Figure 21 the composition of aged AcBFS from Luleå is compared with a conventional material, crushed granite from Luleå, which it replaced in a road construction. The main differences are the relative content of silicon and calcium.

AcBFS is a porous material, but its water absorption is nevertheless low since water remains on the surface instead of being absorbed into the material (Lindgren, 1998). This may be interpreted as implying that susceptibility to frost heave is low although further studies should be made.

The porous nature also leads to low thermal conductivity. λ-values of between 0.3 and 0.7 W/(m ·°C) depending on water content, were obtained for AcBFS from Luleå and Oxelösund. This can be compared with 0.6–1.8 W/(m ·°C) for sandy gravel and 0.5–1,1 W/(m ·°C) for crushed granite (SNRA, 2001). It is therefore surprising that no effect in the form of reduced frost penetration could be confirmed in frost depth measurements on a test road outside Luleå (Ihs, 1998). The explanation could be either a wet position for the slag or a dry position for the reference rock material. Further investigations are needed.

The low thermal conductivity could be used as frost insulation. The low value also implies that courses of AcBFS should not be placed too near the road surface owing to the risk of slippery conditions. According to Swedish regulations, the depth should not be less than 0.25 or 0.5 m depending on thermal conductivity.

Very little variation was found in the properties of blast furnace slag from the two plants in Sweden (Arm, 2000a). This is related to chemical composition, deformation on application of load, durability measured in Ball mill tests and thermal conductivity. The explanation may be that the manufacturing process is controlled.

8.4 Methodology for the assessment of deformation properties

A methodology is proposed that assesses the serviceability of an alternative unbound road material by comparing its deformation properties, evaluated from cyclic load triaxial tests, with the properties of the material normally used in the pavement layer in question (Paper V). There is also a description of how the test results can be used to design an alternative pavement in case the properties of a tested material are too different.

The deformation properties are determined in a series of cyclic load triaxial tests in the laboratory, during which the state of stress and the water content are varied. Testing conditions are chosen in order to simulate present or future field conditions. The results are expressed as resilient modulus as a function of mean normal stress and accumulated permanent axial strain as a function of number of loads. From the permanent deformation properties and a stated limit value for rutting, a permissible load on the material is estimated. This load or 'bearing capacity' tells what maximum load the material can bear if excessive permanent deformation is to be avoided in the unbound layer.

The permissible load could be expressed as permissible vertical stress for certain vertical/horizontal stress ratios in the surface level of the unbound layer. It could also be translated into a certain depth below the road surface, although the traffic load must then be standardised as well as the thickness and stiffness of overlying materials. In this procedure the bound layer is the most important since bound layers have considerable load-spreading capacity.

Discussion

The depth required depends on the traffic load, the stiffness and thickness of the bound layers and, of course, the bearing capacity of the material. Development of deformation due to degradation of the protecting bound layer can also be calculated. This equation should include the vertical stress in, and water ratio of, the unbound layer.

Of course, the specific deformation values obtained in the cyclic load triaxial tests are laboratory results that cannot be expected to appear in the field. However, they can be used for comparing and ranking materials. Furthermore, the methodology for estimating permissible load and suitable depth in the pavement must be developed further. For instance, validation in the field by means of instrumented test sections loaded by a Heavy Vehicle Simulator, HVS, is presently being performed. Small *in situ* methods, such as dynamic cone penetrometer, DCP, are also used. These are intended for checking whether the material meets the specified characteristics. The *in situ* methods are necessary since the construction work, such as laying and compaction, is a critical part that could impair the material properties determined in advance.

According to the cyclic load triaxial tests performed within several research projects at VTI, the permissible load for natural aggregate materials varies between 30 and 1,000 kPa if 5% deformation is allowed (Arm, 2000b). The variation depends mostly on the particle size distribution, but also on the relative water content. The natural aggregates included materials from single-sized sand to well-graded crushed rock material 0–64 mm. The same relative density was used.

This thesis has shown that the permissible load for crushed concrete, MSWI bottom ash and air-cooled blast furnace slag also varies but not to the same extent, due to a lower range in particle size distribution.

8.5 Test methods and material characterisation

Several test methods and the impact from European standardisation in this area have been studied in the project. It was then revealed that some of the test methods that are used, or suggested be used, for the testing of different properties of unbound material are more or less unsuitable for recycled aggregates and residues. There is thus a need for new test methods. Some new methods already exist but have not yet been standardised and some need to be developed, perhaps as a modification of existing methods.

8.5.1 Test methods for measuring the organic content of aggregates for unbound road layers

The minor investigation of organic matter content in MSWI bottom ash, crushed concrete and crushed granite from the test road in Törringe gave different results using different methods (Table 4).

Table 4. Content of organic matter measured using three different methods. (Arm, 2000a)

Method	Content of organic matter (weight percent)			Standard
	MSWI bottom ash (M –99)	**Crushed concrete**	**Crushed granite**	
Colorimetric measurement	1.7	<0.2	<0.2	SS 02 71 07
Ignition at 550ºC	4.1	1.76	0.74	SS 02 81 13
Ignition at 950ºC	6.3	6.3	1.1	SS 02 71 05

The three materials varied in sensitivity. The colorimetric measurement generally gave a lower content than the ignition methods. Furthermore, the content evaluated from ignition depended on the temperature used.

Discussion

The differences between the two method types can be attributed to the fact that some inorganic components can also be lost during ignition, such as carbon dioxide from carbonates. The risk of this is increased when the temperature is increased, which explains the difference in results from the two ignition methods.

The differences between results from different methods in this study (Table 4) agree with the findings of Larsson et al. (1985), who also wrote that ignition normally shows greater scatter than the colorimetric measurement. However, Larsson et al. (op. cit.) stated that the differences between colorimetric measurement and ignition are small as long as the material is lime-deficient. This was confirmed in the present study, where crushed concrete produced the largest relative differences (Table 4).

The most usual method for measuring the organic matter content is loss on ignition, LOI. However, according to Bäckman (1989) and also the SNRA standard (VVMB 34:1984), ignition is not suitable for material containing carbonates or certain clay minerals, for instance from areas with calcareous bedrock. Furthermore, Pavasars (2000) pointed out that bottom ash from waste incineration differs greatly from sewage sludge and sediment, which are what the LOI method was developed for. Several parameters can contribute to the LOI for a bottom ash, such as entrapped water and tightly-bound water of hydration released at 550˚C (Pavasars, op. cit.).

For an unbound road material, organic matter has a harmful effect on the stiffness. However, it is the amount of non-combusted material that is relevant, not the amount of water of hydration. Thus, the results from this project suggest that for road design purposes the LOI at 950˚C is not representative for the content of organic matter in MSWI bottom ash, or in crushed concrete. A more suitable method is the LOI at 550˚C or the colorimetric measurement. An alternative method is Total organic carbon, TOC, which is used for characterising waste. A comparison between LOI and TOC performed on inert waste showed that the TOC-value was 40–50% of the LOI 550ºC-value (Hjelmar et al., 1998, quoted in Fällman et al., 1999).

LOI at 975±25˚C is the present European standard method for measuring organic content in aggregates for unbound road materials (EN 1744-1). However, the resulting loss of weight at this high temperature does not reflect the organic matter content relevant for deformation properties. Consequently, there ought to be an amendment of the standard, considering that recycled aggregates and residues should also be covered.

8.5.2 Test methods for measuring deformation properties

In Paper V it is indicated that there are no European standard methods that measure the bearing capacity, providing input for the design process, especially not for residues such as MSWI bottom ash, crushed concrete and blast furnace slag. The indirect test methods, Sand equivalent test (EN 933-8) and Methylene blue test (EN 933-9), are intended for clayey materials and the CBR method cannot distinguish between elastic and plastic deformation. The relationships that are established between the CBR value and the E-modulus have therefore been criticised. The criticism is based on the fact that CBR tests result in a combination of elastic and permanent deformation while the E-modulus is a fully elastic parameter (Sweere, 1990).

In the case of the design of a road structure it is generally agreed that the triaxial method is suitable for the determination of stress-strain behaviour of unbound granular materials and soils. For a material with stress-dependent deformation properties, such as granular materials and cohesive materials, it is important to relate a given stiffness or M_r to a certain stress condition. Further advantages of the method are described in Paper V. Recently, a draft for a European standard for the cyclic load triaxial test for unbound materials was presented. This has been prepared by representatives from several European laboratories with experience from cyclic load triaxial tests and will be the subject of a formal vote within the CEN (Paper V).

Neither of the two field methods the static plate bearing test and the FWD measurement can be used on weak subsoil, such as a soft, fine-grained material. The static plate bearing test can measure the results of compaction but cannot locate weak layers in the road pavement at a later date. Furthermore, it produces a static load that is different from the impact of heavy vehicles and it is time-consuming. The falling weight deflectometer produces a dynamic load that simulates the axle-load of a lorry. It is quick and can distinguish between the properties of different material layers. The different makes of mini falling weight deflectometers that have been developed recently have the advantages that they can be carried and that they can produce a small dynamic load that is relevant for the stress levels in the lower part of a pavement. (Arm, 2000a)

8.5.3 Test methods for measuring durability

According to Papers III and IV, both laboratory and field investigations showed smaller deformations in the AcBFS and crushed concrete layers than in the crushed rock layers. It also revealed an increase in stiffness, which did not occur in the crushed rock. However, both residues had poorer results in the durability tests, such as the Los Angeles (LA) test and the Ball mill test, than aggregates of a typical Swedish rock (granite). This justified further study of the present test methods for determining aggregate durability.

There are many methods for measuring durability in terms of resistance to mechanical action. What they all have in common is that a sample of single-sized aggregate is investigated. As described in Section 4.3, the methods produce different kinds of impact on the material tested. This leads to a variation in the influence on the result, depending on the material in question. Test results from crushing methods, such as the LA test, depend mostly on the porosity of the material. Results from Ball mill tests depend on both the porosity and the alteration, even if the result says more about the alteration properties than the resistance to degradation (Bjarnason et al., 1999). Figure 22 illustrates the difference between the methods.

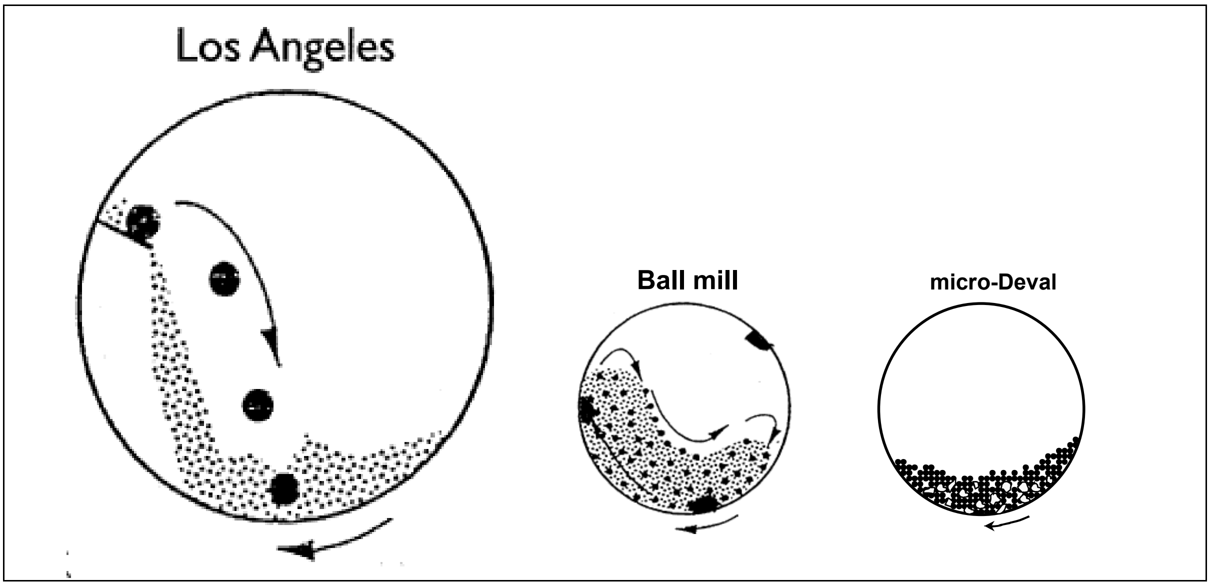


Figure 22. Test principle for the Los Angeles test, the Ball mill test and the micro-Deval test (after Arm, 2000a).

The balls used in the LA test are 3–5 times bigger than the particles tested, whereas the Ball mill uses balls that are of about the same size as the aggregates. In the LA drum there is a steel shelf, which causes the particles to fall freely, whereas the Ball mill causes a stirring movement. In the LA drum the treatment is dry, whilst in the Ball mill it is wet. This causes the particles to be crushed in the LA drum. This is indicated by both the particle size and the particle shape (Stenlid, 1996). The LA test therefore shows a negative effect of brittle minerals, such as feldspar, and fissured material, whereas in the Ball mill test the feldspar and quartz contribute to a good resistance to wear which results in opposite test results. However, for the mica-rich and micaceous sedimentary rock types the LA and Ball mill tests probably register similar properties.

In the two American standards for the LA test (ASTM C131 and C535) a warning is included that the LA value should not be used for purposes other than to show the relative qualities of aggregates from sources with similar mineral composition. This should be borne in mind before the LA test is introduced as a standard method for all aggregate materials, including recycled aggregates and residues.

In the ALT-MAT project a comparison between a number of durability tests was made. The aim was to compare how the resistance to wear of common alternative aggregate materials was assessed using the different methods. In particular, it was believed that methods like the LA test should cause too much degradation of soft alternative materials.

The methods compared were micro-Deval test, LA test according to EN, LA test according to ASTM, vibrating table and gyratoric compaction. The materials investigated were crushed concrete, processed MSWI bottom ash from different plants and some natural reference materials. The following conclusions were drawn:

- The micro-Deval test and the two LA tests ranked the materials similarly.
- Methods including sieving of the material before and after could give misleading results for weak materials due to disintegration during sieving.
- The prescribed number of rotations in micro-Deval and LA tests could be questioned for alternative materials. The reason is that for conventional aggregates there is a linear relationship between the disintegration and the number of rotations, whereas the alternative materials are worn more in the beginning than in the end.
- The different versions of LA tests did not produce significantly different test results.
- The micro-Deval test separated the different materials the best.
- Gyratoric compaction showed good results and further development was recommended.
- The vibrating table is not suitable for durability testing. (TRL, 2000)

It could be concluded that the methods that have been developed for testing resistance of conventional materials are often unsuitable for alternative materials, which produce poor results owing to weak, porous particles. However, even though recycled products often generate a lot of fine materials, the fines are not plastic as is the case with natural materials of poor resistance (Höbeda & Chytla, 1999). On the contrary, it is these fines that produce the binding effect that leads to an increase in stiffness. This should be considered when limit values are set. As an example, some American states already take this into consideration for blast furnace slag, meaning that they have simply rejected the LA test for the slag (Chesner et al., 1998).

It could be concluded that there is currently no standardised method that characterises resistance to mechanical action in materials for unbound layers. The existing methods investigate a sample of single-sized aggregate. The reason is probably that the tests were originally developed for aggregates in bituminous mixes that have ‘short fractions’. In addition, the way of testing with rotating drums is influenced by the asphalt industry. The resistance or particle strength of a single-sized aggregate depends on the individual particle – on its geometrical shape, mineral composition and hardness and its structure and texture. However, it could be questioned how well the traditional way of testing reflects the resistance of a layer with unbound material. In a layer, the resistance also depends on how particles are in contact, for example how densely it is compacted, which is in turn a function of particle size distribution etc.

All the standard methods mentioned above were developed originally for aggregates in bound layers. In recent times a search has started for new methods that reflect better the resistance of unbound materials to mechanical action. One alternative to the traditional methods is *gyratory compaction*, which has been tested within the European project ALT-MAT and in Norwegian and Finnish research projects (TRL, 2000; Hoff & Natvik, 2000; Tielaitos geokeskus, 1994 and Keto, 1995 cited in Natvik, 1998; Laaksonen et al., 1999). Another alternative is the *Icelandic Bg index* (Bjarnason et al., 1999). Höbeda & Chytla (1999) mention the *wet abrasion test in a rotary attrition apparatus* as an interesting method for some residues, such as crushed concrete and metallurgical slag. These methods are interesting since a large portion, or even the whole, of the unbound material is used in the test. Further development is needed to find methods that do not involve sieve analysis, since this can in itself cause degradation of weak materials.

In *freeze–thaw* tests and tests with magnesium sulphate, degradation in many residues, owing to their porous nature, is greater than in most Swedish natural materials. The same difference has not yet been demonstrated in the field. The aim in the European harmonisation process is to obtain more information on the applicability of freeze–thaw tests for alternative aggregate materials.

8.5.4 Material characterisation

As already shown, the Swedish specifications for unbound road materials are of the recipe type, describing what a material should consist of rather than what mechanical properties it should have. This also holds for many other countries. The reason is that most specifications are empirically based. To simplify the introduction of new materials, so-called end-product specifications, prescribing engineering properties such as stiffness and stability, should be introduced.

This requires, on the one hand, laboratory methods that could be used in advance to characterise the material and to choose the most appropriate use of it. On the other hand, it requires rapid and simple methods for measuring *in situ* properties, thus checking if the material meets the specified characteristics.

The direct material specifications for an unbound material could be related to the following properties and test methods (Arm, 2000a).

- Permissible load according to load tests,
- resistance to mechanical action according to gyratoric compaction,
- resistance to freeze-thaw actions according to a modified freeze-thaw test,
- frost susceptibility according to a freezing test,
- thermal conductivity according to the sond method and
- potential environmental pollution through determination of total contents of main and trace elements, determination of their leachability according to availability tests, the two-stage batch test and the pH static test.

Load tests should be performed as cyclic load triaxial tests. There are many reasons for this. It is a laboratory method that exposes the material to loads that produce conditions similar to traffic. The whole composite material, up to a certain particle size, is tested instead of separate particles being exposed to stresses. Consequently, it could be said that the performance of the whole specimen is tested. Cyclic load triaxial testing also has the advantage that the stress-dependency of the deformations could be analysed. This knowledge can be used when a suitable position in the pavement is to be given. Results from triaxial testing of well-known approved materials can be used as a reference when alternative materials are introduced and when their equivalence to the material they replace should be proved.

Gyratoric compaction is in fact a method for producing specimens from bitumen-bound material. However, it was used in the ALT-MAT project and was adjudged to have potential for development for future use for unbound materials.

Modified freeze-thaw tests means that the test is not just carried out on saturated material.

The differences compared with the present characterisation in Sweden are:

Instead of bearing capacity, several indirect methods are used, such as particle size distribution, amount of crushed material and content of organic matter. Instead of gyratoric compaction, Ball mill or micro-Deval is used, which test single particles rather than the whole material. Freeze-thaw tests are not prescribed today and freezing tests are seldom carried out. No general method is currently prescribed to determine thermal conductivity. Environmental impact tests are not generally performed today.

It is also important to point out that at the same time that new test methods are developed, new requirements or limits also need to be stipulated. It is then important that the level of the requirements that are stipulated is related closely to use, such as the greater the stress, the higher the level of resistance to deformation and to wear and disintegration.

9 CONCLUSIONS

The following conclusions can be drawn from this project as regards *material properties*.

- As expected, the resilient deformation properties of MSWI bottom ash are greatly impaired by a high proportion of non-combusted material.
- Certain MSWI bottom ash could be used, not only in embankments and capping layers but also to bear the stress levels expected in a sub-base.
- With a thorough incineration process, together with careful and standardised sorting and sieving, the variation in stiffness and stability of MSWI bottom ash could be kept low.
- In order to increase the quality of MSWI bottom ash, the incineration process should be tuned. After several years of concentration on flue gas cleaning, it is time to devote some effort to the quality of the bottom ash.
- If crushed demolition concrete is to be capable of being used for high-quality applications, far-reaching selective demolition of buildings and constructions is necessary. This is already taking place to a large extent.
- Crushed concrete and AcBFS have evident strength development properties. Knowledge of the underlying mechanism should be used in optimising strength development.
- The road application properties (in unbound layers) of AcBFS and crushed concrete with few impurities are as good as or even better than those of crushed granite.

The following conclusions hold for *test methods*.

- The LA test and other tests developed for single-sized aggregates do not really justify the performance of the materials studied.
- Method development is needed with regard to tests for resistance, both to mechanical and climatic action.
- LOI at 975±25°C is unsuitable for the determination of organic content in MSWI bottom ash intended for use in unbound road layers.
- Cyclic load triaxial tests are suitable for assessing the deformation properties of alternative aggregate materials, e.g. when their equality to the replaced material is to be proved.
- It is anticipated that a wider use of cyclic load triaxial tests for characterising unbound road materials, both conventional and alternative, will result in suitability for purpose use and thus more efficient use of natural materials and more sustainable resource management.
- Environmental impact limit values, in the form of maximum permissible leaching under certain conditions, must be introduced as soon as possible for all aggregate materials.

The following conclusions can also be drawn from this project with regard to *specifications and design*.

- So-called end-product specifications prescribing engineering properties, such as stiffness and stability, should be introduced. This requires, on the one hand, laboratory methods that could be used to characterise a material and to choose the most appropriate use of it. On the other hand, it requires fast and simple methods for measuring *in situ* properties, thus checking whether the material meets the specified characteristics.
- The quality requirements for materials could be more differentiated and thus allow a wider range of materials to be used than is the case at present. As an example, more stringent requirements are needed for materials placed directly under a thin asphalt surfacing than under a thick surfacing.
- The assessment technique suggested in this thesis has clearly shown the impact of particle size distribution, water content, compaction and particle shape on the deformation properties. It can be introduced as a part of performance-related material specifications.
- The risk of overloading a weak, unbound material is greatest in roads with thin bound surface layers. This should be borne in mind when it is recommended that residues should be used in roads with low traffic volumes.
- In order for both ‘good’ and ‘poor’ aggregate materials to be used correctly, a new design method must be introduced as soon as possible. A proposal based on the permissible load/bearing capacity classification is presented in this report.

10 PROPOSALS FOR GUIDELINES/SPECIFICATIONS

It could be concluded from this project that: Recycled aggregates and industrial residues should be used near the source of production, since it is then that they are price-competitive. The environmental gains also vanish if the haulage distance is too great. In practice, this implies that their use will be greatest in urban areas and the immediate surroundings.

The limitations of the residues must be taken into consideration. There is otherwise a risk that the material will be blamed when the road deteriorates, although in actual fact it is a faulty design that is the cause. This will result in the material acquiring a bad reputation and nobody will want to use it. None of the three materials studied should therefore be laid as a base course below a thin surfacing layer, and construction traffic on MSWI bottom ash should be avoided.

The properties of the residues should be used to the greatest possible extent. This implies that the materials should be laid in road pavements at the depth that their bearing capacity permits. Use should be made of the special frost-insulating properties of blast furnace slag and MSWI bottom ash, and the strength development in crushed concrete and blast furnace slag.

In summary, if performance-based test methods, a proper design and common sense are applied in using alternative aggregate materials, natural resources, money and space can be saved and environmental impact can be reduced.

Use of processed MSWI bottom ash

According to present Swedish regulations, processed MSWI bottom ash cannot be used unless the organic matter content is less than 2%, as measured by the colorimetric method (NB not by LOI!) Nor can bottom ash from the incineration of hazardous waste be used.

Owing to its road engineering properties, bottom ash can be laid 25–35 cm below the road surface, depending on the type and thickness of the surface layers. The same stiffness as for natural gravel may be assumed. This means that MSWI bottom ash can be used not only as embankment fill and capping layer but also in the sub-base, depending on the handling of the material and the thickness of the surfacing layer and the base course. Construction traffic on the compacted layer should be avoided. The bottom ash should be stored outdoors before use. The longer the storage time the better, but at least six months. Owing to its environmental properties, special measures are required.

Use of AcBFS and crushed concrete

AcBFS and clean crushed concrete could be used as embankment fill, as a capping layer and as a sub-base. Their properties are best used in a sub-base. The materials can also be used in base courses, depending on the thickness of the bound surfacing layer. The slag should not be used close to the road surface according to the present restrictions on thermal conductivity. Crushed concrete should not be used in pavements with thin, bound courses, where large traffic load stresses may be expected in the base course.

When AcBFS and crushed concrete is used as sub-base material, the same resilient modulus values can be used in the design as for crushed rock. If special investigations are made of the material concerned, and if details of the planned construction are known, a higher modulus value may be applied. In this way use can be made of the increase in strength exhibited by the material.

The thermal conductivity should be checked in order to use the insulating properties of the slag properly.

If unaged AcBFS is laid on poorly-drained subsoil, problems due to the leaching of sulphur may arise.

In Table 5 some characteristics of the materials studied in this project are summarised.

Table 5. Characteristics of the materials studied

Material	Application options	Substitute for	Availability	Density	M_r (MPa)			Per-missible load if 2% perm. def. accepted (kPa)	Durability degradation (%)				Comments
			Approx. annual production (tonnes)	Max. mod. pr. dens. (t/m^3) *)	σ_v: 90 σ_h: 20	200 60	560 kPa 120 kPa		m-D	Ball mill	LA	Freeze -thaw	
MSWI bottom ash	Embankment, fill, capping layer, (sub-base)	Sand, **natural gravel**, (crushed rock)	400,000	1.5–1.8	70–110	210–230	260–300	150–200	26-39	NT	45	1-8	High internal friction due to angular particles
Crushed concrete	Embankment, fill, capping layer, **sub-base**, (base)	Sand, natural gravel, **crushed rock**	1 million	1.8–2.0	NT	220–450	320–470	200–400	NT	23-53	33-39	8-17 15-59	Stiffness increase if low content of foreign particles: +21% after 2 months
AcBFS	Embankment, fill, capping layer, **sub-base**, (base)	Sand, natural gravel, **crushed rock**	400,000		NT	300 Aged at 50°C for one month: 450	420 Aged at 50°C for one month: 600		NT	22-26	35	NT	Stiffness increase. Low thermal conductivity in dry conditions. λ=0.3-0.7 depending on the water content

*) depends on the particle size distribution.
NT means not tested.
NB: The table only includes test results from this project.

A proposal for future material characterisation

For all materials, the following properties should be declared:

- Bearing capacity is determined at different stress states and water contents by means of cyclic load triaxial tests. The result can be used for design purposes (how close to the road surface can the material be laid). Any strength development should also be elucidated here.
- Resistance to mechanical action is determined using modified gyratory compaction. This determines whether compaction and construction traffic must be adjusted. Design must ensure that no crushing can occur in the finished construction.
- Resistance to climatic action is ascertained by a modified freeze–thaw test. This determines whether there are any limitations regarding use of the material, both geographically and in the construction. Close to the road surface there could be problems with de-icing salt for instance.
- Susceptibility to frost heave and also its load-dependence should be determined in the laboratory by freezing tests with different overloads.
- Thermal conductivity is determined when a divergent value could be assumed – in the case of porous materials for instance.
- Environmental impact should be ascertained through a series of tests when the chemical composition and leachable quantity of a range of elements is determined as well as the dependence of leaching on water content, pH and redox potential. The organic matter content should also be determined.

Note that the testing described above should act as a first characterisation of a new material. It should take the form of a fundamental investigation that leads to a 'type approval' or product certification for both conventional and alternative materials. Production control or quality control can then be carried out using other, quicker methods, where only certain critical parameters are investigated and compared with the fundamental characterisation in order to ensure that the material is the same as the material certified. It should be noted that all materials should be covered by the requirement regarding investigation of environmental impact.

11 PROPOSALS FOR FUTURE RESEARCH

In addition to what is already taking place according to Section 1.5 the following issues are urgent:

Use of the special properties of residues.

In this and in other research projects it has been shown that alternative aggregate materials, such as MSWI bottom ash, crushed concrete and air-cooled blast furnace slag, could replace conventional aggregate in several applications. This leads to cheaper material, shorter transport distances or decreased costs for disposal. It also leads to decreased extraction from gravel pits and rock quarries, decreased need for landfills and less air pollution from transport.

However, there is more to be gained if advantage could be taken of the special properties of certain residues. One example is the low thermal conductivity that can be used for insulation purposes, thus giving rise to thinner road pavements. Another example is the increase in strength over time, which can be made use of where stiff constructions are needed.

More detailed research is needed to be able to predict how much of the property could be used. Some examples are

- laboratory and field tests, where MSWI bottom ash and AcBFS are used as an insulation layer,
- test sections, where layers of crushed rock are replaced by thinner layers of crushed concrete and
- an accelerated method for predicting strength development. Existing methods could be developed further.

Improvement of the properties of MSWI bottom ash.

Fundamental practical studies of the waste incineration process could identify measures that improve the properties of bottom ash. This increases the usability of MSWI bottom ash and thus reduces the load on landfill sites.

A new test method for resistance to mechanical action.

A method is needed that simulates actual conditions in unbound layers and tests the entire material. Perhaps one of the existing methods can be modified. A method that does not involve sieving should be developed.

Investigation of the resistance of different residues to chemical action.

Development of limit values for the environmental impact of aggregate materials.

This is a very important area, which is not dealt with in this project. However, it may be said with some exaggeration that there is no shortage of proposals, only decisions. The proposals made have been put in the pending tray by some authorities. The limit values that will in time be introduced should be based on an assessment of the risk due to the expected impact, with account being taken of the vulnerability of the site concerned.

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REFERENCES

Arell, L., (1997). *Sekundära ballastmaterial.* Licentiate thesis, TRITA-AMI LIC 2022. Kungliga Tekniska Högskolan, Stockholm. (In Swedish with an English summary)

Arm, M., (1996), (1998), (2000b) *Obundna vägmaterials mekaniska egenskaper. Bärförmåga hos obundna vägmaterial bestämd med hjälp av dynamiska treaxialförsök.* Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Arm, M., (2000a) *Egenskaper hos alternativa ballastmaterial – speciellt slaggrus, krossad betong och hyttsten.* Licentiate thesis. TRITA-AMI LIC 2063. Kungliga Tekniska Högskolan, Stockholm. (In Swedish with an extensive summary in English)

Arm, M., Svensson, J. & Ydrevik, K., (1995) *Deformationsegenskaper hos finkornig jord.* VTI meddelande 770. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Barbery, O. & Ghodsi, A., (2000) *Trapping of chloride, sulfate and heavy metals.* In: Dhir, Dyer & Paine (edited) Sustainable Construction: Use of Incinerator Ash, International Symposium in Dundee 2000. Thomas Telford Ltd, London. pp 173–183.

Belevi, H., (1998) *Environmental engineering of municipal solid waste incineration.* Hochschulverlag AG an der Eidgenössische Technische Hochschule ETH, Zürich.

Bill 1992/93:180. *Riktlinjer för en kretsloppsanpassad samhällsutveckling.* Fritzes offentliga publikationer, Stockholm. (In Swedish)

Bill 1997/98:145. *Svenska miljömål – miljöpolitik för ett hållbart Sverige.* Fritzes offentliga publikationer, Stockholm. (English short version *Swedish Environmental Objectives* available at http://miljomal.nu/english)

Bill 2000/01:130. *Svenska miljömål – delmål och åtgärdsstrategier.* Fritzes offentliga publikationer, Stockholm. (English short version *Swedish Environmental Objectives – Interim Targets and Action Strategies* available from the Swedish Environmental Objectives Council)

Bjarnason, G., Erlingsson, S. & Petursson, P., (1999) *COURAGE – Aggregate resistance to fragmentation, weathering and abrasion – comparison of different test methods.* BUSL report no E-37. Icelandic Building Research Institute, Reykjavik.

Boverket, (1998) *Gruvavfall från norr blir ballastmaterial i söder; Ekonomi och transport sjövägen.* Boverket, Karlskrona. (In Swedish)

Boverket, (1999) *Betong i vägar – materialstudie. Om möjligheterna att återvinna betong från husrivning.* Boverket, Karlskrona. (In Swedish)

Bruinsma, J.E., Peterson, K.R. & Snyder, M.B., (1997) *Chemical approach to formation of calcite precipitate from recycled concrete aggregate base layers.* Transportation Research Record no 1577. Transportation Research Board, National Research Council, Washington, D.C.

Bäckman, L., (1989) *Bestämning av organisk halt i grovkorniga material (bärlager).* VTI notat V84. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Carlsson, H., (2001) *Förstärkningslagermaterial av krossad betongslipers.* Lägesrapport 2001, VTI notat 76-2001, Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Chandler, A.J., Eighmy, T.T., Hartlén, J., Helmar, O., Kosson, D.S., Sawell, S.E., van der Sloot, H.A. & Vehlow, J., (1997) *Municipal Solid Waste Incinerator Residues.* Studies in Environmental Science 67. The International Ash Working Group (IAWG), Elsevier.

Chesner, W.H., Collins, R.J. & MacKay, M.H., (1998). *User Guidelines for Waste and By-Product Materials in Pavement Construction.* Publ FHWA-RD-97-148. US Department of Transportation, Federal Highway Administration, Georgetown Pike, Virginia.

COST, (1997) *New Road Monitoring Equipment and Methods.* Transport Research. COST 325.

CROW, (1999) *Secundaire bouwstoffen voor de wegenbouw*. Publicatie 143. CROW, Ede, The Netherlands. (In Dutch)

DWW, (1998) *Handleiding Wegenbouw, Ontwerp Verhardingen*. Dienst Weg- en Waterbouwkunde, Rijkswaterstaat, Delft. (In Dutch)

FAS, (1995) *FAS Asfaltbok*. Föreningen för asfaltbeläggningar i Sverige, Stockholm. (In Swedish)

Fernandez, M.A., Martinez, L., Segarra, M., Garci, J.C. & Espiell, F., (1992) *Behavior of heavy metals in the combustion gases of urban waste incinerators*. Environmental Science & Technology 26, pp 1040–1047.

FINNRA, (2000) *Use of Reclaimed Concrete in Pavement Structures, Design Manual and Construction Specifications*. Lohja Envirotec, Viatek, VTT and others. Finnish Road Administration, Helsinki.

Fuller, W.B., (1905) *Proportioning concrete*. In: Taylor & Thompson (edited) A treatise on concrete, plain and reinforced. John Wiley, New York. pp 183–215.

Fällman, A-M., (2000) *Significance in the results of total composition and potential leachability of screened MSWI BA from different plants and sampling periods*. In: Woolley, Goumans & Wainwright (edited) Waste Materials in Construction. Elsevier Science Ltd. pp 355–370.

Fällman, A-M., Larsson, L. & Rogbeck, J., (1999) *Slaggrus Miljömässiga och materialtekniska egenskaper*. Svenska Renhållningsverksföreningen RVF, Malmö och Statens geotekniska institut, Linköping. (In Swedish)

Galjaard, P.J. & Cools, P., (1995) *Analysis of in-situ measurements on a full-scale test pavement*. In: Dawson & Jones (edited) Unbound Aggregates in Roads, Proceedings of the UNBAR4. Department of Civil Engineering, University of Nottingham. pp 281–291.

GRAAB, (1996) *Avfallskraftvärmeverket i Sävenäs, Göteborg*. Broschyr. Göteborgsregionens Avfalls AB, Göteborg. (In Swedish)

Henneveld, R., (1995) *In-situ measurements of the bearing capacity of granular base courses*. In: Dawson & Jones (edited) Unbound Aggregates in Roads, Proceedings of the UNBAR4. Department of Civil Engineering, University of Nottingham. pp 293–304.

Hjelmar, O., Holm, P.E., Asmusen, O. & Isaksen, J., (1998) *Karakterisering af avfald*. Miljöprojekt no 414, 1998. Miljöstyrelsen, Köpenhamn. (In Danish)

Hoff, I. & Natvik, J., (2000) *Undersøkelser i laboratoriet. KPG-Prosjektet*. Våre Veger no 2B February 2000, Oslo. (In Norwegian)

Höbeda, P., (1976) *Masugnsslagg som vägmaterial, en litteraturstudie*. Rapport nr 111. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Höbeda, P., (1988) *Nötningsegenskaper. Aktuell metodutveckling – kulkvarnen*. Föredrag vid vägmaterialdagarna i Linköping 1987. I: VTI meddelande nr 559. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Höbeda, P. & Chytla, J., (1985) *Nötning av beläggningssten i kulkvarn*. VTI notat nr 444, Statens väg- och trafikinstitut, Linköping. (In Swedish)

Höbeda, P. & Chytla, J., (1999) *Undersökning av "finmaterialkvalitet" hos vägmaterial i synnerhet hos obundna material*. VTI notat nr 60-1999. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Höbeda, P., Jacobson, T. & Viman, L., (1985) *Undersökning av förbränningsrest från SYSAV, Malmö, genom laboratorie- och provvägsförsök*. VTI meddelande nr 413. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Ihs, A., (1998) *Halkbenägenhet hos olika överbyggnadstyper*. VTI notat nr 63-1998. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Jacobsson, T. & Viman, L., (1989) *Provvägsförsök Gärstad –87, Slagger från kol- och sopförbränning*, Lägesrapport 8901. VTI notat nr V80. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Johansson, H., (1997) *Återanvändning och återvinning av restprodukter, vägbyggnadsmaterial och sekundära, mineraliska material i vägar och gator*. KFB & VTI forskning/research 21 1997. Väg- och transportforskningsinstitutet, Linköping. (In Swedish with an English summary)

Johansson, H., Ydrevik, K. & Arvidsson, H., (1996) *Krossad betong – ett material för användning i vägar och gator*. VTI notat nr 1-1996. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Kalisch, H., (1998) *Untersuchungen an Asphaltbauweisen mit wiederverwendeten Baustoffen in der ungebundenen Tragschicht des Oberbaus*. Report from projekt no 94 340. Bundesanstalt für Strassenwesen BASt, Bergisch Gladbach. (In German)

Keto, E., (1995) *Estimating the compactibility of the crushed rock aggregates based on IC-testing*. Master thesis, Tampere University of Technology.

Kivekäs, L., (1997) *The use of concrete and brick demolition wastes in earth constructions*. In the Journal "Rakennustekniikka" no 4 1997, Finland.

Kolisoja, P., (1997) *Resilient Deformation Characteristics of Granular Materials*. Dissertation. Publication 223. Tampere University of Technology.

Laaksonen, R., Nordenswan, E., Paakinen, I., Alkio, R. & Törnqvist, J., (1999) *ICT-moduuli*. Tutkimusraportti 365/1999. Technical Research Centre of Finland, Espoo. (In Finnish)

Larsson, R., Nilsson, G. & Rogbeck, J., (1985) *Bestämning av organisk halt, karbonathalt och sulfidhalt i jord*. Rapport No 27. Statens geotekniska institut, Linköping. (In Swedish)

Lindgren, Å., (1992) *Laboratorieundersökningar av masugnsslagg*. Teknisk rapport 1992:09T. Luleå tekniska universitet. (In Swedish)

Lindgren, Å., (1998) *Road Construction Materials as a Source of Pollutants*. Doctoral Thesis. Division of Traffic Engineering, Department of Environmental Engineering, Luleå University of Technology.

Lundgren, T. & Hartlén, J., (1991) *Slagg från avfallsförbränning, Teknik och Miljö*. REFORSK rapport FoU nr 61. REFORSK, Malmö. (In Swedish with an English summary)

Miljø- og Energiministeriet, (2000) *Bekendtgørelse om genanvendelse af restprodukter og jord til bygge- og anlægsarbejder*. BEK nr 655 af 27/06/2000. Miljø- og Energiministeriet, Köpenhamn. (In Danish)

National Stone Association, (1991) *The Aggregate Handbook*. Barksdale (edited). National Stone Association, Washington D.C.

Natvik, J., (1998) *Alternative testmetoder i undersøkelser av tilslagsmaterialer*. Doktor ingeniøravhandling 1998:27. Institutt for geologi og bergteknikk. Norges teknisk-naturvitenskapelige universitet NTNU, Trondheim. (In Norwegian)

NE, (1994) *Nationalencyklopedin*. Bra Böcker, Höganäs, Sweden. (In Swedish)

Nilsson, U. & Larsson, L., (2002) *Miljömässig karakterisering av sorterad bottenaska från Dåva kraftvärmeverk*. Slutrapport. SGI proj 10271. (In Swedish)

Nunes, M.C.M., (1997) *Enabling the use of alternative materials in road construction*. Doctoral thesis, Department of Civil Engineering, University of Nottingham.

OECD, (1997) *Recycling strategies for road works*. Organisation for Economic Co-operation and Development, Paris.

Pavasars, I., (2000) *Composition of organic matter in bottom ash from MSWI*. In: Woolley, Goumans & Wainwright (edited) Waste materials in construction, Science and engineering of recycling for environmental protection, Proceedings of the WASCON 2000. Elsevier Science Ltd. pp 241–246.

Pfrang-Stotz, G. & Reichelt, J., (2000) *Municipal solid waste incineration (MSWI) bottom ash: characteristics and the use in road construction*. In: Dhir, Dyer & Paine (edited) Sustainable Construction: Use of Incinerator Ash, International Symposium in Dundee 2000. Thomas Telford Ltd, London. pp 235–246.

Pihl, K. & Milvang-Jensen, O., (1996) *Bundsikringslag af forbraendingsslagge*. Vejteknisk Institut Rapport 78 1996. Vejdirektoratet, Roskilde. (In Danish)

Rogers, C.D.F., Brown, A.J. & Fleming, P.R., (1995) *Elastic stiffness measurement of pavement foundation layers*. In: Dawson & Jones (edited) Unbound Aggregates in Roads, UNBAR4 Symposium 1995. University of Nottingham. pp 331–338.

RVF, (1997), (2002) *Swedish Waste Management 1997 and 2002* respectively. The Swedish Association of Waste Management. Malmö.

SFS 1999:673. *Lag om skatt på avfall*. Ändrad genom SFS 1999:1321 och 2002:998. Fritzes, Stockholm. (In Swedish)

SFS 2001:512 and SFS 2001:1063. *Förordning om deponering av avfall samt Avfallsförordning*. Fritzes, Stockholm. (In Swedish)

SGU, (2002) *Aggregates. Production and resources 2001*. Per. publ. 2002:3. Geological Survey of Sweden, Uppsala.

SNRA, (1994) *Road 94 General technical construction specifications for roads*. Chap 1–5. Publ 1994:21–23, 1994:86, 1994:25. Swedish National Road Administration, Borlänge.

SNRA, (1996) *Kretsloppsanpassad väghållning. Handlingsplan*. Publ 1996:29. Vägverket, Borlänge. (In Swedish)

SNRA, (2000) *Inriktningsprogram för mark- och vattenfrågor i Vägverket*. Publ 2000:85. Vägverket, Borlänge. (In Swedish)

SNRA, (2001) *Provningsmetoder för alternativa material till vägunderbyggnad*. Publ 2001:34. Vägverket, Borlänge. (In Swedish)

SNRA, (2003a) *ATB VÄG. Allmän teknisk beskrivning för vägkonstruktioner*. Vägverket, Borlänge. (In Swedish)

SNRA, (2003b) *Krossad betong i vägkonstruktioner*. Remissutgåva 2003-02-14. Vägverket, Borlänge. (In Swedish)

Stenlid, L., (1996) *Klassificering av bergarter med Los Angeles-trumma*. Slutrapport SBUF projekt nr 2135. Sveriges byggentreprenörers utvecklingsfond, Stockholm. (In Swedish)

Sweere, G.T.H., (1990) *Unbound granular bases for roads*. Dissertation. Delft University of Technology.

Sweere, G.T.H & Henny, R.J., (1987) *Toename van de elastische stijfheid van betongranulaat met de tijd*. Technische Universiteit Delft, Faculteit der Civiele Techniek, Laboratorium voor Weg- en Spoorwegbouwkunde, Delft. (In Dutch)

SYSAV, (2000) *Krossad betong. Ett återvinningsprojekt*. Delrapport 1 September 2000. Sydvästra Skånes AvfallsAB, Malmö. (In Swedish)

Tielaitos Geokeskus, (1994) *Kalliomurskeiden tilvistyminen ja hienoneminen*, Loppuraportti, ISBN 0788-3722, Oulu. (In Finnish)

TRL, (2000) *ALT-MAT Draft final report for publication*. Report No WP6.TRL.001, www.trl.co.uk/altmat/index.htm. Transport Research Laboratory (TRL), Crowthorne.

Verhulst, D., Buekens, A., Spencer, P.J. & Eriksson, G., (1996) *Thermodynamic behavior of metal chlorides and sulfates under the conditions of incinerator furnaces*. Environmental Science & Technology 30, pp 50–56.

VV-konsult, (2002). *Fallviktsmätning vid Dåvamyrans värmeverk*. Unpublished data. Vägverket Konsult.

Ydrevik, K., (1996) *Dynamic triaxial testing of roadbase and lower sub-base materials*. VTI notat no 4-1996. Swedish National Road and Transport Research Institute, Linköping. (In Swedish, but partly translated into English)

Ydrevik, K., (1999a), (1999b) *Förstärkningslagermaterial av krossade betongslipers. Erfarenheter från provsträckor på väg 869 vid Stenstorp*. Del 1: Byggnadsrapport. VTI notat nr 5-1999 and Del 2: Resultatrapport hösten 1999. VTI notat nr 45-1999. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Ydrevik, K., (1999c) *Utlåtande beträffande krossad vägbetongs mekaniska egenskaper och lämplighet som användning i bär- och förstärkningslager*. VTI utlåtande nr 644. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Ydrevik, K., (1999d) *Återvägen, Råd och vägledning för återvinning av krossad betong som ballast i gator och vägar*. VTI notat nr 67-1999. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Ydrevik, K., Hellström, V. & Molin, C., (1996) *Krossad betong som vägbyggnadsmaterial, Samband mellan cylindertryckhållfasthet och vägtekniska egenskaper*. VTI notat nr 46-1996. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Yoshikane, T., (1988) *The instances of concrete recycled for base course material in Japan*. In: Kasai (edited) Proceedings of the Second International RILEM Symposium on demolition and reuse of concrete and masonry, Vol 2. Chapman & Hall. pp 756–765.

Zheng, J., Johnson, P.F. & Reed, J.S., (1990) *Improved equation of the continuous particle size distribution for dense packing*. Journal of American Ceramic Society, Vol. 73, No. 5. pp 1392–1398.

Personal communication

Carlsson, Håkan, (2000): Väg- och transportforskningsinstitutet, Linköping.

Jansson, Håkan, (1999), (2000): Väg- och transportforskningsinstitutet, Linköping.

Standards

DIN 18 134. (1990) *Plattendruckversuch: Baugrund, Versuche und Versuchsgeräte*. Deutsche Normen, Köln

Draft prEN 933-11:2002 (E). *Classification test for the constituents of coarse recycled aggregates*. CEN.

EN 1744-1. *Tests for chemical properties of aggregates – Chemical analysis*. CEN.

prEN 13137. August 1999. *Characterization of waste, Determination of total organic carbon (TOC) in waste, sludges and sediments*. CEN.

EN 13242. *Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction*. CEN.

EN 13285. *Unbound mixtures – Specification*. CEN.

prEN 13286-7. April 2003. *Unbound and hydraulically bound mixtures. Part 7: Cyclic load triaxial test for unbound mixtures*. CEN.

prEN 13286-47. *Unbound and hydraulically bound mixtures – Test method for bearing capacity – Determination of California Bearing Ratio, immediate bearing index and linear swelling*. CEN.

FAS 259-98. *Bestämning av kulkvarnsvärde*. FAS Metoder 1998. Metodbeskrivningar för provning av stenmaterial, bituminösa bindemedel, asfaltbeläggningar och –massa. Föreningen för asfaltbeläggningar i Sverige (FAS), Stockholm.

SS 02 71 05. (1990). *Geotekniska provningsmetoder – Organisk halt i jord – Glödgningsförlustmetoden. (Glödgning vid 950ºC).* Svensk standard Standardiseringskommissionen i Sverige (SIS), Stockholm.

SS 02 71 07 utgåva 3. (1991). *Geotekniska provningsmetoder – Organisk halt i jord – Kolorimetermätning.* Svensk standard Standardiseringskommissionen i Sverige (SIS), Stockholm.

SS 02 81 13. (1981). *Vattenundersökningar – Bestämning av torrsubstans och glödgningsrest i vatten, slam och sediment. (Glödgning vid 550ºC).* Svensk standard Standardiseringskommissionen i Sverige (SIS), Stockholm.

VVMB 34:1984. *Jordmaterial: glödgningsförlust.* Publ 1987:168. Vägverkets metodbeskrivningar, Vägverket, Borlänge.

VVMB 606:1993. *Jord- och vägbyggnadsmaterial: Bestämning av bärighetsegenskaper med statisk plattbelastning.* Publ 1993:019. Vägverkets metodbeskrivningar, Vägverket, Borlänge.

VVMB 112:1998. *Deflektionsmätning vid provbelastning med fallviktsapparat.* Publ 1998:80. Vägverkets metodbeskrivningar, Vägverket, Borlänge.

PAPER I

MECHANICAL PROPERTIES OF PROCESSED MSWI BOTTOM ASH, evaluated from laboratory and field tests

Maria Arm
Swedish Geotechnical Institute (SGI), SE-581 93 Linköping, Sweden
maria.arm@swedgeo.se

ABSTRACT

In order to facilitate the design of Swedish roads with municipal solid waste incinerator (MSWI) bottom ash, its mechanical properties were studied in the laboratory and in the field and the results were compared to corresponding results for natural aggregates, such as sand, gravel and crushed rock. In the laboratory, an extensive series of cyclic load triaxial tests were carried out together with standardised durability tests. Test sections with ash in the sub-base were monitored with falling weight deflectometer (FWD) measurements and compared with reference sections with natural aggregates.

The triaxial test results showed that the bottom ash was as stiff as natural gravel but showed less permanent deformation due to more angular particles. When tested at high stresses the bottom ash had a lower resilient modulus than crushed granite, but did not fail until the stress reached levels beyond what was expected in a Swedish sub-base. The micro-Deval tests and the freeze-thaw tests caused greater fine-material formation and disintegration in the ash materials than is normally obtained in natural aggregates.

It could be concluded from the field tests that one year after construction the mean stiffness for the bottom ash sections was lower than for the reference sections with crushed granite. Slow stiffness increase with time was observed in a 12-year-old bottom ash layer but as yet not in the one-year-old test sections.

The conclusion of the study was that the MSWI bottom ash can replace not only sand but also natural gravel in unbound layers, if the content of organic matter is kept low. It can also replace crushed granite if the bound surface layer reduces the mean normal stress in the bottom ash layer to below 150 kPa. This implies that the ash is best used in roads with high traffic volumes since these have the thickest bound layers.

Key words: MSWI bottom ash; mechanical properties; cyclic load triaxial tests; resilient modulus; road materials; residues

1 INTRODUCTION

In recent years several environmental objectives have been adopted by the Swedish Parliament. One example is that natural gravel should only be used for construction purposes when there is no suitable alternative considering the use in question. Furthermore, by 2010 at least 15% of the total aggregate consumption should consist of reused material (Bill 2000/01:130, 2001). Since 2000 there has been a waste tax on waste deposited at landfill sites (SFS 1999:673). At the moment, the tax is SEK 370 per tonne of waste deposited. For deposited material that is reused in some way, such as in road construction, the waste tax is repaid.

Bottom ash from the incineration of municipal solid waste, here called MSWI bottom ash, has not yet been used in Sweden in large-scale projects. According to the literature, there is little data on deformation properties of MSWI bottom ash. Chandler et al. (1997) refer to a limited database on California Bearing Ratio (CBR) values, where bottom ash generally shows higher (=

better) values than those of most natural soil materials. They conclude that more laboratory data are required to ensure that the comparison is statistically valid. Sweere (1990) reports some data from triaxial tests on Dutch waste incineration slag (AVC). Furthermore, the resistance to wear and disintegration of Swedish MSWI bottom ash has hardly ever been investigated.

The aim of this project was to facilitate the design of Swedish roads with MSWI bottom ash. Its mechanical properties were therefore investigated and a comparison of the results with conventional Swedish road materials was made. The main part of the investigation consisted of cyclic load triaxial tests, a method that is being used more and more, especially in performance testing. The results from a large number of cyclic load triaxial tests already performed on Swedish natural aggregates and soils were used for comparison (Ydrevik, 1996; Arm, 1996, 1997, 1998).

Variations in the deformation properties caused by different incinerators and seasonal variations in the waste were also studied and are reported elsewhere (Arm, 2003).

2 MATERIALS

Processed MSWI bottom ash from five different plants was studied. All the plants were of the mass-burn type with moving grates and incinerate both industrial and municipal waste. The age of the furnaces varied between 1 and 14 years.

At four of the plants – Stockholm (S), Gothenburg (G), Malmö (M) and Linköping (L) – bottom ash was sampled during four different periods over a year (autumn 1996–summer 1997). Twenty sub-samples of approximately 500 kg each were collected from each plant and for each period. The sampling procedure and detailed results from that study are described in (Arm, 2003). From one of those plants, Malmö, new samples were taken in 1999. The producer took these samples from stockpiled ash. In 2001, bottom ash from a new plant, Umeå (U), completed the study.

All the materials were processed. They were screened to remove magnetic material and >50-mm particles and then stored outdoors. The U –01 material was stored for 5–6 months, the –97 materials for 1–2 years and the M –99 material for more than two years.

The M –99 and the U –01 materials were also used on test roads that were monitored within the project.

3 METHODS

The laboratory methods used in this study were

- Cyclic load triaxial tests: This type of test involves the investigation of the material deformation under simulated traffic conditions. Cylindrical specimens consisting of the composite material up to a certain particle size are compacted and exposed to repeated loads of different magnitudes. A CEN standard is under development and the method will be used to evaluate unbound granular materials in the future (prEN 13286-7). The draft standard describes different loading procedures.

  In this study the testing equipment at the Swedish National Road and Transport Research Institute (VTI) was used. The procedure used at VTI is most similar to the so-called multi-stage procedure, where the same specimen is exposed to several stress levels. The specimen diameter was 150 mm and the height was 300 mm. The target testing conditions were the optimum water content, w_{opt}, and a density of approximately 90% of the maximum density obtained in the modified proctor test. The specimens were compacted with simultaneous vibration and compression in a Vibrocompressor (prEN 13286-52). The values that are discussed here are averages of two or three specimens.

The vertical cyclic stress was varied between 10 and 1,220 kPa and the horizontal stress was kept constant at 10, 20, 60 or 120 kPa resulting in a deviator stress, q, between 30 and 1,240 kPa. During the test, the vertical deformations (both resilient and permanent) were measured and stored. The resilient strain was used to calculate the resilient modulus, M_r.

- Resistance to wear by means of micro-Deval tests (EN 1097-1) with an analysis fraction of 10–14 mm.
- Resistance to freezing and thawing (EN 1367-1) with an analysis fraction of 8–16 mm.
- Content of organic matter through loss on ignition, *LOI* at 550°C (SS 02 81 13).
- Optimum water content and maximum density by means of a modified proctor test.

The main constituents were analysed through visual classification of the 4–32 mm fractions. The choice of classification method and categories was based partly on earlier studies (Höbeda et al., 1985). All particles in a 600–900 g sample were classified into slag, glass, ceramics, metal or other. Each class was weighed and the weight percent of the total was calculated.

Comparisons were made with corresponding laboratory results for the natural aggregates that the MSWI bottom ash could possibly replace, such as sand, gravel and crushed granite with different grading.

In the field, test sections with ash in the sub-base and reference sections with natural aggregates were monitored by means of falling weight deflectometer, FWD, measurements and an evaluation of layer moduli and surface moduli was carried out. Three sites were monitored. One was built in southern Sweden in 1998. In this road the M –99 ash was used. Another was built in 2001 in northern Sweden and there the U –01 was used. In addition, an old test road from 1987 in Linköping was monitored and the result was compared with the results from measurements performed when the road was new.

4 RESULTS AND DISCUSSION

The sorted and stored bottom ash materials had the particle size distributions shown in Figure 1.

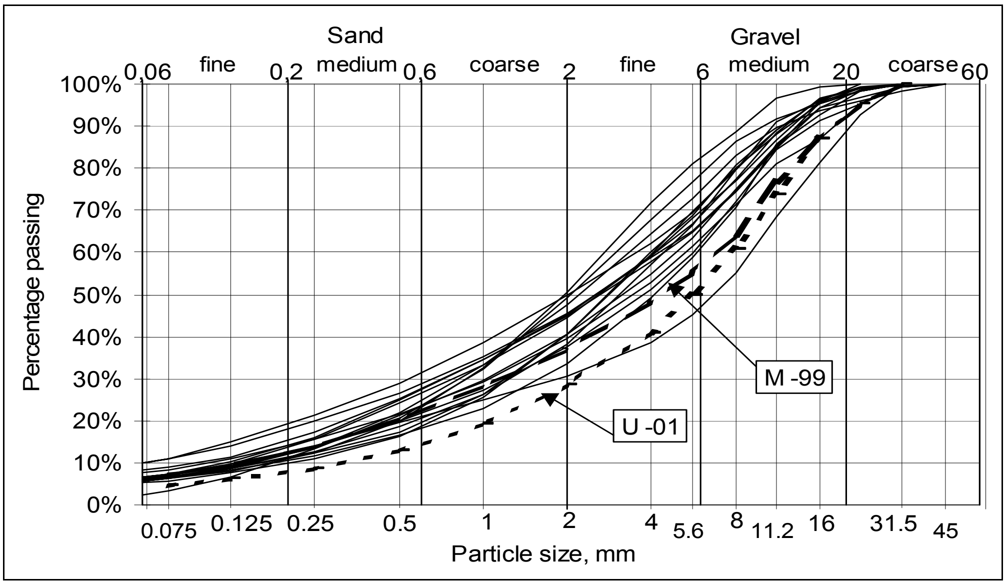

Figure 1. Particle size distribution for bottom ash studied. 18 curves in total, S –97, G -97, M –97 and L –97 with four sample periods each and M –99 and U –01.

The materials were well-graded with curves similar to that of sandy gravel.

The material composition among particles greater than 4 mm is shown in Figure 2. The main part consisted of slag and glass particles. The term 'slag' means sintered particles with no visible glass, ceramics or metal. The 'Other' category includes non-combusted organic matter, such as paper, plastic, textiles and wood.

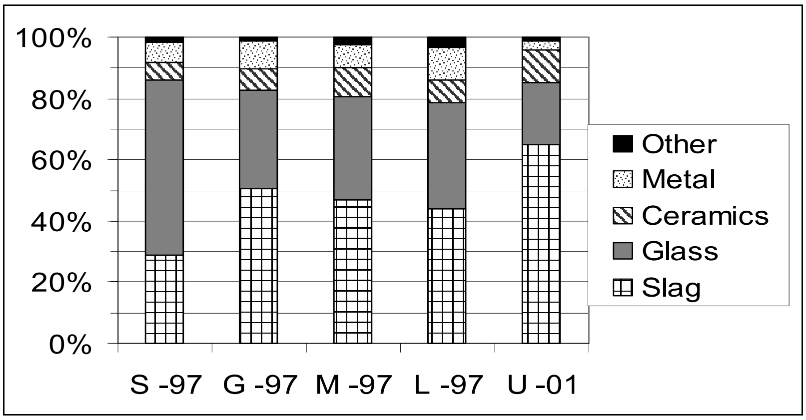

Figure 2. Main constituents of bottom ash particles in the 4–32 mm fractions from five incinerators (wt.-%). Annual mean value for the –97 materials.

The different ash materials contained between 2.3 and 7.7 wt.-% organic matter, expressed as an incinerator mean value from LOI at 550˚C. The value varied between incinerators and sample periods, as shown in Figure 3.

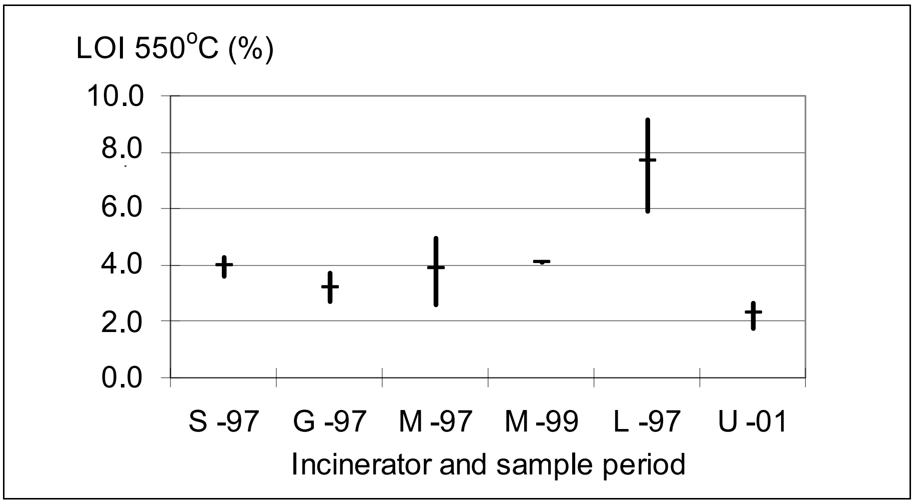

Figure 3. Loss on ignition at 550˚C for bottom ash from five plants. Annual mean value and range for the –97 materials (four duplicate samples each). Mean value and range for the U material (five duplicate samples).

Chemical composition and leaching properties were also investigated and are reported separately (Fällman, 2000). The chemical composition of the –97 materials was dominated by silicon, iron, calcium and aluminium and the dominating trace elements were zinc, copper and lead. In the leaching test at L/S 10, 87–91% of the leachate consisted of calcium, sulphur, chlorine and sodium. Copper dominated among the trace elements (Fällman et al, 1999). The results from U will be reported later.

4.1 Deformation properties according to cyclic load triaxial tests

The study resulted in resilient modulus, M_r, and permanent deformation for processed MSWI bottom ash at different loading conditions.

In Figure 4 the calculated M_r for ash materials studied are plotted against the mean normal stress, p. For comparison, the M_r of two sandy materials with similar grading and the M_r of one crushed granite with a particle size of 0–8 mm are plotted on the same graph.

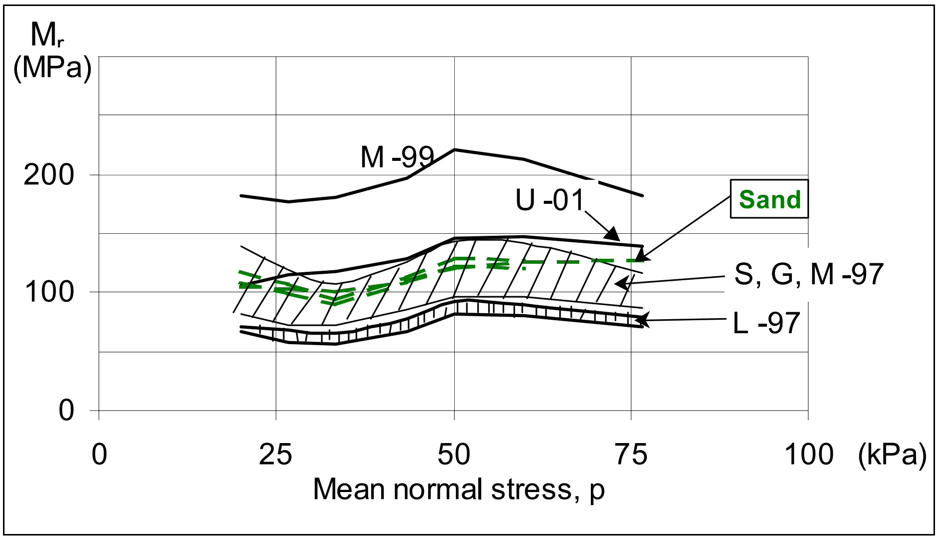

Figure 4. Resilient modulus, M_r, from cyclic load triaxial tests on bottom ash and sand. Two zones for ash materials from 1997 and separate results for two ash materials from 1999 and 2001. (Sand results from Arm, 1998).

The stress dependency of M_r was weak for all ash, as it is for sandy materials (Figure 4). The M –99 material differed strongly from the other. This could possibly be explained by the longer storage period. During the ageing of MSWI bottom ash several chemical reactions take place that stabilise the ash and improve the mechanical properties (Reichelt, 1996). The U –01 ash had only been stored for six months but despite this had a high M_r. However, this ash had the lowest content of organic matter of all the ash tested and also had a coarser particle size distribution.

In Figure 4 there is a significant difference between the zone of L –97 and the zone of S, G and M –97 as well as the U –01 material, which could be explained by different organic matter content (cf. Figure 3). According to earlier conclusions from the '–97 materials', a high organic matter content has a negative impact on M_r (Arm, 2003).

The permanent deformations of all bottom ash were rather small as long as p was lower than 60 kPa (Figure 5). They were also much smaller than the deformations of the sand materials. On the other hand, the permanent deformations of bottom ash and the crushed 0–8 material were about the same, which is not surprising bearing in mind the impact particle shape has on stability.

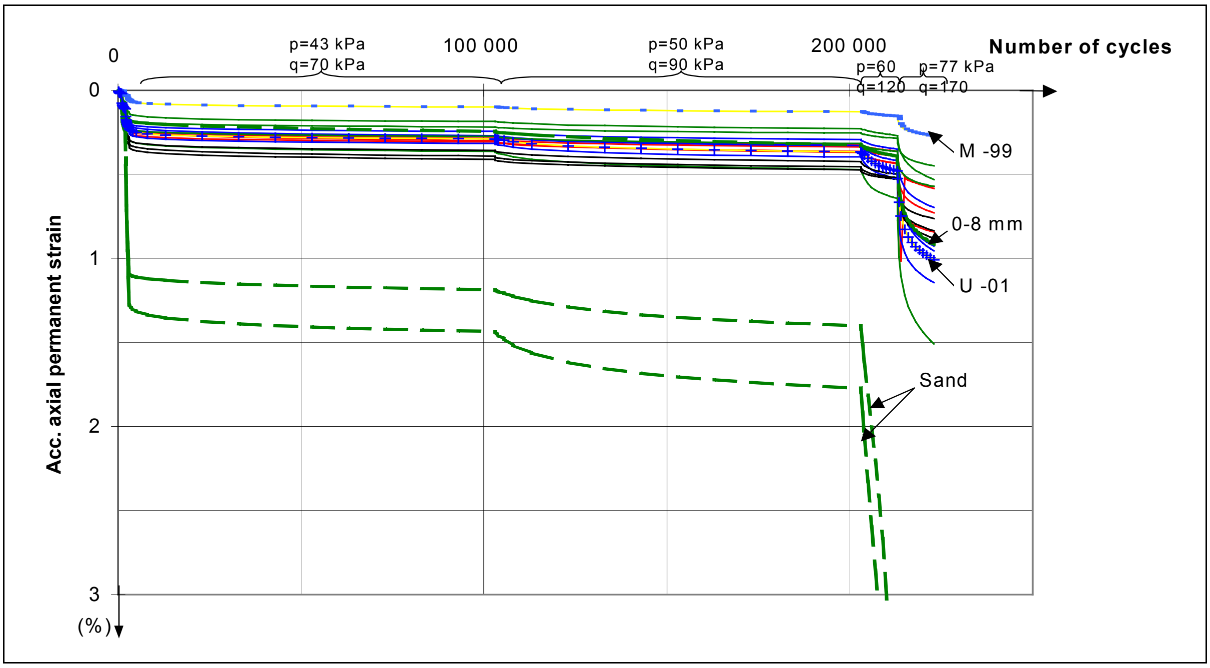

Figure 5. ***Permanent strains of all bottom ash, two sand materials and one crushed granite 0–8 mm.***

Two of the ash materials (M –99 and U –01) were also exposed to a cyclic load triaxial test with higher stresses, which resulted in failure at p = 400 kPa and q = 840 kPa. In Figure 6, comparable results are plotted for the M_r of bottom ash, sand and three well-graded 0–32 mm materials. Two of them were crushed granite materials approved as Swedish base course material and the third was natural gravel.

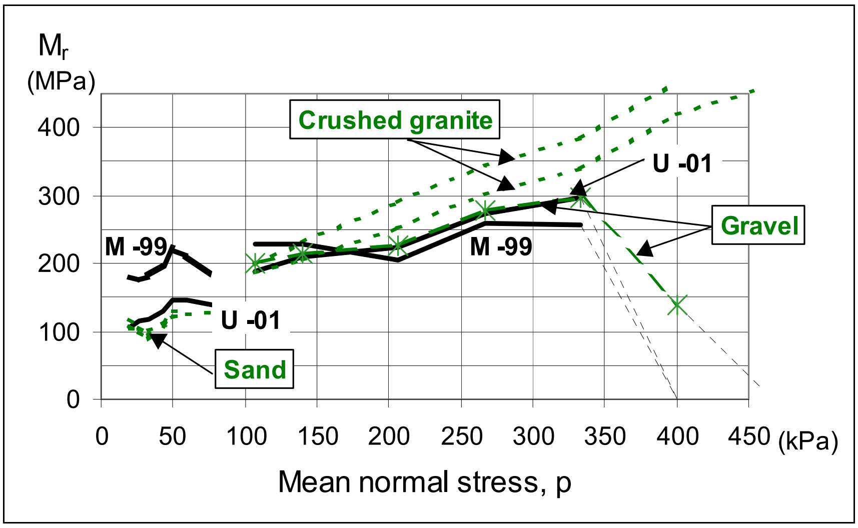

Figure 6. ***Resilient modulus, M_r, from cyclic load triaxial tests on two MSWI bottom ash materials compared with sand, gravel and crushed granite 0–32 mm. (Natural aggregate results from Ydrevik, 1996 and Arm, 1998).***

The M_r of the bottom ash was less stress-dependent than the M_r of the crushed granite. Up to p = 150 kPa the M_r were comparable, but above this the crushed granite was stiffer. The moduli of the U –01 ash and the natural gravel were almost identical (Figure 6).

The permanent deformations at higher stresses are plotted in Figure 7.

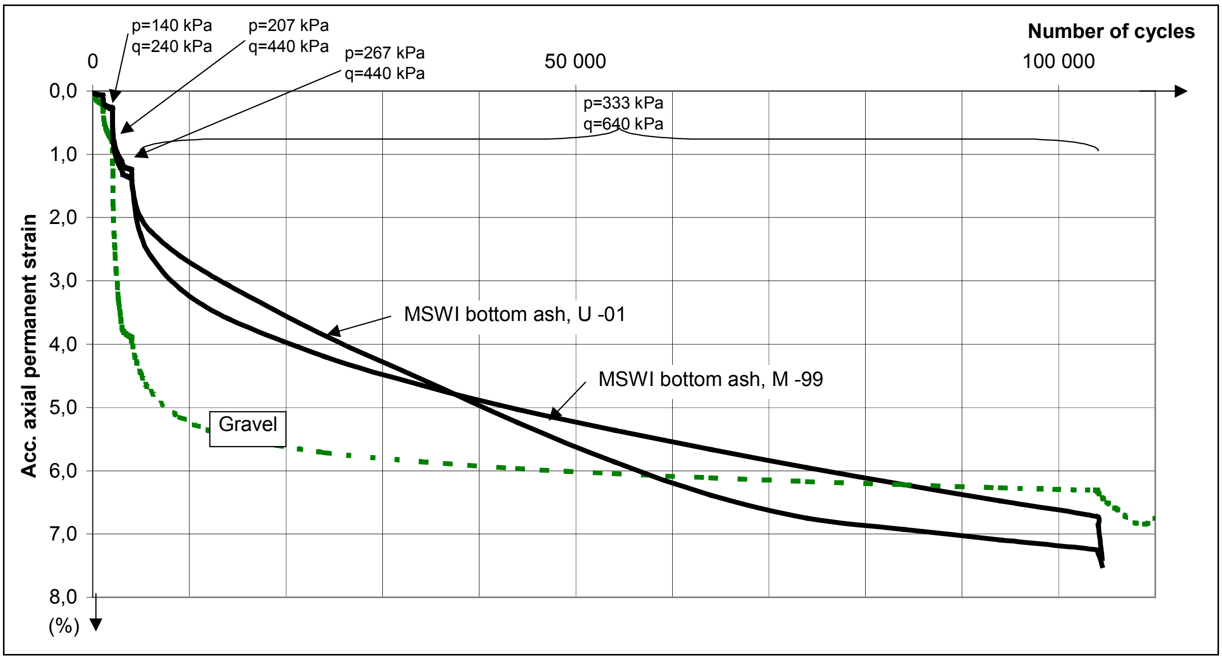

Figure 7. ***Permanent strains of two bottom ash and natural gravel 0–32 mm.***

Figure 7 could together with Figure 6 be interpreted in such a way that $p \approx 150$ kPa (with q/p = 1.7) is a stress limit that should not be exceeded if the deformation is to be limited. Here, it is important to note that besides the level of p the ratio between q and p has a considerable impact on the strain. When the ratio approached 2.0 there was an extreme increase in the strain (Figure 7). This could also be seen, but not to the same extent, in Figure 5.

The susceptibility of deformation properties to water content was investigated for the two ash materials M –99 and U –01. When the water content was increased, the M_r of the bottom ash U –01 decreased, while the M_r of the M –99 ash increased. Figure 8 shows the result at one stress condition, p = 43 kPa and q = 70 kPa, although the pattern was similar on all stress levels.

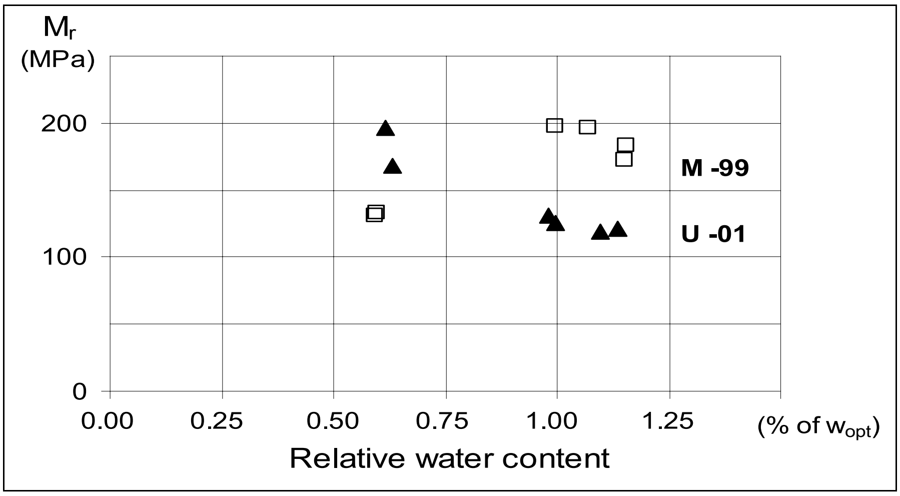

Figure 8. ***Resilient modulus, M_r, at different water contents for two bottom ash materials. From triaxial tests with q/p = 70/43 kPa. Water content related to optimum water content from modified proctor tests.***

U –01 behaved as a natural aggregate, whereas M –99 did not. Well-graded natural aggregates and natural aggregates containing a relatively high proportion of fines have the highest M_r values in the partly-saturated condition (Arm, 1996, 1998; Kolisoja, 1997). The M –99 ash was more porous and more fine-grained than the U ash, which could possibly explain the behaviour. Perhaps w_{opt} was not obtained due to the fine and porous particles. The water content was thus still on the dry side. The permanent deformation result produced the same difference between materials, with the U ash deforming more and more with increasing water content and the M –99 ash becoming more insensitive to both water content and stress level.

4.2 Deformation properties according to FWD measurements

The cross-sections of the three test roads are presented in Figure 9.

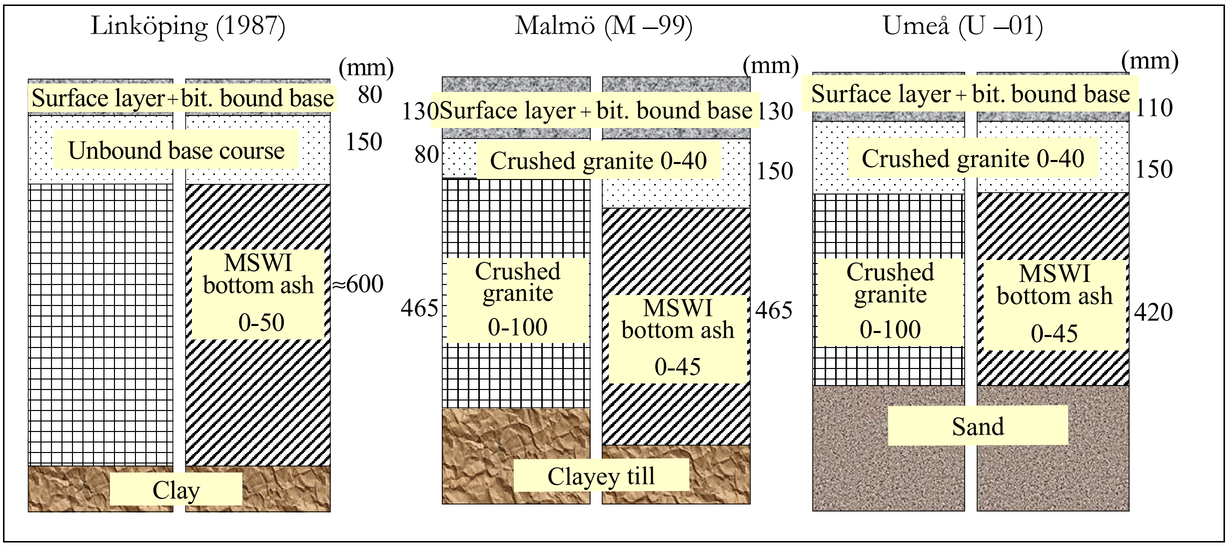

Figure 9. Test sections and reference sections in Linköping, Malmö and Umeå.

The aim for all the test roads was that only the material in the sub-base should differ between the test sections and reference sections. However, the thicker base course on the test section in Malmö (Figure 9) was chosen because the bottom ash was not supposed to cope with just a 13+8 cm overlay. Furthermore, the bottom ash could be regarded as a non-crushed material and in those cases the design guide prescribes a 15 cm base course instead of 8 cm.

Linköping

This test road was built and measured for the first time in 1987 (Jacobson & Viman, 1989). However, the results from the old and the new FWD measurements could not be compared directly since both the measurement system and the calculation procedure had changed. A relative comparison was therefore made, which means the relationship between the ash section stiffness and the reference section stiffness was compared at different times. The results from measurements in 1987, 1988 and 1999 are compared in Table 1.

Table 1. Results from FWD measurements on the test road in Linköping. Ratio between the layer modulus of the bottom ash section and layer modulus of the reference section. The sub-base (SB) and the subgrade were regarded as one layer in the calculation. (Data from Jacobson & Viman, 1989 and Jansson, 1999).

Date	Layer modulus (SB of *ash* + subgrade) / Layer modulus (SB of *gravel* + subgrade)
1987-11	55%
1988-05	79%
1988-10	92%
1999-08	101% (ratio between sub-bases) 143% (ratio between subgrades)

It could be concluded that in November 1987, just after the road was constructed, the compound layer modulus of the ash section was a good half of the value for the reference section. After one year they were almost equal and in August 1999 there was no difference at all.

Malmö

So far, FWD measurements have been carried out on two occasions on the test road outside Malmö. In both measurements the mean stiffness calculated for the ash sub-base was about 65% of the stiffness in the crushed rock sub-base (Figure 10). This lower stiffness in the unbound layer implies a higher tensile strain in the bottom of the asphalt layer. In this case the strain was calculated to be about 50% greater in the bottom ash section.

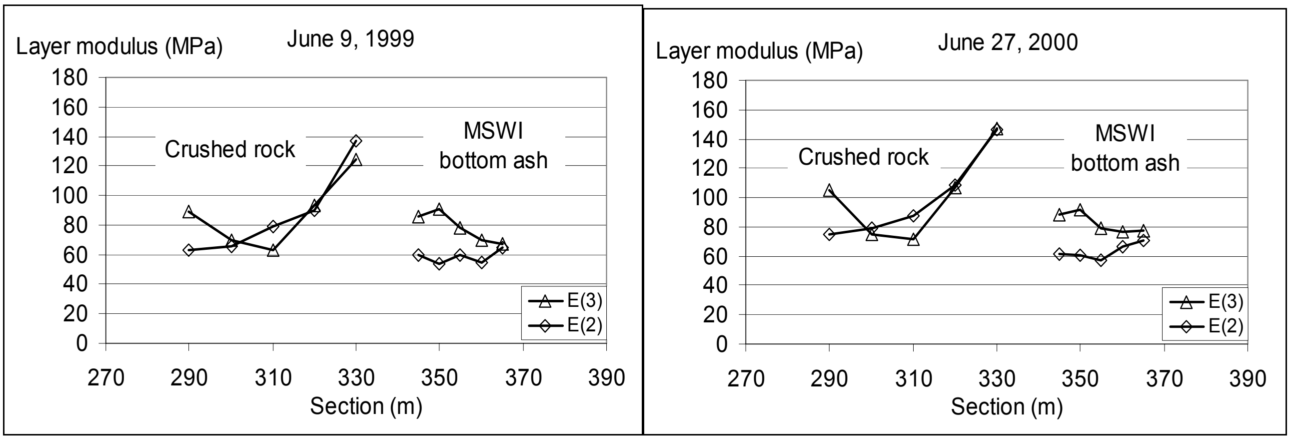

Figure 10. Layer modulus evaluated from FWD measurements on the test road in Malmö in 1999 and 2000 respectively. E(2) is the modulus for a combined layer, including the unbound base of crushed granite and the sub-base of either crushed granite or bottom ash. E(3) is the subgrade modulus. (Data from Jansson 1999 and 2000).

No increase in stiffness has developed so far in the bottom ash layer, except for a slight increase in layer modulus, which is observed in all materials (Figure 10). This increase was explained by traffic compaction and drainage of the subgrade. The fact that the sub-base and base had a similar or even lower layer modulus than the subgrade was probably due to the subgrade type and the poor construction conditions. The subgrade consisted of clayey till, which normally has rather high modulus values. The road was constructed in November–December 1998 with rain, snow and several days with temperatures below zero, which obstructed the compaction of all layers.

Umeå

In Umeå, only one FWD measurement has been carried out so far. This was performed one year after construction, in September 2002, and showed that the ash section stiffness was around 77% of the reference gravel section. In Figure 11 calculated surface moduli are plotted for each deflection point (E0, E200, E300 etc) against the corresponding depth. There are two figures since the measurements showed that the subgrade stiffness varied across the road, with the stiffest part on the right-hand side. The weakest layer in both road sections was the sub-base, which is represented by the depth 260 to 680 mm in Figure 11. If the variation in subgrade stiffness is taken into account, the bottom ash section stiffness was 87% of the reference section stiffness.

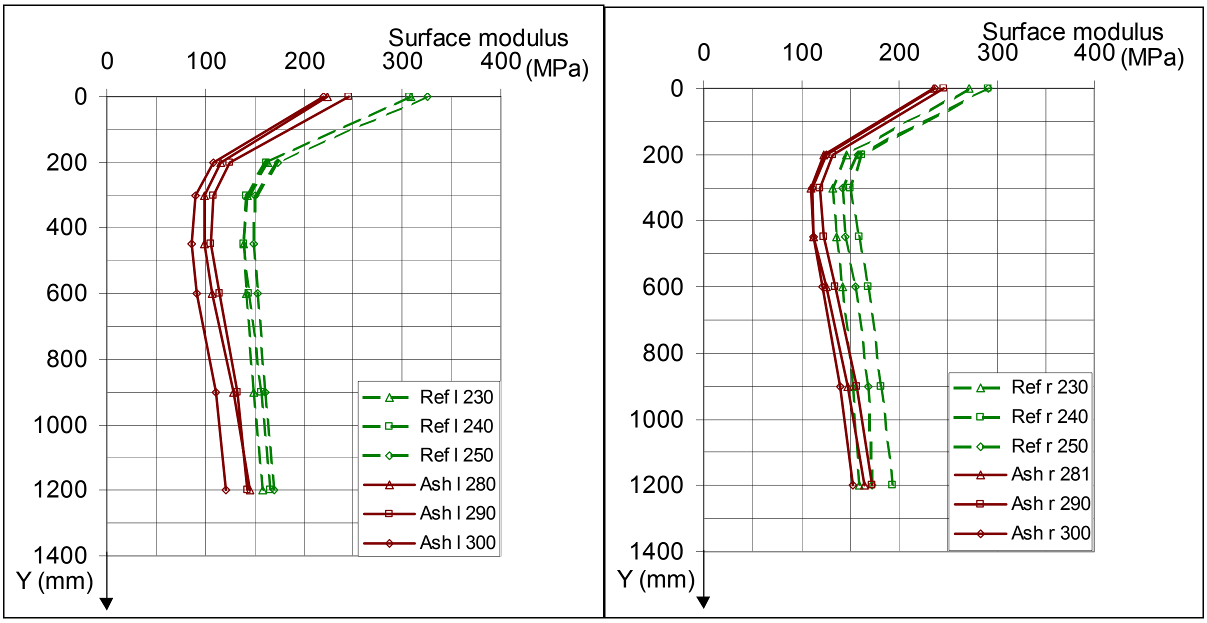

Figure 11.** **Results from FWD measurement on the test road in Umeå, left- and right-hand sides of the road respectively. The dotted lines represent the reference section. (Data from VV Konsult, 2002).

4.3 Resistance to wear and to freeze-thaw changes

The ash materials obtained micro-Deval values of between 26 and 39 and the freeze-thaw test resulted in disintegration values of between 0.8% and 7.7% (Figure 12).

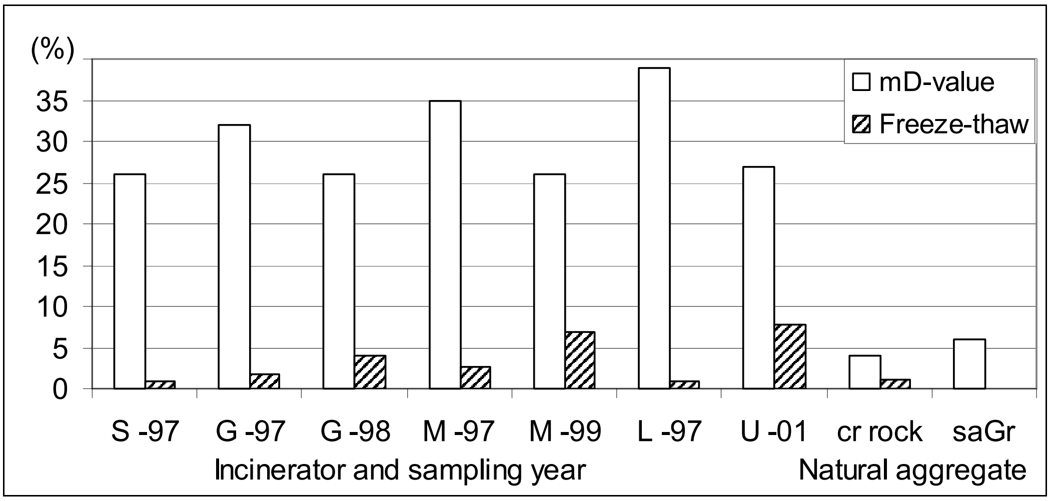

Figure 12. Results from micro-Deval and freeze-thaw tests on materials investigated together with crushed, fine-grained granite and sandy gravel. (Values for G –98 and saGr from SNRA, 2001).

When comparing particle size distribution before and after modified proctor test, low resistance was also obvious. In Iceland, such a comparison is used as a measure of material strength (Bjarnasson et al., 1999). The resistance to disintegration through Los Angeles (LA) tests was not performed within this study but an earlier Swedish study reports a value of 45.4 compared with 22.7 for sandy gravel (SNRA, 2001). The values obtained can be compared with resistance results from foreign investigations. Chandler et al. (1997) reports LA values according to ASTM C131 of between 10% and 90% for bottom ash of varied origin. Pfrang-Stotz & Reichelt (2000) report freeze-thaw disintegration values of between 3.5 and 12.9 wt.-%. Thus, all tests on bottom ash show lower resistance to wear and disintegration than would normally be the case for natural aggregates. This must be taken into account during construction, when choosing the compaction tool and when planning for construction traffic, whereas the final traffic only has a very slight impact if the road is designed properly.

The low resistance to wear and freeze-thaw changes can be attributed to the porous particles. A good relationship was thus obtained between the disintegration and the slag content in the fraction tested (cf. Figure 2). Pfrang-Stotz & Reichelt (op.cit.) also mention the mineral composition and relatively weak carbonate bond between the bottom ash constituents as an explanation.

Finally, the traditional test methods, such as Los Angeles and micro-Deval tests, were developed to evaluate the mechanical properties of natural granular materials. These methods have been called into question more and more in recent times. The ALT-MAT project, for instance, concluded that the usefulness of these tests as indicators of *in situ* performance could be questioned (TRL, 2000). Instead, when comparing performance, performance-related test methods must be used.

5 Conclusions

Ash materials from five different incinerators were studied. The MSWI bottom ash materials were well-graded, with curves similar to that of sandy gravel. The main part consisted of slag and glass particles. The different ash materials contained between 2.3 and 7.7 wt.-% organic matter expressed as an incinerator mean value from LOI at 550°C.

According to the triaxial tests the bottom ash was as stiff as natural gravel but showed less permanent deformation due to more angular particles. Furthermore, it was obvious that a high content of unburned material limits the stiffness. At low stress levels (p <75 kPa) the calculated resilient moduli, M_r, ranged between 75 and 150 MPa, depending on the material and stress level, if the content of organic matter was kept below 5%. One single ash had an M_r of between 175 and 225 MPa, which could possibly be explained by a longer storage period and subsequent ageing before testing. When tested at higher stress levels (p = 100–350 kPa) the bottom ash gave a lower M_r (between 200 and 300 MPa) than crushed granite, but did not fail until the stress reached stress levels beyond what was expected in a Swedish sub-base.

From the FWD measurements it could be concluded that both initially and one year after construction the mean stiffness of the bottom ash sections was lower than for the reference sections with crushed granite. Slow stiffness increase with time was observed in a 12-year-old bottom ash layer but not yet in the new test sections.

All ash materials showed greater fines formation and disintegration than natural aggregates when tested in micro-Deval tests and freeze-thaw tests. This must be taken into account in construction work when choosing the compaction tool and when planning for construction traffic, whereas the final traffic only has a very slight impact if the road is designed properly.

The conclusion of the study was that the MSWI bottom ash can replace not only sand but also natural gravel in unbound layers, if the content of organic matter is kept low. It could also replace crushed granite if the bound surface layer reduces the mean normal stress in the bottom ash layer to below 150 kPa. This implies that the ash is best used in roads with a high traffic volume since these have the thickest bound layers.

ACKNOWLEDGEMENTS

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REFERENCES

Arm, M., (1996), (1997), (1998) *Obundna vägmaterials mekaniska egenskaper.* (*Mechanical properties of unbound road materials.* Annual project reports to the National Road Administration.) Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Arm, M., (2003) *Mechanical properties of residues as unbound road materials – experimental tests on MSWI bottom ash, crushed concrete and blast furnace slag.* Dissertation. Royal Institute of Technology, Stockholm. (In print)

Bill 2000/01:130. (2001) *Svenska miljömål – delmål och åtgärdsstrategier.* Fritzes offentliga publikationer, Stockholm. (English short version *Swedish Environmental Objectives – Interim Targets and Action Strategies* available from the Swedish Environmental Objectives Council)

Bjarnason, G., Erlingsson, S. & Petursson, P., (1999) *COURAGE – Aggregate resistance to fragmentation, weathering and abrasion – comparison of different test methods.* BUSL report no E-37. Icelandic Building Research Institute, Reykjavik.

Chandler, A.J., Eighmy, T.T., Hartlén, J., Helmar, O., Kosson, D.S., Sawell, S.E., van der Sloot, H.A. & Vehlow, J., (1997*) Municipal Solid Waste Incinerator Residues.* Studies in Environmental Science 67. The International Ash Working Group (IAWG), Elsevier.

EN 1097-1. *Tests for mechanical and physical properties of aggregates – Part 1: Determination of resistance to wear (micro-Deval).* CEN.

EN 1367-1. *Tests for thermal and weathering properties of aggregates – Part 1: Determination of resistance to freezing and thawing.* CEN.

prEN 13286-7, April 2003. *Unbound and hydraulically bound mixtures. Part 7: Cyclic load triaxial test for unbound mixtures.* CEN.

prEN 13286-52, April 2003. *Unbound and hydraulically bound mixtures – Methods for making test specimens – Part 52: Making specimens by vibro-compression.* CEN.

Fällman, A-M., (2000) *Significance in the results of total composition and potential leachability of screened MSWI BA from different plants and sampling periods.* In: Woolley, Goumans & Wainwright (edited) Waste Materials in Construction. Elsevier Science Ltd. pp 355–370.

Fällman, A-M., Larsson, L. & Rogbeck, J., (1999) *Slaggrus Miljömässiga och materialtekniska egenskaper.* Svenska Renhållningsverksföreningen RVF, Malmö och Statens geotekniska institut, Linköping. (In Swedish)

Höbeda, P., Jacobson, T. & Viman, L., (1985) *Undersökning av förbränningsrest från SYSAV, Malmö, genom laboratorie- och provvägsförsök.* VTI meddelande nr 413. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Jacobsson, T. & Viman, L., (1989) *Provvägsförsök Gärstad –87, Slagger från kol- och sopförbränning, Lägesrapport 8901.* VTI notat nr V80. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Jansson, Håkan, (1999), (2000) Väg- och transportforskningsinstitutet, Linköping. Personal communication.

Kolisoja, P., (1997) *Resilient Deformation Characteristics of Granular Materials.* Dissertation. Publication 223. Tampere University of Technology.

Pfrang-Stotz, G. & Reichelt, J., (2000) *Municipal solid waste incineration (MSWI) bottom ash: characteristics and the use in road construction.* In: Dhir, Dyer & Paine (edited) Sustainable Construction: Use of Incinerator Ash, International Symposium in Dundee 2000. Thomas Telford Ltd, London. pp 235–246.

Reichelt, J., (1996) *Mineralogische Aspekte bautechnischer und umweltrelevanter Eigenschaften von Müllverbrennungsschlacken.* Institut für Strassen- und Eisenbahnwesen der Universität Karlsruhe.

SFS 1999:673. *Lag om skatt på avfall.* Ändrad genom SFS 1999:1321 och 2002:998. Fritzes, Stockholm. (In Swedish)

SNRA, (2001) *Provningsmetoder för alternativa material till vägunderbyggnad*. Publikation 2001:34. Vägverket, Borlänge. (In Swedish)

SS 02 81 13, (1981) *Vattenundersökningar – Bestämning av torrsubstans och glödgningsrest i vatten, slam och sediment*. Swedish standard SIS, Stockholm. (In Swedish)

Sweere, G.T.H., (1990) *Unbound granular bases for roads*. Dissertation. Delft University of Technology.

TRL, (2000) *ALT-MAT Draft final report for publication.* Report No WP6.TRL.001. www.trl.co.uk/altmat/index.htm. Transport Research Laboratory (TRL), Crowthorne.

VV-konsult, (2002). *Fallviktsmätning vid Dåvamyrans värmeverk*. Unpublished data. Vägverket Konsult.

Ydrevik, K., (1996) *Dynamic triaxial testing of roadbase and lower sub-base materials*. VTI notat nr 4-1996. Swedish National Road and Transport Research Institute, Linköping. (In Swedish, but partly translated to English)

PAPER II

VARIATION IN DEFORMATION PROPERTIES OF PROCESSED MSWI BOTTOM ASH

Results from triaxial tests.

Maria Arm
Swedish Geotechnical Institute (SGI), SE-581 93 Linköping, Sweden
maria.arm@swedgeo.se

ABSTRACT

This study is part of a larger study of the mechanical properties of processed municipal solid waste incinerator (MSWI) bottom ash. The aim was to investigate the variation in deformation properties of the ash for future use in unbound road layers. The effect of the material variation was analysed in particular. Specimens of bottom ash from four different incinerator plants and four sampling periods over a period of one year were tested by means of cyclic load triaxial tests. The results showed that there were variations in the deformation properties of the materials. Although there were significant differences between incinerator plants, the seasonal fluctuations were not significant. The differences were mainly due to the organic matter content. It was also shown that the organic content has a limiting effect on the resilient modulus. For the material studied, the resilient modulus increased by 50% when the content of organic matter was halved. The conclusion was that the technical usability of MSWI bottom ash increases significantly if the content of organic matter can be kept low.

Keywords: MSWI bottom ash; unbound road materials; cyclic load triaxial tests; variation; organic matter content; resilient modulus

1 INTRODUCTION

Despite Sweden's relative richness in natural aggregate reserves there is a political ambition to increase the use of alternative materials, such as residues, in Swedish road construction (SFS 1999:673). One of the environmental objectives is that by 2010 the proportion of reused materials will represent at least 15% of the total aggregate used (Bill 2000/01:130, 2001). Consequently, several research projects are in progress aimed at characterising possible alternative materials and establishing design guidelines and environmental guidelines for utilisation.

One material that has not yet been used in Sweden in large-scale projects, is bottom ash from the incineration of municipal solid waste, in this context called MSWI bottom ash. There have been some demonstration projects and laboratory studies of bottom ash in which the so-called physical properties have been studied (Höbeda & Bünsov, 1979; Höbeda et al., 1985; Hartlén & Elander, 1986; Jacobsson & Viman, 1989). However, the variations in deformation properties between incinerator plants or between sampling periods have never been studied. Furthermore, the earlier studies were made during the 1980s and it is highly likely that waste has changed since then due to the present recycling policy, with a greater degree of sorting of household waste, thus producing another type of bottom ash.

This study is part of a larger study of the mechanical properties of processed MSWI bottom ash for use in unbound road layers. Since MSWI bottom ash is a heterogeneous material, owing to the heterogeneity in the incinerated waste (Chandler et al., 1997), the purpose here was to investigate variations in the deformation properties of the ash and to determine the normal range. Seasonal variations in the waste and different incineration processes were expected to exert an influence. Consequently, tests were conducted on bottom ashes from different sampling periods over the year and from different incinerator plants.

In earlier studies in Sweden and other countries, deformation properties were studied mostly using California Bearing Ratio (CBR) tests. A more recent method is the cyclic load triaxial test, which is being used more and more, especially in performance testing.

In the main study – of which this is part – cyclic load triaxial tests were performed by the Swedish National Road and Transport Research Institute (VTI). Complementary properties, such as composition, particle size distribution, content of organic matter, optimum water content and maximum dry density, were also investigated. A comparison with corresponding properties of conventional materials, such as different sandy materials and crushed granite, was made and is reported separately (Arm, 2003). Chemical and environmental characterisation of the same ash materials was also performed and the results are reported elsewhere (Fällman, 2000).

2 MATERIALS AND METHODS

MSWI bottom ash from four incinerator plants – Stockholm, Gothenburg, Malmö and Linköping – was chosen.

These plants incinerate both municipal and industrial waste. They are among the largest in Sweden and produce both heat and electricity. The production of bottom ash at each plant is between 35 000 and 80 000 tonnes per year (Table 1). This can be compared to the total for Sweden, which in 1997 was around 340 000 tonnes. All incinerators are of the mass-burn type with moving grates.

Table 1. Description of incinerator plants (RVF, 1997)

Incinerator plant	Incinerated waste (1 000 kg)	Proportion municipal waste (%)	Incinerator age and capacity		Remainder (1 000 kg)	
			Manuf. year	Capacity	Bottom ash	Fly ash
Stockholm	263 896	81	1986	15 tonnes/h	51 912	13 263
Gothenburg	381 500	60	1994	22 tonnes/h	79 302	13 457
Malmö	202 166	59	1983	14 tonnes/h	36 850	4 410
Linköping	225 585	76	1982	12 tonnes/h	49 500	9 501

2.1 Sampling

Sampling of the bottom ash was made as part of another study (Fällman, 2000). Great importance was assigned to treating the different materials in a similar way. Sampling took place at all plants at the same time during four two-week periods (September, December, March and June 1996–1997). Twenty sub-samples of approximately 500 kg each were collected from each plant and for each two-week period. The combined sub-samples were sieved in the same equipment to collect the non-magnetic 0–50 mm fractions. Finally, the 16 materials (four plants and four sample periods) were stored outdoors in wooden boxes at the same location. The sample periods are named autumn, winter, spring and summer.

In June 1998, representative samples were taken for triaxial tests and other laboratory analyses within the main project. The materials were then stored indoors until the triaxial testing took place during the period June 1998 – September 1999.

2.2 Material composition and grain size distribution

The composition of each material was defined through visual classification of the 4–32 mm fractions. The classification method and the categories were chosen partly based on earlier studies (Höbeda et al., 1985). All particles in a 600–900 g sample were classified into slag, glass, ceramics, metal or other. Each class was weighed and the weight percent of the total was calculated (Figure 1).

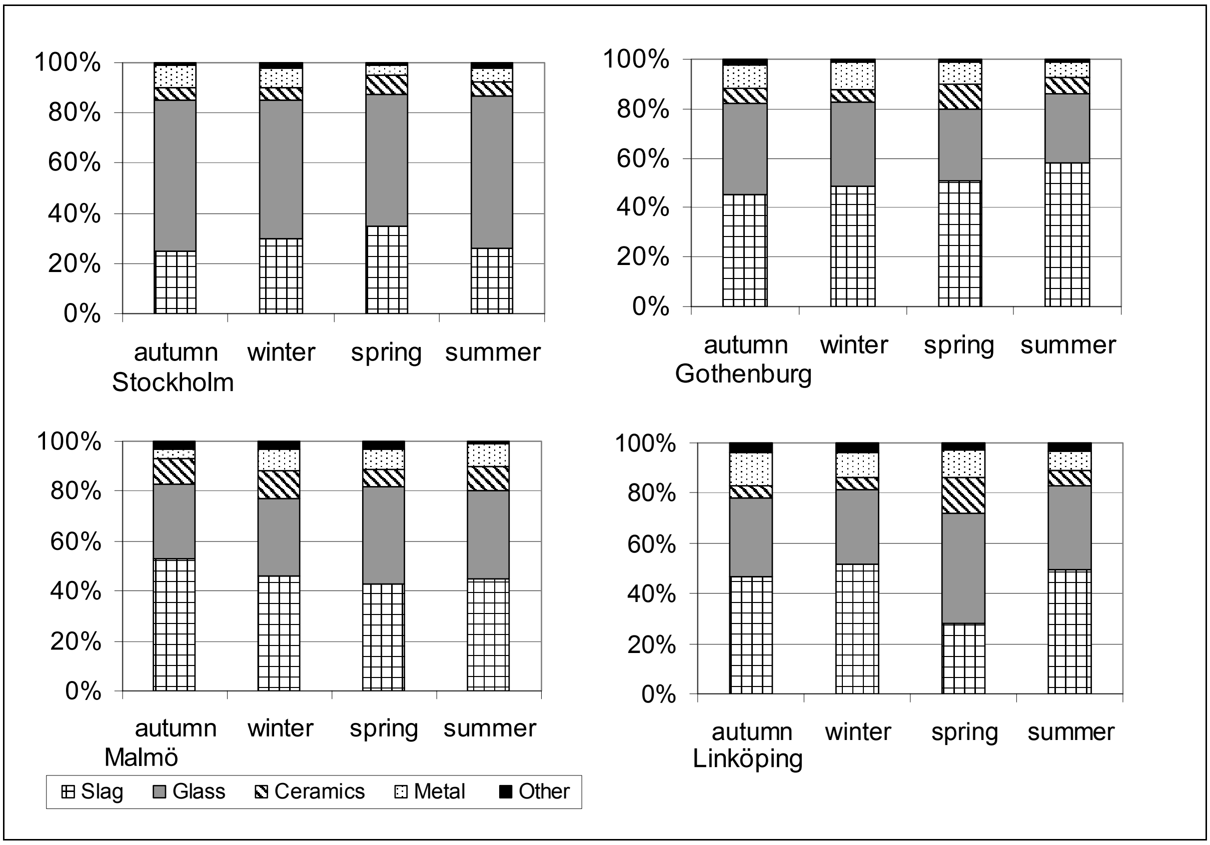


Figure 1. Main constituents of bottom ash particles in the 4–32 mm fractions from four different incinerator plants (wt.-%).

Each bar represents the material sampled during one two-week period. The term 'slag' means sintered particles with no visible glass, ceramics or metal. The 'Other' category includes non-combusted matter, such as paper, plastic, textiles and wood.

The four incinerator plants produce bottom ash with grain size distributions according to Figure 2.

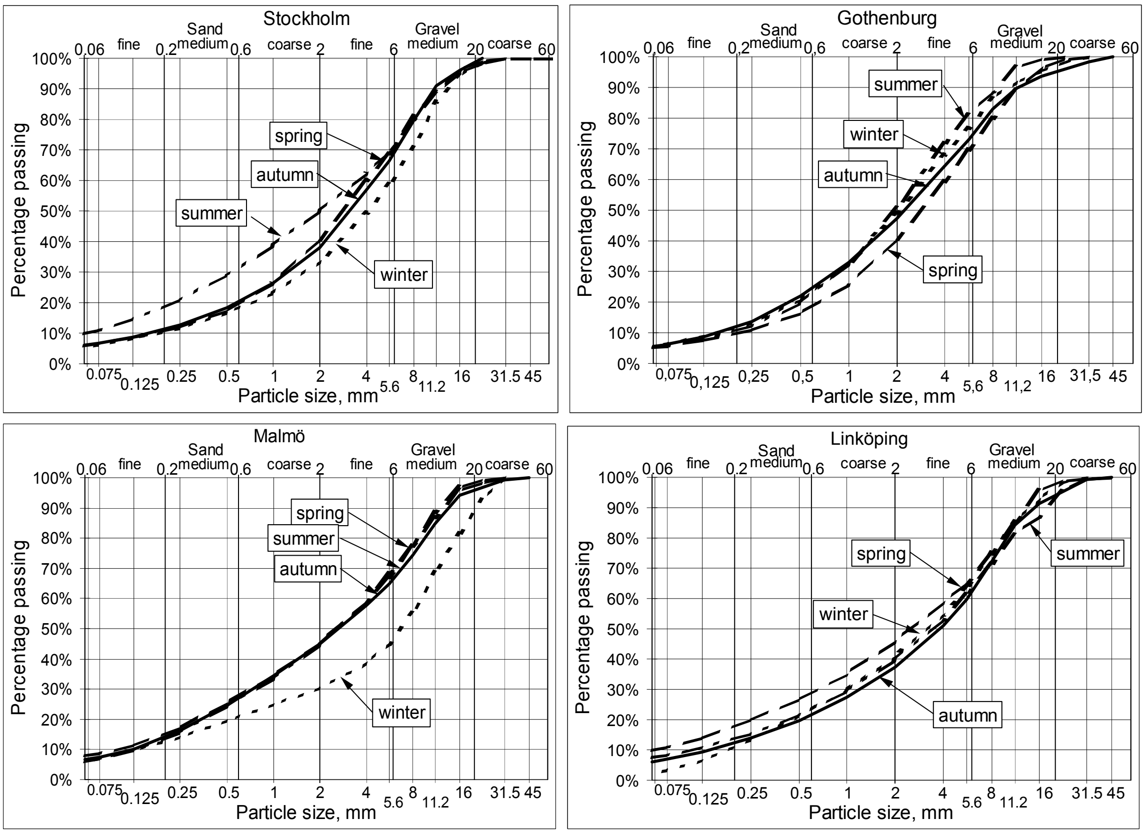


Figure 2. Grain size distribution for the materials studied, produced in 1996–97 and stored for 12–21 months. 0–50 mm fractions, wet-sieving method.

All materials were well-graded, with grain size distribution curves similar to that of sandy gravel.

Compaction tests were carried out by means of modified proctor and the content of organic matter was measured as loss on ignition (LOI) at 550°C. Tables 2 and 3 summarise the results.

Table 2. Maximum dry density and optimum water content, from modified proctor tests

	Stockholm		Gothenburg		Malmö		Linköping	
Season	Max. dens. (t/m³)	Opt. w (%)	Max. dens. (t/m³)	Opt. w (%)	Max. dens. (t/m³)	Opt. w (%)	Max. dens. (t/m³)	Opt. w (%)
Autumn	1.61	13.5	1.73	17	1.65	15–18	1.58	18.0
Winter	1.66	9.0	1.72	16.1	1.63	16.5	1.48	19.5
Spring	1.58	11–19	1.66	17–19	1.64	16.4	1.43	20.5
Summer	1.62	17.4	1.72	15–17	1.69	11–16	1.57	17.8
Mean value	*1.62*	*13.7*	*1.71*	*16.8*	*1.65*	*15.7*	*1.52*	*19.0*
Std dev.	*0.03*	*3.54*	*0.03*	*0.93*	*0.03*	*1.48*	*0.07*	*1.28*

Table 3. LOI at 550°C (%), sample A/sample B

	Stockholm	Gothenburg	Malmö	Linköping
Autumn	3.9/4.1	2.8/4.0	3.8/3.8	6.5/7.0
Winter	3.2/4.0	2.7/2.7	4.4/4.4	9.1/9.2
Spring	4.1/4.3	3.7/3.7	4.9/5.0	8.9/9.0
Summer	4.2/4.3	3.1/3.1	2.6/2.6	5.8/6.0
Mean value	*4.0*	*3.2*	*3.9*	*7.7*
Std dev.	*0.34*	*0.51*	*0.93*	*1.49*

2.3 Cyclic load triaxial tests

A cyclic load triaxial test involves the investigation of material deformation under simulated traffic conditions. The resilient strain is used to calculate the stiffness, expressed as resilient modulus, and the accumulated permanent axial strain can be used for classification purposes. Since the specimen exposed to loading consists of the whole composite material up to a certain grain size, it is in fact the function of the material that is tested. The principle of the method is well known, both for fine-grained and coarse-grained materials, and a CEN standard is under development (prEN 13286-7). The draft standard describes different loading procedures. The procedure used at VTI is most similar to the so-called multi-stage procedure, where the same specimen is exposed to several stress levels (Arm, 1996).

The tests were carried out on undrained specimens, 150 mm in diameter and 300 mm in height, in VTI's servo-hydraulic material testing system (VMS). This allowed ash with a maximum particle size of up to 30 mm to be tested. The specimens were prepared in one layer using vibrating compacting equipment, a 'Vibrocompressor' (prEN 13286-52), in a special cylinder. This is a method that has been used successfully for conventional aggregates. It results in a homogeneously compacted specimen (Paute et al., 1996) and the water content and the density of the specimen can be selected in advance. Furthermore, it is a 'friendly' method that was chosen in this study as earlier experiments showed a crushing tendency for these materials (SNRA, 2001). After compaction, the specimens were pushed out of the cylinder and fitted with plates at both ends and with a thin rubber membrane around them. For each material investigated, three specimens were compacted and tested.

It is well known, and it has also been shown with triaxial tests, that water content and density can affect the deformation properties of granular materials considerably (Sweere, 1990; Kolisoja, 1997). The aim, therefore, was to test all materials at the same relative water content and the same relative density. The conditions that were chosen were optimum water content and 90% of maximum dry density from modified proctor. However, problems in compacting some of the specimens resulted in differences in the attained compaction degree. In addition, the water content was above the optimum in all specimens except one. The difficulties in compacting the specimens were probably caused by material that was too angular and compaction that was too 'careful'. Results from another investigation (SNRA, 2001) showed that MSWI bottom ash had more angular particles than sandy gravel and also more than crushed concrete. The 'actual' testing conditions used are given in Table 4.

Table 4. Compaction degree (actual dry density/max. dry density from modified proctor tests) and relative water content (actual water content/optimum water content) for tested materials. Average of three specimens.

	Stockholm		Gothenburg		Malmö		Linköping	
	Compaction degree (%)	Rel. w (%)	Compaction degree (%)	Rel. w (%)	Compaction degree (%)	Rel. w (%)	Compaction degree (%)	Rel. w (%)
Autumn	84	96	89	100	85	108	86	105
Winter	87	101	89	105	90	102	88	103
Spring	88	100	89	105	82	105	80	104
Summer	86	106	89	105	88	110	83	105
Mean	*86*	*101*	*89*	*104*	*86*	*106*	*84*	*104*
Std dev.	*1.71*	*4.11*	*0*	*2.5*	*3.5*	*3.5*	*3.5*	*0.96*

2.4 Statistical evaluation

Significant parameter differences between incinerator plants and between sampling periods were analysed. The resilient modulus, the permanent deformation, the grain size distribution parameters and the content of organic matter were compared by means of analysis of variance (ANOVA). The grain size distribution parameters studied were fines content, uniformity coefficient c_u and d_{90}. (d_{90} means the mesh of the sieve, through which 90% of the material pass). A difference between groups was considered significant at the significance level of 95%.

In the ANOVA evaluations only those specimens that attained between 83 and 89% of maximum dry density and 100–108% relative water content were analysed. The purpose was to eliminate the impact of differences in relative density and water content on deformation at testing. The two intervals chosen reflect the mean value and the standard deviation in density and water content for all specimens. This means that 30 of the 48 specimens were analysed in the ANOVA evaluations. All groups were still represented except the spring material from Linköping.

3 Results

In Figure 3, the stiffness, expressed as resilient modulus for different stresses, is plotted for all 16 materials studied. It consistently shows the same weak stress dependency.

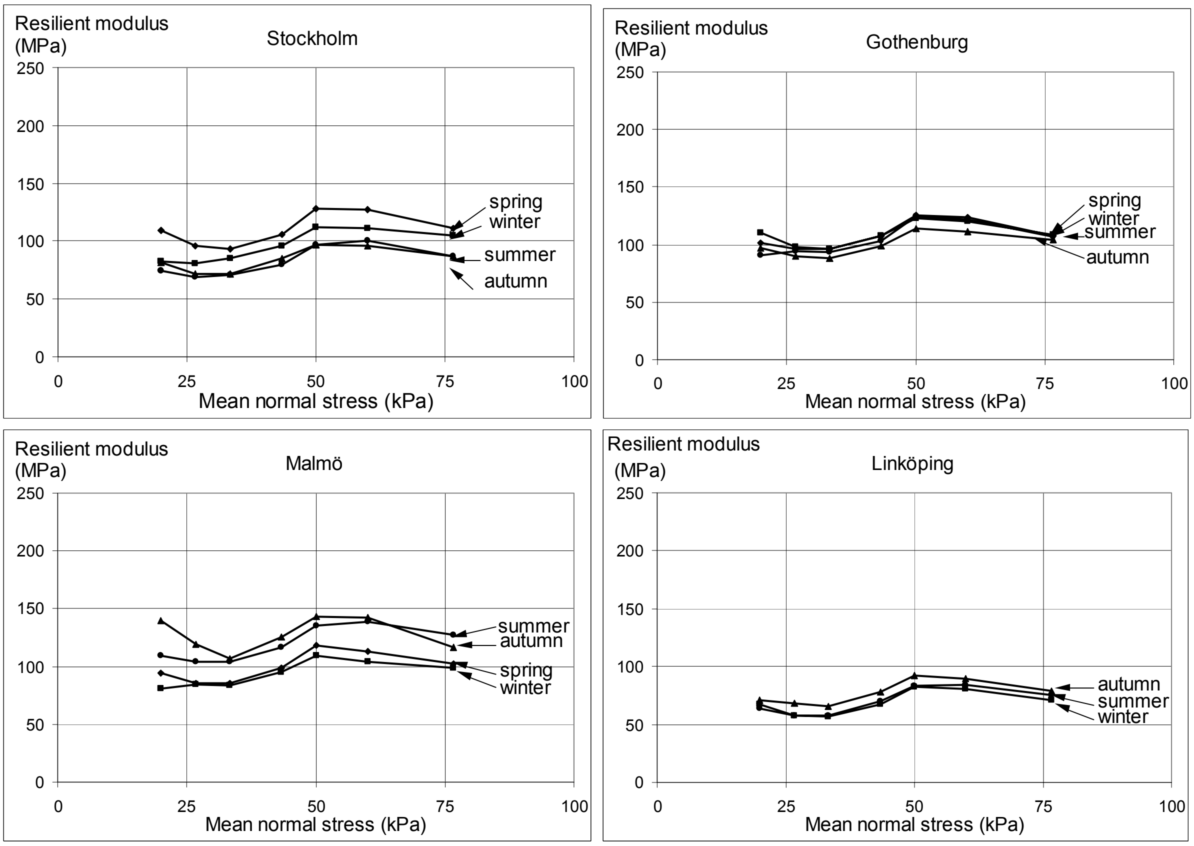


Figure 3. Stiffness expressed as resilient modulus for MSWI bottom ash from four plants. Specimens with 83–89% of maximum density and 100–108% of optimum water content obtained by modified proctor.

Figure 4 shows the results from all four incinerator plants in one stress condition. Both the seasonal variations and the spread among the specimens tested are illustrated for each material.

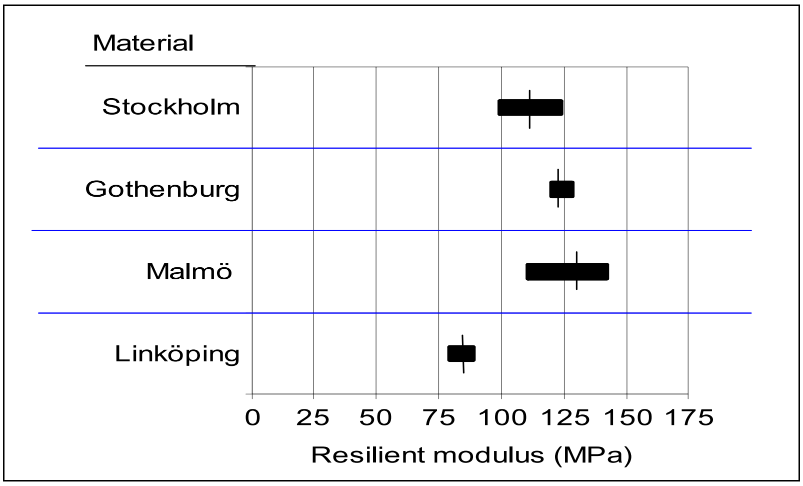
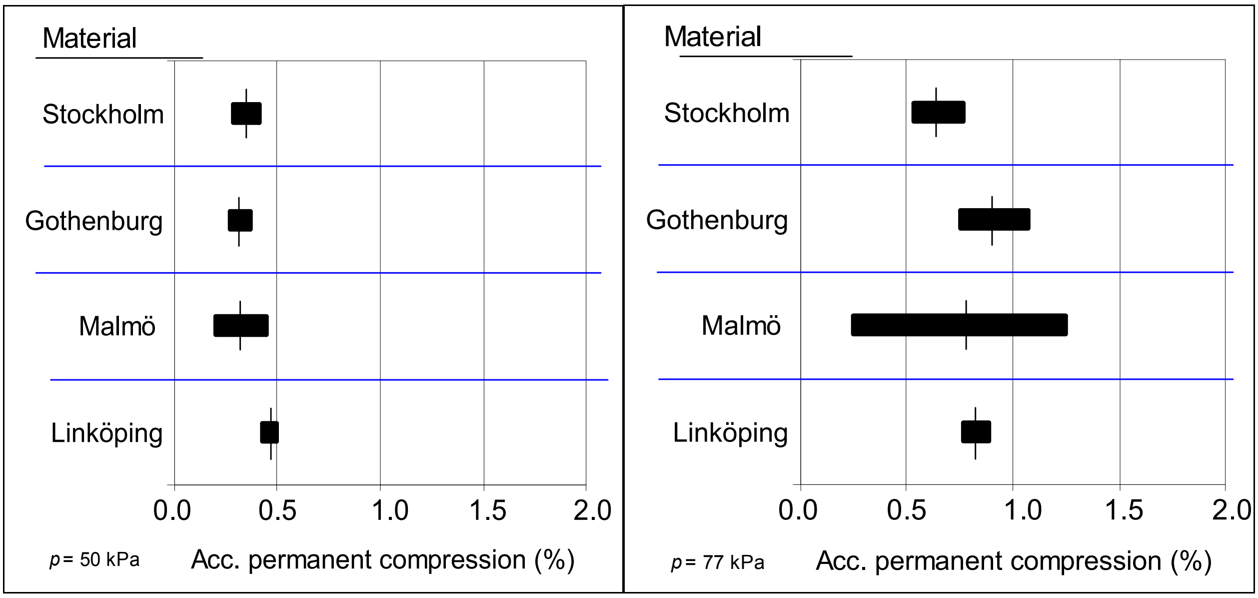


Figure 4. Resilient modulus in one stress condition. 95% confidence intervals. Specimens with 83–89% of maximum density and 100–108% of optimum water content. Mean normal stress p = 50 kPa, vertical stress/horizontal stress, σ_v/σ_h = 3.5.

In Figure 5, the stability, expressed as accumulated permanent axial compression at two mean normal stresses, 50 kPa and 77 kPa, is indicated.



Figures 5a and 5b. Accumulated permanent axial compression. 95% confidence intervals. Specimens with 83–89% of maximum density and 100–108% of optimum water content. p = 50 kPa (5a) and 77 kPa (5b). σ_v/σ_h = 3.5 (5a) and 7.5 (5b).

4 Discussion

The hypothesis was that there are significant differences in deformation properties between incinerator plants and between sampling periods.

The results show that time-dependent (seasonal) differences for resilient moduli of the bottom ash could be found within the different incinerator plants (Figure 3). However, the differences were not significant ($p < 0.05$) and the relative position of the curves was not general. When the ash from all four incinerator plants was compared (Figure 4), it was evident that one plant differed from the others. The resilient modulus for the Linköping material differed significantly ($p < 0.05$) from the other materials.

There could be several explanations for the differences in deformation properties. In a pavement layer, deformation properties depend to a large extent on how well compaction has succeeded and this in turn depends on the grain size distribution and the grain shape of the compacted material. The compaction tool and the water supply are also important parameters, but they are not material properties. Furthermore, if the deformation properties were to be retained the grains must be resistant to both mechanical and climate wear, but this is not discussed in this study. The organic matter content is a parameter that has a detrimental impact, but is seldom problematic for common Swedish road materials.

Here, the impact of grain size distribution and organic content on the test results obtained is discussed.

The *grain size distribution* has a major impact on deformation under loading for natural aggregates (Donbovand, 1987 quoted in Sweere, 1990; Thom, 1988; Kolisoja, 1997). However, the time-dependent variation in grain size distribution between the ashes studied was small (Figure 2). Even between the incinerator plants the difference was small. No significant difference ($p < 0.05$) in fines content, uniform coefficient c_u or d_{90} was found. Consequently, the grain size was not thought to be the main cause of the deformation variations among the ashes studied.

The *organic matter content* is generally agreed to have a detrimental impact on the stiffness of granular materials. Most countries have limited the organic content in road materials. According to the ANOVA evaluation, the bottom ash from Linköping had a significantly higher content of organic matter ($p < 0.05$) than the other ashes. The ash from this station also proved to have the lowest stiffness, expressed as resilient modulus. An attempt to relate the resilient modulus and the permanent deformation to the content of organic matter was made in Figure 6.

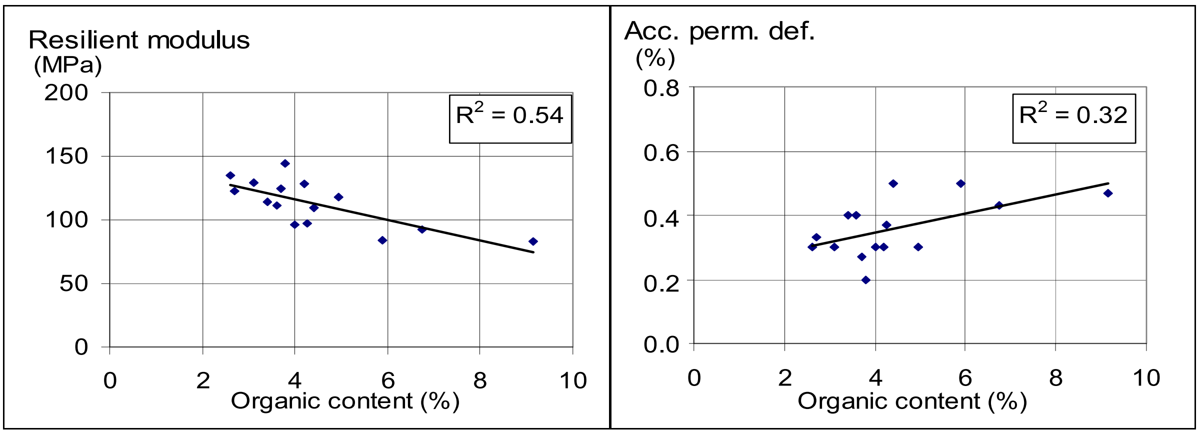


Figure 6. Resilient modulus and accumulated permanent compression related to organic content, measured as LOI at 550°C. Mean values for sample periods and incinerators. Results from triaxial tests with $p = 50$ kPa and $\sigma_v/\sigma_h = 3.5$.

An early Swedish study of fine-grained granite (Bäckman, 1989) showed that 6 wt.-% of organic matter in the material with a grain size of less than 2 mm, resulted in considerable deterioration in the E-modulus. The study in this paper also confirms this for MSWI bottom ash and suggests that even permanent deformation is influenced negatively.

The differences in organic content between ash materials could have several explanations. Both the raw material and the incineration control the organic content of bottom ash. The organic matter is mainly composed of non-combusted cellulose and lignin, which are found in large amounts in the original MSW as paper and woody or plant material (Pavasars, 2000). Municipal waste contains more organic material than industrial waste (Chandler et al., 1997). In this study the four incineration stations incinerate both municipal and industrial wastes. Stockholm and Linköping incinerated a higher proportion of municipal waste than Gothenburg and Malmö, approximately 80% compared with 60% (Table 1). This could perhaps explain the two main differences in material composition between the ashes, the Stockholm ash contained significantly more glass than the rest and the Linköping ash contained significantly more of the 'Other' category than the rest (Figure 1).

5 CONCLUSIONS

The results suggest that the deformation properties expressed as calculated resilient modulus and measured permanent deformation from cyclic load triaxial tests were reasonably uniform for each incinerator plant. There was a significant difference between the stations but not within a station ($p < 0.05$). The difference in resilient deformation properties between the stations is probably caused by different levels of organic content.

It was also shown (as expected) that the organic content has a limiting effect on the resilient modulus. For the material studied, the resilient modulus increased by 50% (from approximately 80 MPa to 120 MPa) when the content of organic matter was halved (from 8% to 4%). The technical usability of MSWI bottom ash thus increases significantly if the organic matter content can be kept low.

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REFERENCES

Arm, M., (1996) *Dynamic testing of fine-grained subgrade soils*. In: Correia (edited) Flexible Pavements. Balkema. pp 135–142.

Arm, M., (2003) *Mechanical properties of residues as unbound road materials – experimental tests on MSWI bottom ash, crushed concrete and blast furnace slag*. Dissertation. Royal Institute of Technology, Stockholm. (In print)

Bill 2000/01:130. (2001) *Svenska miljömål – delmål och åtgärdsstrategier*. Fritzes offentliga publikationer, Stockholm. (English short version *Swedish Environmental Objectives – Interim Targets and Action Strategies* available from the Swedish Environmental Objectives Council)

Bäckman, L., (1989) *Bestämning av organisk halt i grovkorniga material (bärlager)*. VTI notat V84. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Chandler, A.J., Eighmy, T.T., Hartlén, J., Helmar, O., Kosson, D.S., Sawell, S.E., van der Sloot, H.A. & Vehlow, J., (1997*) Municipal Solid Waste Incinerator Residues*. Studies in Environmental Science 67. The International Ash Working Group (IAWG), Elsevier.

Donbovand, J., (1987) *The results obtained from repeated load triaxial tests*. Internal Report. Pavement Research Group, University of Nottingham.

Fällman, A-M., (2000) *Significance in the results of total composition and potential leachability of screened MSWI BA from different plants and sampling periods*. In: Woolley, Goumans & Wainwright (edited) Waste Materials in Construction. Elsevier Science Ltd. pp 355–370.

Hartlén, J. & Elander, P., (1986) *Residues from waste incineration – chemical and physical properties*. SGI varia 172. Swedish Geotechnical Institute, Linköping.

Höbeda, P. & Bünsov, L., (1979) *Slagg från förbränning av hushållsavfall som vägmaterial*. VTI meddelande nr 180. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Höbeda, P., Jacobson, T. & Viman, L., (1985) *Undersökning av förbränningsrest från SYSAV, Malmö, genom laboratorie- och provvägsförsök*. VTI meddelande nr 413. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Jacobsson, T. & Viman, L., (1989) *Provvägsförsök Gärstad –87, Slagger från kol- och sopförbränning*, Lägesrapport 8901. VTI notat nr V80. Statens väg- och trafikinstitut, Linköping. (In Swedish)

Kolisoja, P., (1997) *Resilient Deformation Characteristics of Granular Materials*. Dissertation. Publication 223. Tampere University of Technology.

Paute, J-L., Dawson, A.R. & Galjaard, P.J., (1996) *Recommendations for repeated load triaxial test equipment and procedure for unbound granular materials*. In: Correia (edited) Flexible Pavements. Balkema. pp 23–34.

Pavasars, I., (2000) *Composition of Organic Matter in Bottom Ash from MSWI*. In: Woolley, Goumans & Wainwright (edited) Waste Materials in Construction. Elsevier Science Ltd. pp 241–246.

prEN 13286-7, April 2003. *Unbound and hydraulically bound mixtures. Part 7: Cyclic load triaxial test for unbound mixtures*. CEN.

prEN 13286-52, April 2003. *Unbound and hydraulically bound mixtures – Methods for making test specimens – Part 52: Making specimens by vibro-compression*. CEN.

RVF, (1997) *Statistik 1997*. The Swedish Association of Waste Management, RVF Service AB, Malmö. (In Swedish)

SFS 1999:673. *Lag om skatt på avfall (The Waste Tax)*. Ändrad genom SFS 1999:1321 och 2002:998. Fritzes, Stockholm. (In Swedish)

SNRA, (2001) *Provningsmetoder för alternativa material till vägunderbyggnad*. Publikation 2001:34. Vägverket (Swedish National Road Administration), Borlänge. (In Swedish)

Sweere, G.T.H., (1990) *Unbound granular bases for roads*. Dissertation. Delft University of Technology.

Thom, N.H., (1988) *Design of road foundations*. PhD thesis Department of Civil Engineering. University of Nottingham.

PAPER III

PERGAMON

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Self-cementing properties of crushed demolished concrete in unbound layers: results from triaxial tests and field tests

Maria Arm *

Swedish National Road and Transport Research Institute (VTI), SE-581 95 Linköping, Sweden

Abstract

A 2-year study is underway to evaluate the expected growth in stiffness in layers of crushed concrete from demolished structures. This growth is said to be a result of self-cementing properties. The study consists of repeated load triaxial tests on manufactured specimens after different storing time together with falling weight deflectometer, FWD, measurements on test sections. Results so far show a clear increase with time in resilient modulus and in back-calculated layer modulus for all concrete materials. The increase is the largest in the first months and then diminishes. The field measurements show a more considerable growth in stiffness than the laboratory tests, with a doubled value two years after construction. Comparative investigations on natural aggregates, mostly crushed granite do not show any growth in stiffness, neither in the laboratory nor in the field. Consequences for the choice of design modulus are discussed.

Keywords: Recycling; Reuse; Recycled aggregates; Demolition waste; Self-hardening; Stiffness increase; Strength; Design modulus; Resilient modulus; Sub-base; Base;

1. Introduction

Several research projects are ongoing in Sweden with a view to facilitate and increase the use of alternative materials in Swedish roads [1–5]. Both design and environmental guidelines are under development. This study deals with the use of crushed demolished concrete in unbound layers. According to experiences in other countries [6,7] a growth in stiffness, depending on self-cementing properties could be expected in layers with crushed concrete. The aim with this project was therefore to study the stiffness evaluation in unbound layers of crushed concrete in the laboratory and in the field. Later on, the results will help to establish design manuals where the specific properties of crushed concrete are utilized.

2. Methods

In the laboratory, the strength properties were studied using repeated load triaxial tests. Complementary properties such as composition, optimum water content and maximum dry density were also investigated. Specimens of crushed concrete from one source, a demolished industrial building in Grums, were manufactured and then tested after different storing periods. The specimens were prepared according to the following routine. First, the material was proportioned to a chosen grading, which was a well-graded curve in the centre of the approved zone for base course material in the Swedish guidelines, ROAD 94 [8], as shown in Fig. 1.

The water content chosen was 60% of optimum. Then, 300-mm high specimens with a diameter of 150 mm were compacted, in one layer by simultaneous vibration and compression in a Vibrocompresseur, to 97% degree of compaction. Finally, the specimens were wrapped in plastic foil and stored indoors. After a certain storing time (1, 7, 15, 28, 60, 180, 365 or 730 days) the specimens were exposed to repeated load in triaxial tests and the resilient moduli for different stress conditions were calculated. Comparison with corresponding properties of natural aggregates, such as crushed granite was made.

In the field, test sections with crushed concrete in the sub-base or base course have been constructed at different places in Sweden [9–12]. These are monitored by FWD measurements and for the purpose of this study the layer moduli have been evaluated by means of back-calculation. For comparison, values from refer-

* Tel.: +46-13-204277; fax: +46-13-141436.
E-mail address: maria.arm@vti.se (M. Arm).

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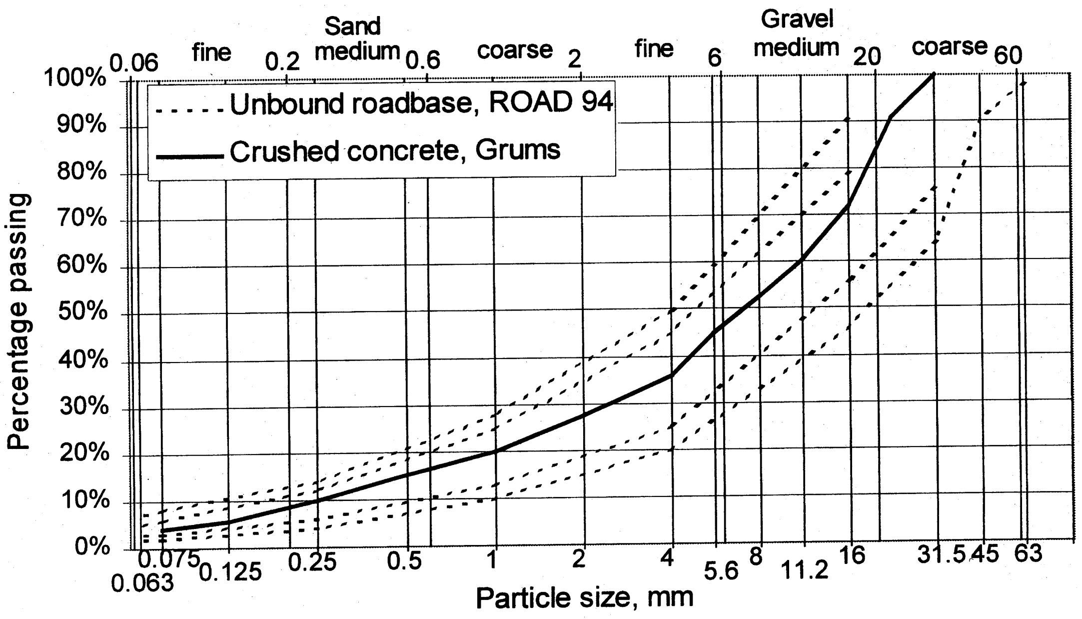


Fig. 1. Grain size distribution for material tested in repeated load triaxial tests.

ence sections at the same site with natural aggregates in the unbound layers have been used.

The tests are continuing and will be finished during this year.

3. Results

3.1. Laboratory tests

The material used in the laboratory tests was considered very pure, since more than 95% consisted of concrete (Fig. 2).

The compaction tests gave an optimum water content for the material of 9.4% and maximum dry density from both modified proctor and vibrator table of 2.0 t/m^3.

In Fig. 3, the results of the repeated load triaxial tests are plotted as the calculated resilient moduli at different stress conditions. Each point represents the mean value of three specimens. The results will be completed with data from 2-year-old specimens, but the tendency so far is a growth in resilient modulus with age.

Tests have also been performed on specimens that were stored in wet sand during the first 30 days, since previous Finnish field experiences have shown a positive effect of watering the surface of the concrete layer. Preliminary results, however, suggest a very small increase in resilient modulus for these specimens, which is the opposite of what was expected.

The permanent deformations have not been analysed yet.

3.1.1. Comparison with natural aggregates

The effect of storing time was studied in a small investigation on natural aggregates some years ago [13]. In that investigation, specimens of crushed and uncrushed granite were tested in repeated load triaxial tests after 1, 3, 7, 28 and 90 days and the resilient moduli were calculated. The results of those tests showed no increase in the resilient modulus, neither for the crushed material nor for the uncrushed material.

	weight-%
mortar + aggregate	43%
mortar	28%
aggregate	26%
brick	2%
other (bitumen, etc)	1%
wood etc.	0%
Total	100%

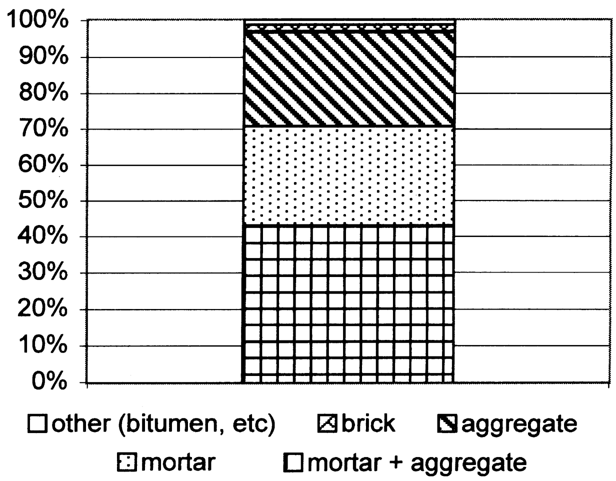


Fig. 2. Main constituents of crushed concrete from Grums, wt.% of washed material > 4 mm.

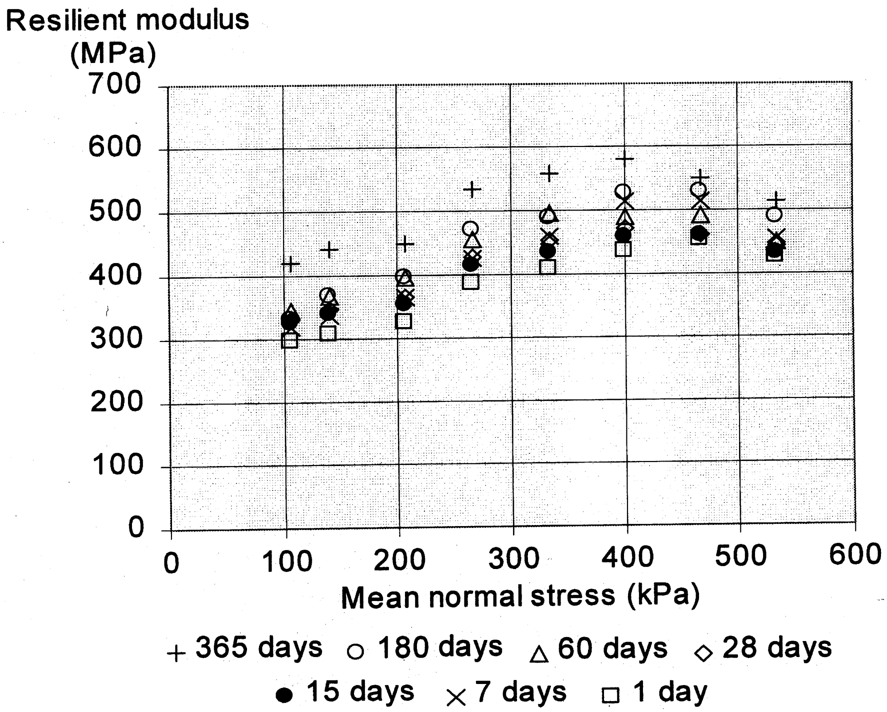


Fig. 3. Growth in resilient modulus for crushed concrete from Grums. Results from repeated load triaxial tests after different storing time.

Since all the testing parameters and even the grain size distribution were the same for the granite as for the concrete, it is interesting to compare the calculated resilient modulus. In Fig. 4, values for "fresh" and stored material are plotted and curve fits are made.

Two things can be observed. First, the modulus for concrete shows a clear growth at all stress levels, which the modulus for granite does not. Second, the concrete has a less stress dependent resilient modulus than the granite has, which means that at lower stress levels the modulus for the concrete is the highest, while at higher stress levels the modulus for granite is the highest.

3.2. Field tests

Results from FWD measurements at three different sites and at a different age after construction are shown in Fig. 5. In Västerås all the unbound layers consist of crushed concrete [10], while in Björsbyn [11] and in Ekeby it is only the sub-base that contains concrete. Furthermore, at all sites it is concrete from demolished buildings that has been used. It should be noticed that even though it is concrete from different sources, they all have very little impurities, in all cases less than 10%.

The results from the FWD measurements show a clear growth in layer moduli for the layers with crushed concrete. The growth is largest the first months and then diminishes. The modulus after 2 years is about twice as high as that is reached after one month.

However, the layer moduli for the unbound layers in the reference sections, do not increase at all or very little

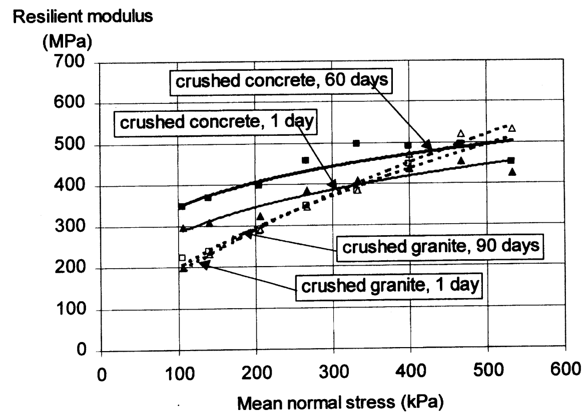


Fig. 4. Comparison of modulus change. Resilient modulus from repeated load triaxial tests on granite and concrete.

during the same period. This results in very big differences in layer modulus between concrete sections and reference sections after the first months.

4. Discussion

To make use of the self-cementing properties in the design process, there is a need of knowledge of the material stiffness, expressed as a modulus and its growth rate.

Experiences from this study as well as from other laboratory tests at VTI [9–12,14] show that crushed concrete from demolished structures has equal or higher stiffness than natural aggregates, expressed as calculated

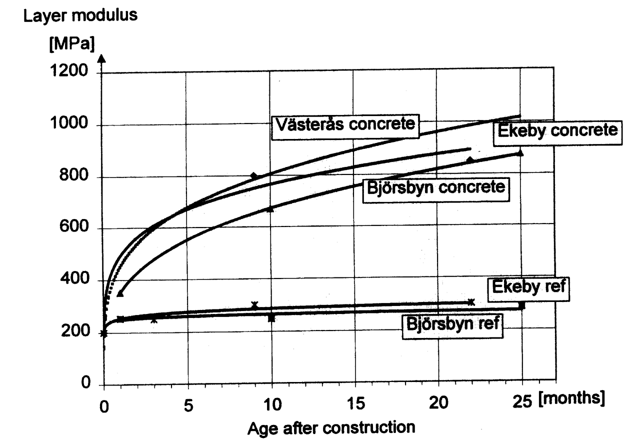


Fig. 5. Increase in stiffness. Back-calculated layer moduli for unbound layers of crushed concrete. From FWD measurements.

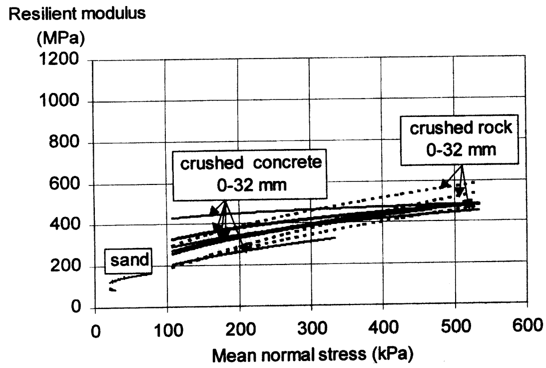


Fig. 6. Comparison of stiffness. Crushed concrete and crushed rock with different origin.

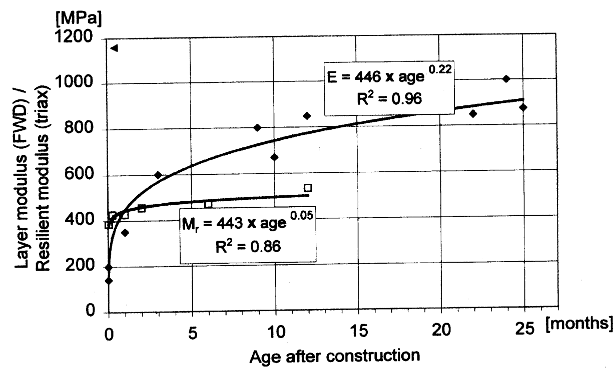


Fig. 7. Increase in stiffness. In the field: back-calculated layer moduli from FWD measurements at three sites. In the laboratory: resilient modulus for crushed concrete from Grums. Results from repeated load triaxial tests with vertical stress 400 kPa and horizontal stress 120 kPa. Mean value of three specimens.

resilient modulus from triaxial tests. (This is true for lower and medium high stress levels, that is, for stress levels relevant for a Swedish sub-base. The use of crushed concrete in a base course can also be suitable, but not in general, due to the fact that some of the tested concrete materials showed greater sensitivity to high stresses than the natural aggregate did. A fact that is significant in a construction with thin bound layers, where high traffic induced stresses could be expected in the base course.) In Fig. 6, the results from these triaxial tests, namely resilient moduli for a range of materials, are plotted and curve fits are made. The crushed concrete originates from different sources, such as demolished buildings, a demolished concrete road and railway sleepers. Granite, gneiss and limestone represent the crushed rock. For all materials the grain size distribution, compaction degree and relative water content is the same. The specimens have been tested without previous storing.

There is a certain range in the resilient modulus for different concrete types. However, this is also the fact for different rock types. A more detailed value of the modulus requires an investigation of the specific concrete material and knowledge of the construction that is planned. For example the thickness and type of bound layers have great impact on the stress conditions in the construction.

In this study, both laboratory and field results show a growth in stiffness with time, which is not present for natural aggregates. The growth is considerably larger in the field tests than in the laboratory test. The question is how much of the growth that can be taken into account in the design process.

In Fig. 7, curve fits have been made with equations on the form $M = A \times (\text{age in months})^B$.

The field tests gave B values of 0.19, 0.25 and 0.28 for separate sites and 0.22 for all sites together, while the laboratory tests gave the B value 0.05. If, for example, the design modulus should include the increase in stiffness during the first 2 months, this would give an increase of the start value with $(2\ \text{months})^{0.22} = 16\%$ and $(2\ \text{months})^{0.05} = 4\%$, respectively. Similarly, the modulus increases with 48 and 9%, respectively, if the value after 6 months is chosen.

However, if the design is made with a modulus value that is reached first after 6 months, there is a risk that the construction will deteriorate, being "under-designed" the first half year. This can probably be solved during the design or the planning phase.

The analyses and discussions on this subject are continuing and will hopefully help to form the basis for design manuals for constructions with crushed concrete in the unbound layers. A first step is taken by the guide for recycled crushed concrete as aggregate in roads, which will be published during winter 1999/2000 [15].

5. Conclusions

In this study, both laboratory and field results show a growth in stiffness for unbound layers with crushed concrete, which is not present for unbound layers with natural aggregates. This growth is considerably larger in the field tests than in the laboratory test. The growth is largest the first months and then diminishes. This means that the layer modulus 2 years after construction is about twice as high as that reached after 1 month.

Even though this is a limited study, some general conclusions can be drawn about the choice of design modulus for crushed concrete. For example, when crushed concrete with very little impurities is used as a Swedish sub-base material, at least the same design modulus can be used as for natural aggregates. However, a special investigation of the specific concrete material, together with knowledge of the planned construction, can give the opportunity to use a higher value on the modulus and by that benefit from the increasing stiffness caused by self-cementing.

Acknowledgements

In this research, several colleagues at the Swedish National Road and Transport Research Institute have taken part. The author wishes to express her appreciation for the co-operation, especially to Krister Ydrevik who planned and led the investigations, to Håkan Arvidsson who did most of the laboratory tests and to Håkan Carlsson and Anders Swenson who did the FWD measurements. The Swedish National Sand, Gravel and Crushed Stone Association (GMF) and the Development Fund of the Swedish Construction Industry (SBUF) provided funding of the study.

References

[1] Arell L. Sekundära ballastmaterial. Licentiate thesis, Stockholm: Royal Institute of Technology, 1997 (in Swedish with an English summary).
[2] Johansson HG. Återanvändning och återvinning av restprodukter, vägbyggnadsmaterial och sekundära, mineraliska material i vägar och gator. (KFB & VTI forskning/research 21). Linköping: Swedish National Road and Transport Research Institute, 1997 (in Swedish with an English summary).
[3] Betong i vägar — förstudie. Om möjligheterna att återvinna betong från husrivning. Karlskrona: Boverket 1998 (in Swedish).
[4] Andersson H, et al. Provningsmetoder för alternativa material till vägunderbyggnad — Undersökning av rostereldad kolbottenaska, slaggrus och krossad betong. Linköping: Swedish National Road and Transport Research Institute, 1999 (in Swedish).
[5] ALT-MAT. Alternative Materials in Road Construction. Interim report 18 May 1999. Project funded by European Commission 4th Framework Programme. Crowthorne: TRL, 1999.
[6] Sweere GTH. Unbound granular bases for roads. Dissertation, Delft University of Technology, Delft, 1990.
[7] Kivekäs L. Recycling of concrete and bricks. Oral contribution at Nordic seminar on sustainable use of materials in road constructions, Copenhagen, 11–12 October 1999.
[8] ROAD 94. Unbound pavement layers. In: General technical construction specifications for roads (Publ. 1994:86E). Borlänge: Swedish National Road Administration, 1994 (Chapter 5).
[9] Johansson HG, et al. Krossad betong — ett material för användning i vägar och gator (VTI notat no. 1-1996). Linköping: Swedish National Road and Transport Research Institute, 1996 (in Swedish).
[10] Ydrevik K. Krossad rivningsbetong som bär- och förstärkningslager i gatubyggnad. McDonalds hamburgerrestaurang i kvarteret Elstolpen i Västerås. (VTI notat no. 54-1996). Linköping: Swedish National Road and Transport Research Institute, 1996 (in Swedish).
[11] Jansson H, Ydrevik K. Utvärdering av fallviktsmätning våren 1998 på Björsbyvägen, provsträckor med krossad betong (VTI utlåtande no. 664). Linköping: Swedish National Road and Transport Research Institute, 1998 (in Swedish).
[12] Ydrevik K. Förstärkningslagermaterial av krossade betongslipers. Erfarenheter från provsträckor på väg 869 vid Stenstorp. Del 1: Byggnadsrapport (VTI notat no. 5-1999). Linköping: Swedish National Road and Transport Research Institute, 1999 (in Swedish).
[13] Arvidsson H. Dynamisk treaxiell provning för bedömning av lagringstiders inverkan på styvhets- och deformationsegenskaper hos bärlagergrus (VTI notat no. 10-1995). Linköping: Swedish National Road and Transport Research Institute, 1995 (in Swedish).
[14] Ydrevik K. Utlåtande beträffande krossad vägbetongs mekaniska egenskaper och lämplighet som användning i bär- och förstärkningslager (VTI utlåtande no. 644). Linköping: Swedish National Road and Transport Research Institute, 1999 (in Swedish).
[15] Ydrevik K. Återvägen. Råd och vägledning för återvinning av krossad betong som ballast till gator och vägar (VTI notat no. 67-1999). Linköping: Swedish National Road and Transport Research Institute, 1999 (in Swedish).

PAPER IV

Performance-Related Tests on Air-Cooled Blast-Furnace Slag and Crushed Concrete

VTI Activities in the European ALT-MAT Project

Maria Arm*, **Hans G. Johansson and Krister Ydrevik**
Swedish National Road and Transport Research Institute (VTI)

ABSTRACT

Several research projects are ongoing in Sweden with a view to facilitate and increase the use of alternative materials in Swedish roads. Among other things Sweden has taken part in the ALT-MAT project, which was funded by the European Commission. This article presents the work that was carried out at VTI.

Air-cooled blast furnace slag (AcBFS) and crushed concrete were studied and the performance of these materials was compared with the performance of crushed granitic rock. The activities involved both laboratory tests and field measurements. In the laboratory, repeated load triaxial tests were performed. Complementary analysis such as Los Angeles tests (LA), frost resistance tests and California Bearing Ratio (CBR) tests were also carried out. The field measurements included inspection of an existing road with AcBFS in the sub-base. Furthermore, test sections on two trial roads with crushed concrete in the sub-base were monitored and compared with reference sections with crushed rock. Inspection and monitoring were carried out by means of test pits and by measurements with falling weight deflectometer (FWD).

Both laboratory and field investigations showed smaller deformations in the AcBFS and crushed concrete layers than in the crushed rock layers. It also revealed an increase of stiffness, which did not occur in the crushed rock. This was caused by self-binding which from both engineering and environmental points of view is very attractive. However, both alternative materials had poorer results in the LA-test and other tests developed for single sized aggregates.

This leads to the following conclusions: The road application properties of air-cooled blast furnace slag and crushed concrete are as good as or even better than those of the crushed rock. The Los Angeles drum and the CBR test do not really justify the performance of these materials. Therefore, air-cooled blast furnace slag and crushed concrete should be analyzed using performance-related tests like repeated load triaxial tests in the laboratory and falling weight deflectometer measurements in the field. This statement holds for other alternative road materials as well.

INTRODUCTION

Several research projects are ongoing in Sweden with a view to facilitate and increase the use of alternative materials in Swedish roads. Both design and environmental guidelines are

* corresponding author

under development. Among other things Sweden has taken part in the ALT-MAT project, which was funded by the European Commission. The overall object of that project was to develop methods to assess the suitability of alternative materials for use in road construction. Furthermore, it should assess the suitability of the materials investigated. The work that was carried out at VTI included three case studies with both laboratory tests and field measurements on two selected materials – air-cooled blast furnace slag (AcBFS) and crushed concrete.

Swedish blast furnace slag derives from two major steel manufacturing plants. AcBFS from these plants has been used locally in road constructions in quite large quantities for many decades. The use of crushed concrete for this purpose is however relatively unexplored. Each year, building demolition operations generate about 1 million metric tons of waste. It is generally generated in a rubble form consisting mainly of concrete and brick. 75% of the debris is placed in landfills, 20% is reclaimed and 5% is too hazardous and has to be deposited.

In this study the performance of AcBFS and crushed concrete was compared with the performance of a reference material of crushed rock (granite). The Swedish Geotechnical Institute, SGI, analysed the total composition and performed leaching tests on the same materials. Their results are reported separately. [1]

METHODOLOGIES

The VTI activities involved laboratory tests and field measurements. In the laboratory, repeated load triaxial tests were performed. Complementary analysis such as Nordic Ball Mill tests (EN 1097-9), Los Angeles tests (EN 1097-2) and frost resistance tests according to a VTI method[2] (a modification of EN 1367-1) were also carried out. According to the standards, all these tests were performed on a sample of single-sized aggregate; the Nordic Ball Mill on 11.2–16 mm, the Los Angeles test on 10–14 mm and the frost resistance tests on 11.2–16 mm.

Furthermore, California Bearing Ratio tests (AASHTO T 193-81) were carried out. However some small changes in comparison with the standard test were done. The maximum particle size of the tested specimen was 31.5 mm. Test specimen was compacted at w_{opt} and it was tested directly after compaction without soaking and surcharge.

In Sweden, the Nordic Ball Mill test is used to assess abrasion caused by heavy vehicles during the construction period. The Los Angeles (LA) test and the California Bearing Ratio (CBR) test are not specified for Swedish road construction requirements. Although, they were used in this study in order to compare the results with what has been obtained in other countries and on various alternative road materials.

The field measurements included inspection of an existing road with AcBFS in the sub-base. It was partly constructed in 1986 and partly in 1996. Furthermore, test sections on two trial roads with crushed concrete in the sub-base were monitored and compared with reference sections with crushed rock. Inspection and monitoring were carried out by means of test pits and by measurements with Falling Weight Deflectometer (FWD). Each case studied is described in subsequent sections.

Repeated Load Triaxial Tests

In the repeated load triaxial tests, the material deformation under simulated traffic conditions is investigated. The resilient strain is used to calculate the stiffness expressed as resilient modulus and the accumulated permanent deformation can be used for classification purpose. Since the specimen exposed to loading consists of the whole composite material up to a certain grain size, it is in fact the function of the material that is tested. The method is well known both for fine-grained and coarse-grained materials. An American standard exists[3] and a European standard is under development[4]. The VTI tests were performed according to a method[5] similar to the American standard.

The tests were carried out on undrained specimens with the dimensions of 150 mm diameter and 300 mm height in VTI:s servo-hydraulic material testing system (VMS). This allowed material with maximum particle sizes up to ca 30 mm to be tested. The specimens were manufactured in one layer by means of vibrating compacting equipment called Vibrocompresseur in a special cylinder. This is a “friendly” method that was recommended in earlier experiments[6]. It was also chosen here because earlier experiments at VTI showed crushing tendency for porous materials. After compaction the specimens were pushed out of the cylinder and equipped with platens in both ends and a thin rubber membrane around.

The testing conditions that were chosen were ca 60% of optimum water content and ca 97% of maximum dry density from modified proctor. For each material investigated, three specimens were compacted and tested. The results that are discussed here are averages for these three specimens.

Investigations with the Falling Weight Deflectometer (FWD)

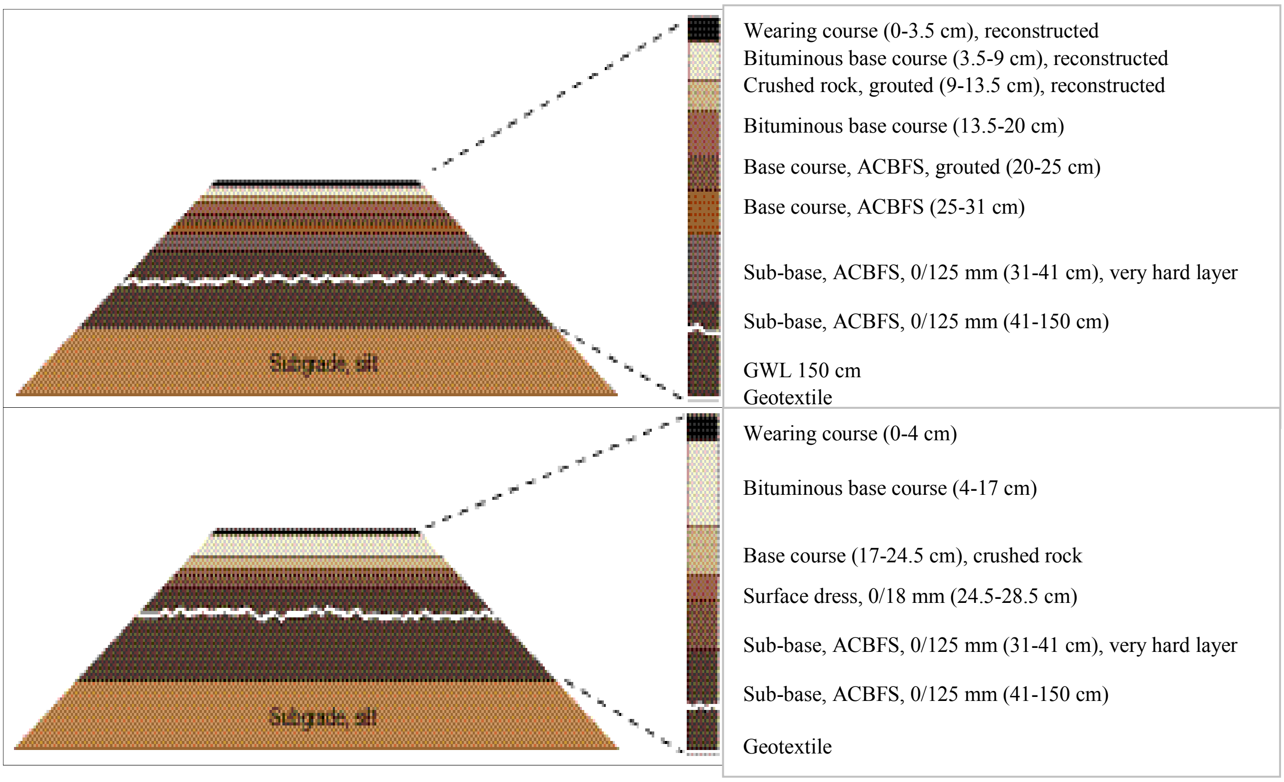
The FWD is an equipment which simulates the deflection of a road construction corresponding to the load produced by the wheel of a passing lorry. A falling weight impacts a circular plate with a specified diameter resting on the road surface. The deflection is measured by means of a number of seismometers, one placed in the centre of the loading plate and the others in straight line radial from it[7]. Data are collected on a disc for later calculation and estimation of the elastic modulus of the layers in the road construction.

The VTI measurements were carried out with a load of approximately 50 kN applied on a plate with a diameter of 300 mm. Six seismometers were used.

CASE 1: AIR-COOLED BLAST FURNACE SLAG (ACBFS)

In 1986, AcBFS was used in the sub-base in one part of road E4. The inspected site is situated in southern Sweden and was originally designed as a Motorway. In 1996, the road was reconstructed as a Highway with double lanes in each direction using the old sub-base in the two north running lanes and a new design in the opposite two lanes. One section from each was studied in the ALT-MAT project. Test pits were excavated and all layers were inspected. Figure 1a shows the pavement strata for the reconstructed northbound carriageway and figure 1b for the new southbound carriageway.

Figures 1a and 1b. Constructions with AcBFS according to test pit results.[8]



AcBFS Material Properties According to Field Measurements

In 1995, measurements with FWD were carried out on the old road section constructed with AcBFS as well as on a reference section with crushed granite in the sub-base. From the analysis and the back calculations a mean layer modulus of 600 MPa was obtained for the AcBFS sub-base to compare with 300 MPa for the crushed rock.

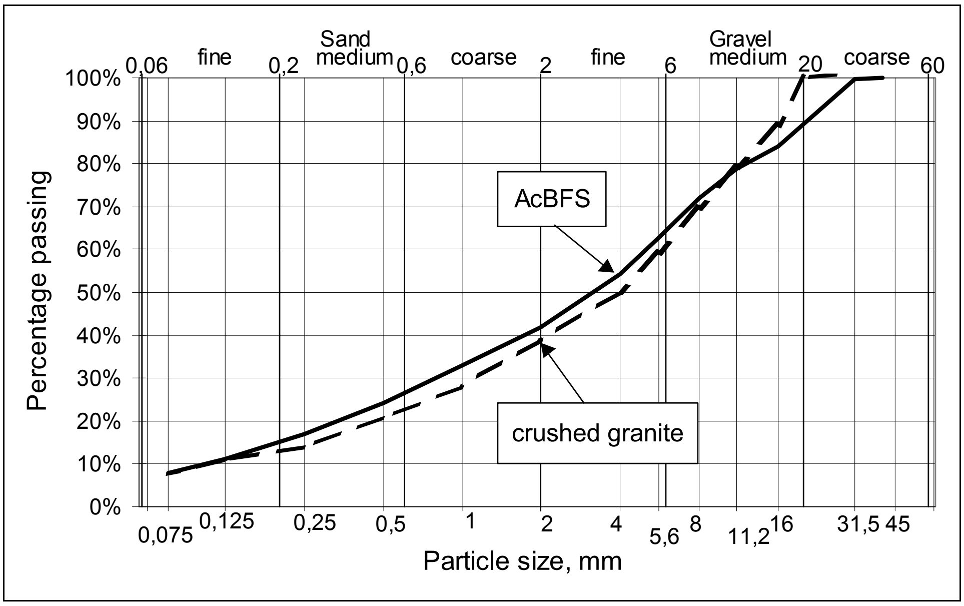
In 1998, the new Highway was measured and a layer modulus for the whole AcBFS layer was calculated. The AcBFS in Figure 1a obtained the modulus 1250 MPa and the AcBFS in Figure 1b obtained 750 MPa.

AcBFS Material Properties According to Laboratory Results

Materials from the test pits were analyzed in the laboratory. The Nordic Ball Mill value obtained for the AcBFS was 23 (mean value). This can be compared with the value for a reference material of crushed Swedish fine-grained granite, which is usually less than 18. The mean LA value was 35 to be compared with 20–25 for the reference material. Both results imply that the AcBFS is more sensitive to abrasion and crushing than the crushed granite is. The CBR mean value was 145; no CBR test was performed on the reference material.

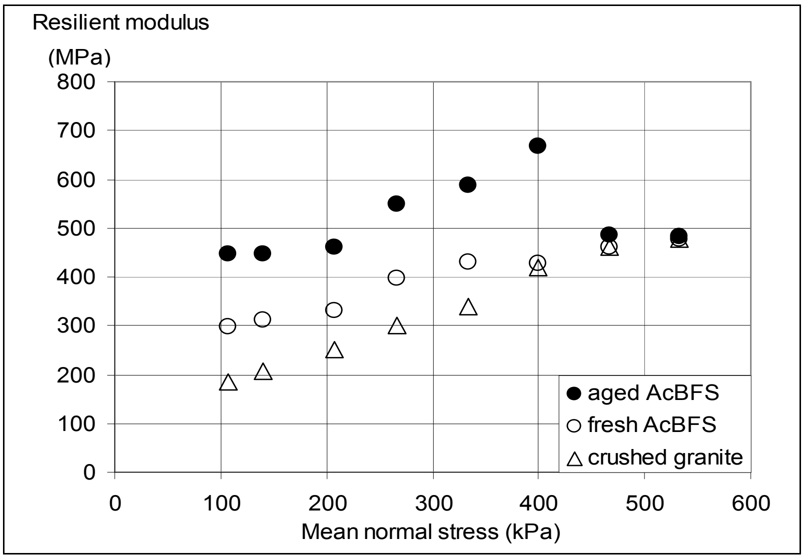
Repeated load triaxial tests were made on "fresh" AcBFS of the same type that was used in the road. Material with a grain size between 0 and 32 mm and a grading curve according to figure 2 was tested.

Figure 2. Particle size distribution for AcBFS and reference material tested in triaxial tests.



Previous experiences have shown that AcBFS has self-binding properties, therefore aged material was tested also. To simulate the ageing the AcBFS was stored in 50ºC during 28 days before testing. Results from the triaxial testing are shown in Figure 3.

Figure 3. Comparison of stiffness. Fresh and aged air-cooled blast furnace slag together with crushed granite. Stiffness expressed as resilient modulus from repeated load triaxial tests.



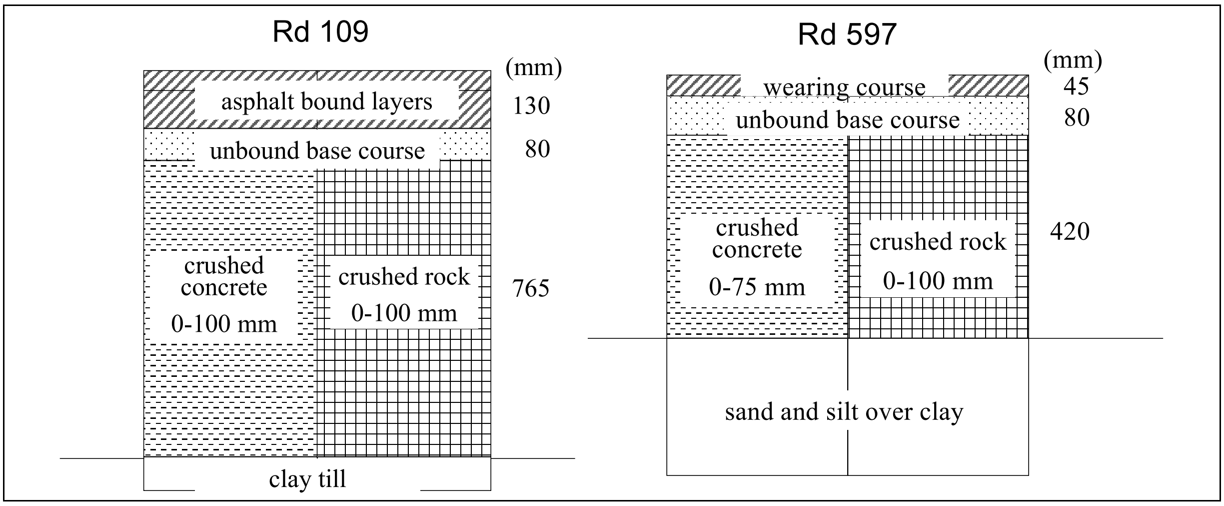
It can be seen that the stiffness of AcBFS increased after the ageing procedure. It increased in comparison to the fresh slag as well as to the reference material, which had a similar grading. The drop of the resilient modulus at mean normal stresses above 400 kPa is probably caused by deterioration of the particles. Stresses at this level will not occur in an ordinary Swedish base course or sub-base if the pavement is readily designed. During the construction period, however, high stresses could be expected.

Ageing the AcBFS also improved the permanent deformations. This fact coincides well with the results obtained in the FWD measurements described earlier.

CASE 2 AND 3: CRUSHED CONCRETE

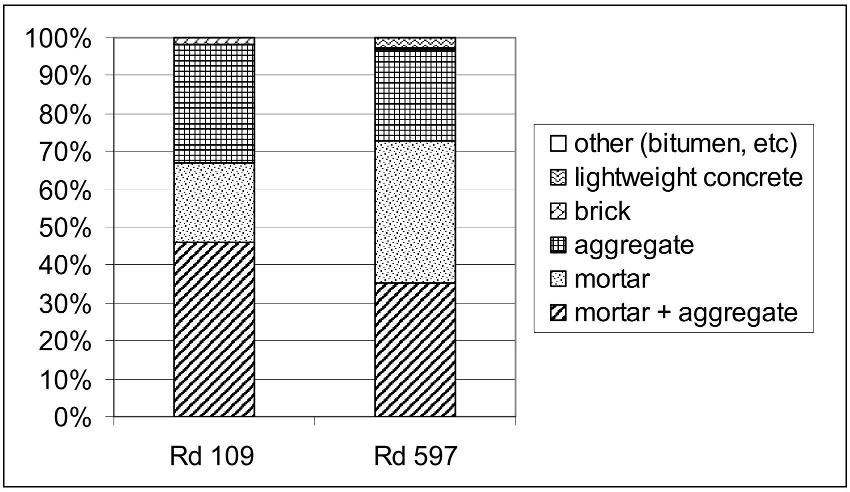
Two test roads, Rd 109 (south of Sweden) and Rd 597 (north of Sweden), with crushed concrete in the sub-base were investigated and monitored. In both cases comparison was made with reference sections with crushed granite. Figure 4 shows the pavement strata for the two test roads.

Figure 4. Constructions with crushed concrete in the sub-base. Reference sections with crushed granite.



Even if both concrete materials originated from demolished buildings they were regarded as crushed concrete according to their purity (Figure 5).

Figure 5. Content of various matter in crushed concrete used in Rd 109 and Rd 597. (% by mass).



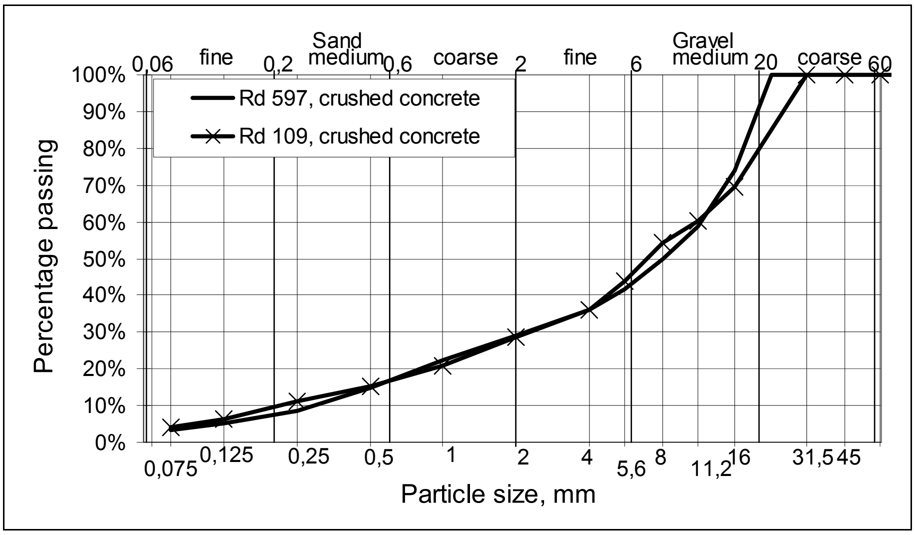
The Rd 109 material was very pure thanks to an extended processing including removing of reinforcing bars and extraneous material like wallpaper, wood etc. In Rd 597 also, reinforcing bars were removed, but the final material contained 2.7% by mass of lightweight concrete. What influence this might have is discussed later.

Crushed concrete properties according to laboratory analysis

Analysis with conventional laboratory methods gave the following results. Abrasion value obtained with the Nordic Ball Mill was 22–28, which is higher than for a reference material of crushed Swedish fine-grained granite (usually less than 18). The Rd 109 concrete obtained the LA value 33 and the CBR value 247. Frost resistance test on both concrete materials resulted in 10–17% by mass breakdown when tested in water and 33–43% breakdown when tested in NaCl.

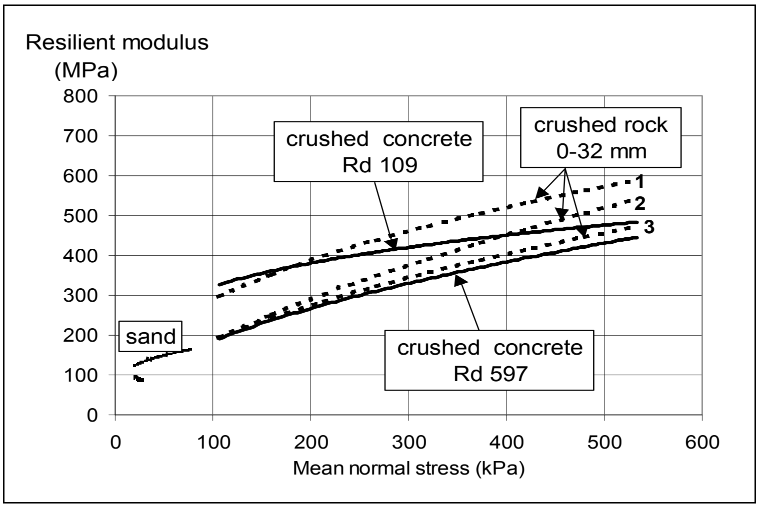
Repeated load triaxial tests were performed on the 0–32 fraction of the two concrete materials. Grading curves are shown in Figure 6.

Figure 6. Particle size distribution for concrete materials exposed to repeated load triaxial tests.



The specimens were tested at 96% of maximum dry density (determined with modified proctor), which varied between 1.8 and 2.0 t/m3. The water content was 65% of the optimum, which varied between 10 and 12%. Triaxial results are shown in Figure 7.

Figure 7. Comparison of stiffness for crushed concrete in Rd 109, Rd 597 and crushed rock with different origin (limestone 1, granite 2 and gneiss 3). Results from repeated load triaxial tests where all materials had the same particle size distribution.



The stiffness for the crushed concrete materials varies between 200 and 480 MPa depending on stress and material. Within the range of stresses in a Swedish sub-base, that is a mean normal stress of 100-200 kPa, the resilient modulus M_r varied between 200 and 400 MPa. The M_r is lower for the Rd 597 concrete than for the Rd 109 concrete and the different crushed rock materials as well. This can have many reasons. First, the maximum grain size was only 20 mm for the Rd 597 concrete compared with 32 mm for the other materials.

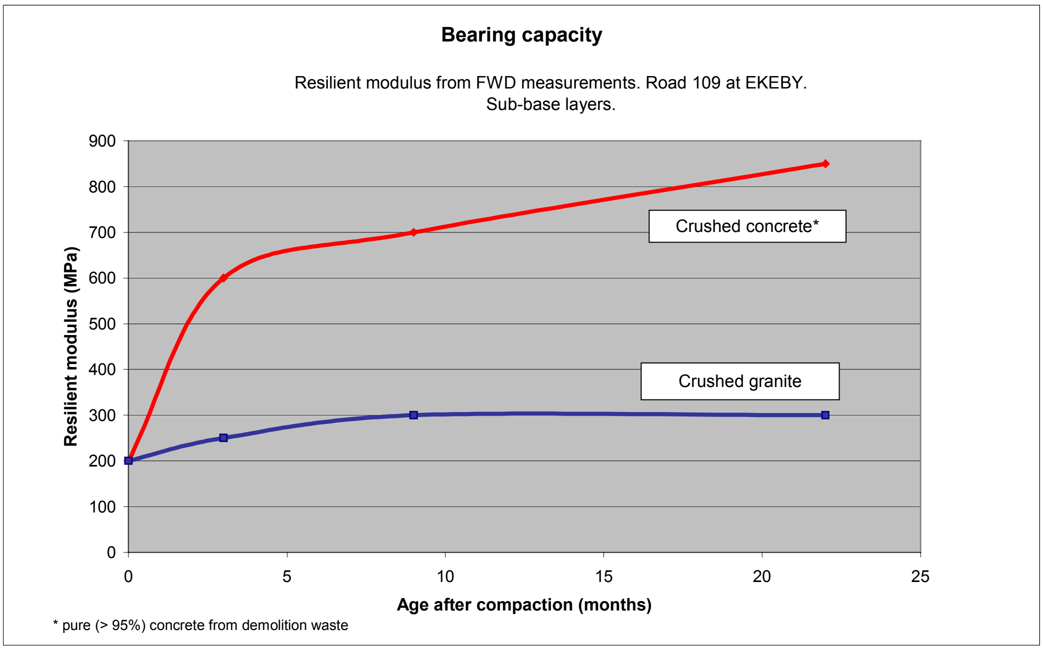
Earlier experiments have shown that the more coarse particles, the stiffer material. Second, as mentioned earlier the concrete in Rd 597 contained some quantity of lightweight material and this might have caused a lower M_r. Finally, the content of mortar was much higher in this material (38% by mass compared with 21%).

No triaxial tests were performed on stored material, but earlier tests on other crushed concrete materials and also results from other countries[9] have shown that an increase in stiffness by time can be expected. This was also evident from the field measurements.

Crushed Concrete Properties According to Field Measurements

This section is quoted from the national report[10]. "In order to determine whether there were any changes over time in stiffness of the materials in the sub-base on Rd 109, measurements with the FWD were performed on the carriageway after the road was completed. The measurements were repeated on two further occasions – three and nine months after completion of the road. After 3 months, the M_r of the sub-base had increased from 200 to 600 MPa; i.e. more than 300%. This increase can be compared with the reference section, where the increase of M_r was only about 25% for the same time (Figure 8).

Figure 8. M_r for the sub-base, Rd 109 (by K.Ydrevik, VTI).



The stiffness of the crushed concrete (CC) in the sub-base on Rd 597 has also increased with time. Even though the stiffness of the test sections was measured during the thawing period, 4 –6 months after completion of the road the M_r of the CC was found to be 370 MPa compared with the M_r of 170 MPa for crushed rock (CR) at the same occasion.

The increase of M_r in CC, which has been identified in both roads, is caused by two processes. Carbonization binds the free lime in the CC and hydration causes unhydrated cement to react with water. These processes probably took place quite quickly after construction. It has to be noted that the M_r of the CC increased without extra watering during laying! The self-cementing properties of the CC have also been verified by visual inspection and in samples taken from the sub-base."

DISCUSSION

Both laboratory and field investigations showed smaller deformations in the AcBFS and crushed concrete layers than in the crushed rock layers. It also revealed an increase of stiffness, which did not occur in the crushed rock. However, both alternative materials had poorer results in the LA test, the Ball Mill test and the CBR test than aggregates of a typical Swedish rock (granite).

Those methods that have been developed for testing resistance of conventional materials are often unsuitable for alternative materials, which obtain poor results depending on porous and weak particles. However, even though recycled products often form a lot of fine materials, the fines are not plastic as in the case of natural materials of poor resistance[11]. On the contrary it is these fines that gives the binding effect that leads to an increase of stiffness.

Furthermore, in all the conventional tests like frost resistance test, Ball Mill, Los Angeles and also micro-Deval test, a sample of single sized aggregate is investigated. The reason is that the tests are originally developed for aggregates in bituminous mixes that have "short fractions". Today, there is no standardized method that characterizes resistance to mechanical action in materials for unbound layers. In a layer of unbound particles the resistance is depending on how particles are in contact, for example how it is compacted which in its turn is a function of particle size distribution etc. Tests using gyratory compaction have given promising results, but need further development[8].

Nor are there any standard methods that measure the bearing capacity and give an input to design actions. The CBR method cannot differentiate between elastic and plastic deformation. Since the relation between the two types of deformations can differ between materials the CBR method is unsuitable for determining E-modulus[9]. However, the repeated load triaxial tests according to the existing American standard and the coming European standard will be very useful for this purpose.

CONCLUSIONS

- The road application properties of air-cooled blast furnace slag and crushed concrete are as good as or even better than those of natural aggregates.
- The LA-test and other tests developed for single sized aggregates do not really justify the performance of the materials studied.
- Air-cooled blast furnace slag and crushed concrete as well as other alternative road materials should be analyzed using repeated load triaxial tests in the laboratory and

FWD measurements in the field. Both of which taking in account the whole composite material or layer. In reality, this statement holds for all unbound materials.

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Corresponding author is Maria Arm, former researcher at the Swedish National Road and Transport Research Institute, SE-581 95 Linköping, Sweden. From 2002-03-01 researcher at the Swedish Geotechnical Institute, SE-581 93 Linköping, Sweden;
e-mail: maria.arm@swedgeo.se

REFERENCES

1. Håkansson, K.; Nilsson, U. *National Report for Leaching Tests for Sweden*; Report No. ALT-MAT WP5.SGI.007; Swedish Geotechnical Institute (SGI): Linköping, Sweden, 1999.

2. Höbeda, P.; Jacobson, T. *Frystöväxlingsförsök på stenmaterial med svaga lösningar av halkbekämpningsmedel*; VTI meddelande No. 244; Swedish Road and Traffic Research Institute: Linköping, Sweden, 1981. (in Swedish).

3. Alavi, S.; Merport, T.; Wilson, T.; Groeger, J.; López, A. *LTPP Materials Characterization Program: Resilient Modulus of Unbound Materials (LTPP Protocol P46) Laboratory Startup and Quality Control Procedure*; Publication No. FHWA-RD-96-176; Federal Highway Administration: Georgetown Pike, VA, 1997.

4. *prEN 13286-7, Unbound and Hydraulic Bound Mixtures. Part 7: Cyclic Load Triaxial Tests for Unbound Mixtures*; European Committee for Standardization.

5. Ydrevik, K. *Dynamic Triaxial Testing of Roadbase and Lower Sub-Base Materials;* VTI notat No. 4-1996. Swedish Road and Transport Research Institute: Linköping, Sweden, 1996.

6. Galjaard, P.J.; Paute, J-L.; Dawson, A.R. Comparison and Performance of Repeated Load Triaxial Test Equipment for Unbound Granular Materials. In *Flexible pavements*; Correia, A.G., Ed.; 1996; pp 7–21.

7. *New Road Monitoring Equipment and Methods*. Transport Research. COST 325. 1997.

8. *ALT-MAT Final Report.* Deliverable D7, Version 2.0, 08/01/01; Transport Research Laboratory (TRL): Crowthorne, U.K., 2001.

9. Sweere, G.T.H. *Unbound Granular Bases for Roads*. Dissertation, Delft University of Technology: Delft, 1990.

10. Johansson, H.G. *National Report for Mechanical Laboratory Analysis and Field Tests of Air-Cooled Blast Furnace Slag (AcBFS) and Crushed Concrete (CC*); ALT-MAT Report No WP6.01; Swedish National Road and Transport Research Institute (VTI): Linköping, Sweden, 1999.

11. Höbeda, P.; Chytla, J. *Undersökning av finmaterialkvalitet hos vägmaterial i synnerhet hos obundna material.* VTI notat No. 60 1999; Swedish Road and Transport Research Institute, Linköping, Sweden, 1999. (in Swedish).

PAPER V

ASSESSMENT OF THE DEFORMATION BEHAVIOUR OF ALTERNATIVE UNBOUND ROAD MATERIALS by means of results from cyclic load triaxial tests

Maria Arm
Swedish Geotechnical Institute (SGI), Linköping, Sweden
Krister Ydrevik
Swedish National Road Administration (SNRA), Linköping, Sweden

ABSTRACT

By tradition, only the resilient properties are expressed in the present Swedish design method. It is assumed that rutting is prevented through material, compaction and drainage specifications. However, for both 'good' and 'poor' aggregate materials to be used correctly, the pavement should be designed according to the potential and limitations of the materials used. The idea expressed in this paper is a small step on the way to performance-based specifications and design. The paper presents a proposal for material assessment based on permissible load, evaluated from permanent deformations in cyclic load triaxial tests. The serviceability of an alternative road material is assessed by comparing its deformation properties to the properties of the material normally used in the pavement layer in question. In the event that the properties are too different, it is recommended that an alternative pavement should be designed. This paper proposes methodology for this design, where the permissible load is not exceeded for the materials included.

Key Words: unbound materials; deformation properties; cyclic load triaxial test; pavement design

1 INTRODUCTION

For alternative aggregate materials, such as recycled aggregates and residues, to make a breakthrough as unbound road materials, they must be included in the national guidelines for road constructions, for example the Swedish ATB VÄG (SNRA, 2003).

Today, the road construction industry has insufficient guidelines on how and where residues can be used. According to the present specification, the contractor must show that the proposed residue performs as well as the conventional aggregate that it replaces. Several indirect parameters are used to indicate bearing capacity, such as particle size distribution, the amount of crushed particles and the organic matter content. These parameters, as well as the limit values, are based on experience from conventional aggregates only. New, performance-based test methods and corresponding design guidelines are therefore required to make fair and relevant comparisons between conventional and alternative aggregate materials.

Alternative aggregate materials cannot at all times replace conventional aggregate materials in all respects. In order for the alternative materials to still be used nevertheless, it is sometimes necessary to employ a different design that takes the properties of these materials into account. This may also be relevant for conventional materials that do not meet the normal specifications. Even in this case a general method that assesses how well the material will perform in the road is desirable.

This paper aims at describing a methodology for assessing the serviceability of alternative unbound road materials on the basis of their deformation properties. The work was commissioned by the Swedish National Road Administration (SNRA) and should result in an interim solution that is practical and capable of being implemented in the ATB VÄG in a short time. The work should use existing data from material already tested and the result should be usable for both alternative materials and conventional materials that do not meet the specifications.

2 Present material classification system

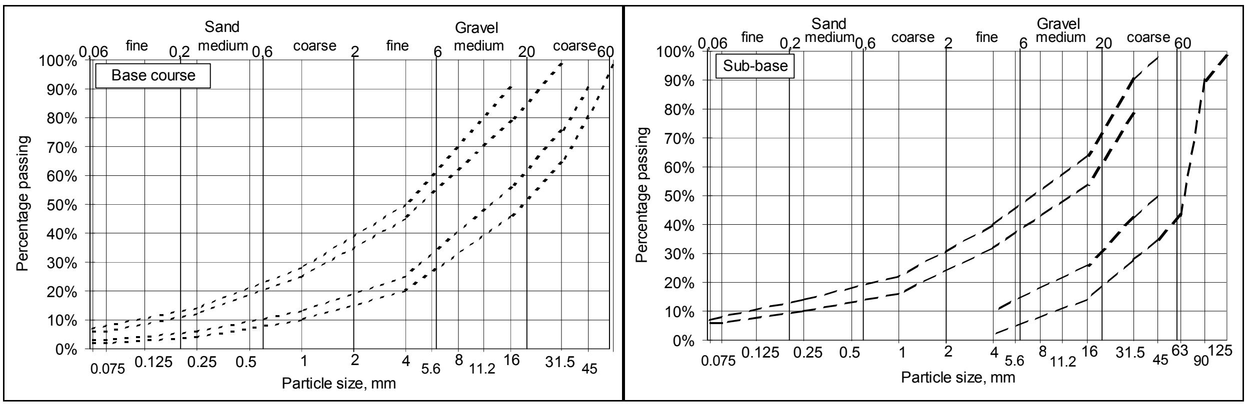
In the Swedish specifications for road constructions, subgrade soils are by tradition classified indirectly according to their particle size distribution (grading). In a similar way, unbound granular materials for use in the pavement are divided into base course material, sub-base material or capping layer material and are classified indirectly according to their grading and their resistance to wear and disintegration. For each of these material categories special requirements have been specified.

Table 1 and Figures 1a and 1b include a summary of the present requirements for unbound materials in paved roads with a flexible construction according to ATB VÄG.

Table 1. Present requirements for unbound materials in paved roads with a flexible construction according to ATB VÄG (after SNRA, 2003).

Use	Property or method	Limit value
Base course	• micro-Deval value / Ball mill value[1] • Organic matter content[2,3] • Particle size distribution • Amount of uncrushed particles[2] • Maximum particle size	– If used by construction traffic: max. 17 / max. 23, otherwise: max. 30 / max. 37. – Max. 2 wt.-% of fraction <2 mm. – According to Figure 1a. – <30 wt.-% of material >16 mm. – Depends on layer thickness.
Sub-base of crushed material[4]	• micro-Deval value/ Ball mill value[1] • Organic matter content[2,3] • Particle size distribution • Amount of uncrushed particles[2] • Maximum particle size	– Max. 30/max. 37. Recommendation: If used by construction traffic, max. 17/max. 23. – Max. 2 wt.-% of fraction <2 mm. – According to Figure 1b. – <30 wt.-% of material >16 mm. – Depends on layer thickness.
Capping layer	• Organic matter content[3] • Fines content / capillarity[1]	– Max. 2 wt.-% of fraction <2 mm – Max. 11 wt.-% / max. 1 m
Unbound layer	• Thermal conductivity	– If 0–25 cm from road surface: >0.6 W/(m · K). If 26–50 cm below surface: >0.3 W/(m · K).

[1]Alternative method. [2]If material other than crushed rock. [3]Through colorimetric measurement. [4]For use as a sub-base of uncrushed material the requirements on the amount of uncrushed particles do not apply.



Figures 1a and 1b. Outer and inner limit curves for particle size distribution of base course and sub-base materials in paved roads (after SNRA, 2003).

There are also additional requirements to Figure 1 that prohibit gap-graded material and material with too high a clay content from being used. The organic matter content requirement is intended for all pavement materials and fills situated within 1 m of the formation level. If cement-bound layers are used in the pavement, the limit applies within 2 m of the formation level.

When alternative materials, such as residues and recycled aggregates, are introduced they generally fail to meet these requirements. For crushed concrete and air-cooled blast furnace slag (AcBFS) it is mostly the micro-Deval/Ball mill values that are not met. For ash materials, such as municipal solid waste incinerator (MSWI) bottom ash, neither the grading nor the resistance values could be achieved. Nevertheless, case studies show that these alternative materials perform as well as natural aggregates when used in unbound road layers (TRL, 2000).

3 TEST METHODS FOR MEASURING DEFORMATION PROPERTIES

A large number of methods have been developed for measuring the deformation properties of a material under load.

A well-known laboratory method is the *California Bearing Ratio* method, CBR, which is widely used in many countries but is not applied in Sweden. Here, soil materials are by tradition classified indirectly according to particle size distribution. In line with the introduction of analytical design in the United Kingdom and the USA for instance, empirical relationships between the CBR value and elasticity modulus (E-modulus) have been established. The relationships were necessary as the design systems require an E-modulus as input. However, the CBR method cannot distinguish between elastic and plastic deformation. Since the ratio of the two types of deformations may differ from one material to another this means the CBR method is unsuitable for the determination of a purely elastic parameter such as an E-modulus (Sweere, 1990).

The indirect test methods, *Sand equivalent test* (EN 933-8) and *Methylene blue test* (EN 933-9), are intended for clayey materials and are therefore unsuitable for residues such as MSWI bottom ash, crushed concrete and blast furnace slag (Schmith, 1984; Pihl & Milvang-Jensen, 1996).

There are thus no European standard methods that measure the bearing capacity, providing input for the design process. However, the cyclic load triaxial test according to the future European standard (prEN 13286-7) will be very useful for this purpose.

It has been shown that the resilient modulus, M_r, for both granular materials and cohesive materials are stress-dependent, but in different ways (Sweere, 1990; Arm et al., 1995; Ydrevik, 1996). The M_r of granular material increases with increasing vertical stress, whereas it decreases for a cohesive soil. For the so-called intermediate type of soil, silt for instance, the M_r is almost stress-independent. Thus, for a material with a stress-dependent M_r it is important to relate a given stiffness or M_r to a certain stress condition. In the case of the design of a road structure it is generally agreed that the triaxial method is suitable for the determination of stress-strain behaviour of unbound granular materials and soils.

Cyclic load triaxial tests

The cyclic load triaxial test is a laboratory method that is being used increasingly, especially in the context of performance testing of unbound aggregates. It is based on the principle that the entire material, up to a certain particle size, e.g. 64 mm, is exposed to a simulated traffic load, whereupon the deformation is recorded. Cylindrical specimens are compacted and exposed to repeated loads of different magnitudes. The stiffness or load-spreading ability can be evaluated from the resilient strain and the result is expressed as a resilient modulus in different stress states. The permanent deformation appearing in the material at different loads can also be evaluated and used as a stability measurement.

There are several advantages with the cyclic load triaxial test compared with other laboratory tests measuring deformation properties: It exposes the material to a simulated traffic load. The measured resilient strain could be used for calculation of the M_r, which can be used as input in analytical design. The accumulated permanent deformation could be used for classification and ranking. The specimen that is exposed to loading is composed of the entire material, up to a certain particle size. It could therefore be said that it is the performance of the material that is tested, which is useful when new or unknown materials should be classified. The method is well-known for both fine-grained and coarse-grained materials and a CEN standard is under development. The drawback with the cyclic load triaxial test is that the test equipment is relatively expensive and consequently there are only a few in Europe. In the USA and Australia, however, they have invested in the method and there is equipment at several laboratories. These are used for routine testing, whereas the research institutes and the universities have the equipment for research purposes (Alavi et al., 1997; AS, 1995).

The European pre-standard, which will be the subject of a formal vote within the CEN, has been prepared by representatives from several European research laboratories with experience from cyclic load triaxial tests on coarse-grained unbound granular materials. These are Tampere University of Technology, SINTEF in Trondheim, University of Dresden, University of Nottingham, Laboratoire Centrale des Ponts et Chaussées (LCPC) in Nantes, Laboratório Nacional de Engenharia Civil (LNEC) in Lisbon and Swedish National Road and Transport Research Institute (VTI). The Royal Institute of Technology (KTH) in Stockholm, the Geotechnical Institute of Norway (NGI) in Oslo and Delft University of Technology also possess cyclic load triaxial test equipment and have performed several tests on coarse-grained unbound granular materials.

The cyclic load triaxial test has been used at VTI for comparison of granular materials for many years (Arm et al., 1995; Ydrevik, 1996; Arm, 1996a, 1996b, 1997, 1998, 2000a). Krister Ydrevik described the first VTI tests and results on coarse granular materials in 1993; both at seminars in Sweden and at workshops in the European network Pavement Foundation Group, PAFOG. Furthermore, co-operation has taken place in the Nordic countries, especially between VTI and Tampere University of Technology (Dr Pauli Kolisoja) when VTI started the tests on coarse-grained materials. In addition, advice was given and experience was transferred from VTI to SINTEF in Trondheim, which was also developing equipment for cyclic load triaxial tests on coarse-grained, unbound material. In recent years, Denmark and Iceland have followed.

The method described in this paper is a result of the experiences gained during the first ten years of testing at VTI. The research has been performed as commissioned research for SNRA. The results have therefore only been reported in Swedish to the customer and have not been published in English.

The European draft pre-standard describes different loading procedures. The procedure used at VTI is most similar to the so-called multi-stage procedure, where the same specimen is exposed to several stress levels (Ydrevik, 1996). The tests are performed using constant confining pressure (CCP), which means that a cyclic axial (vertical) load is applied to a cylindrical specimen under static radial (horizontal) pressure. Vertical and horizontal loads represent the imposed traffic load and overburden pressures from overlying pavement layers.

In the tests on which this paper is based, the specimen diameter was 150 mm and the height was 300 mm (Figure 2).

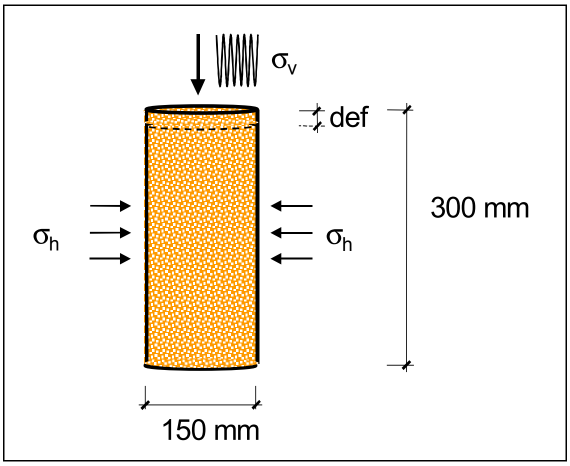


Figure 2.   Principle of cyclic load triaxial tests used in the study.

Every specimen was exposed to several consequential loading sequences. Two types of tests were used, the 'capping-layer test' and the 'base course test'. In the first test, the vertical cyclic stress, $\sigma_{v,c}$, was varied between 10 and 150 kPa and the horizontal stress, σ_h, was kept constant at 10 or 20 kPa, resulting in a deviator stress, q, of between 30 and 170 kPa. In the second test, $\sigma_{v,c}$ was varied between 120 and 1,220 kPa and σ_h was kept constant at 60 or 120 kPa, resulting in a q between 140 and 1,240 kPa (Ydrevik, 1996).

Each test was continued until all the sequences were completed or until the permanent deformation registered at one single sequence exceeded 20 mm. In such a case the specimen was considered as failed and the test was stopped automatically.

4 SUGGESTED METHODOLOGY FOR MATERIAL ASSESSMENT

The methodology proposed in Section 4.2 assesses the serviceability of an alternative unbound road material by comparing its deformation properties, evaluated from cyclic load triaxial tests, with the properties of the material normally used in the pavement layer in question. Section 4.3 describes methodology for designing an alternative pavement where the permissible load is not exceeded for the materials included.

The deformation properties are determined in a series of cyclic load triaxial tests in the laboratory during which the state of stress and the water content are varied. Testing conditions are chosen in order to simulate present or future field conditions. The results are expressed as a permissible vertical stress that could be translated into a certain placement depth below the road surface.

4.1 Assumptions

A range of assumptions and simplifications were made for this work: Parameters influencing the permanent deformation in the unbound layers were assumed to be load, water content and material properties. The load was expressed as vertical and horizontal stress; in the first step only as vertical stress. In the simplest form it could be stated as the thickness of a bound surface layer. The water content was expressed as relative water content (related to the optimum achieved in the modified proctor test) and could in the simplest form be stated as being the 'degree of drainage', a term that is used in ATB VÄG. Material properties were expressed as permissible load or bearing capacity and could in the simplest form be stated as being 'Material type' using particle size distribution and particle shape. The term permissible load or bearing capacity was defined as the maximum stress a layer of material can support without undergoing unacceptable permanent deformations. The permissible load for a material must thus be related to a definite limit value of permanent deformation in the layer. This limit value could be related to the road standard or road category chosen.

4.2 Methodology for comparison purposes

This methodology is based on cyclic load triaxial tests used to compare material performances and to show the similarity in behaviour according to the following.

- Specimens are compacted at three different water contents: 60, 90 and 110% of optimum water content, w_{opt}. They are compacted to a density that is 90% of the maximum achieved in the modified proctor test. At least two specimens on each level are compacted.
- Appropriate stress levels are chosen ('base course test' or 'capping-layer test') depending on the planned use and thereby depending on the thickness and resilient modulus, M_r, of materials overlying the material in question.
- The tests are performed.
- The results are presented in graphs (M_r as a function of mean normal stress, accumulated axial permanent strain as a function of number of loads) (Figures 3 and 4).
- The results are compared to the corresponding results for a well-known material (the one that is normally used in the layer in question in this pavement). Interesting parameters to study are resilient modulus and maximum permanent strain at relevant stresses. Relevant stresses depend on the planned use of the material. However, attention should be paid to the behaviour at even higher stress levels since this could offer guidance to possible degradation tendency, e.g. during construction.

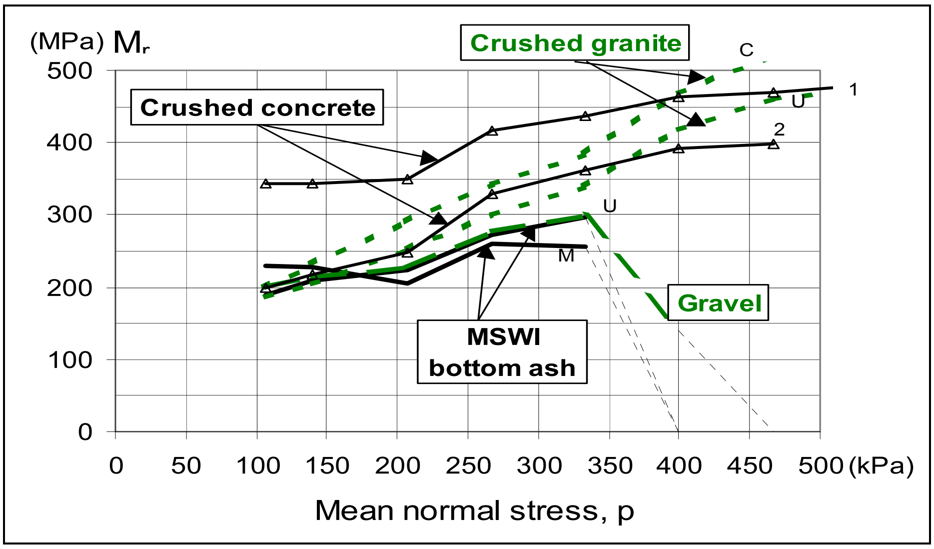


Figure 3. Examples of resilient modulus, M_r, for some materials as a function of mean normal stress.

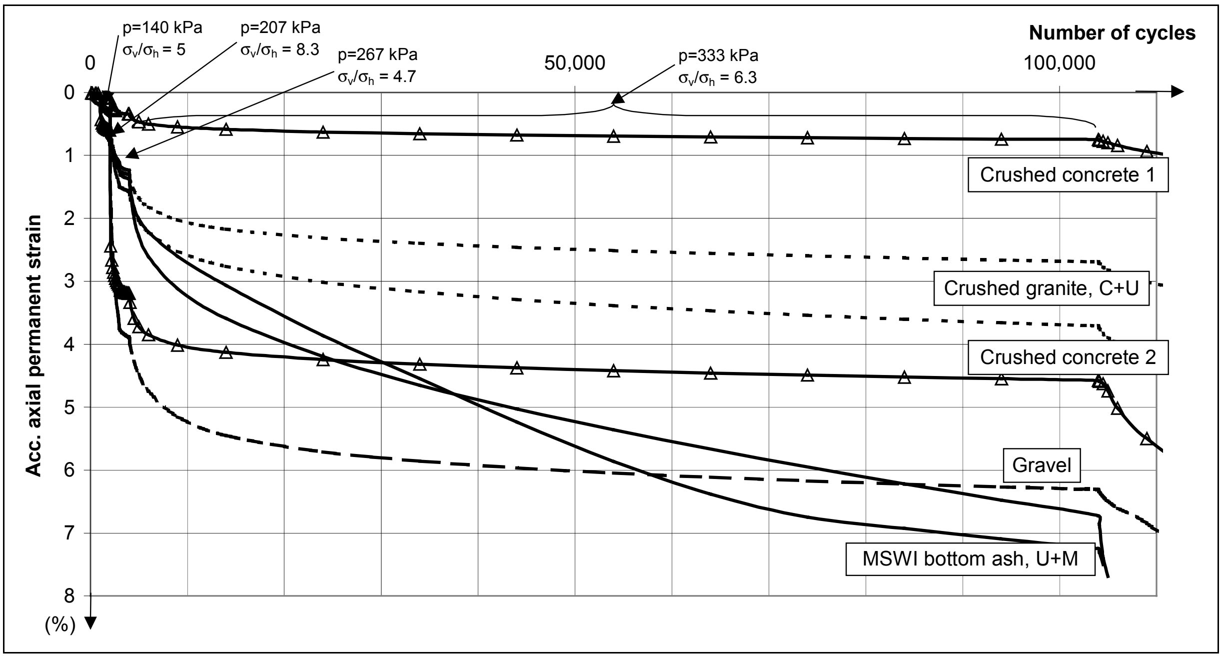


Figure 4. Examples of permanent deformation behaviour for the same materials as in Figure 3.

4.3 Methodology for designing an alternative pavement

In case the properties of a tested material differ too greatly from the properties of the material normally used, it is recommended that an alternative pavement should be designed. For this purpose the permissible load must be evaluated for each material that is planned to be used and a pavement is then designed where the permissible load is not exceeded in any material layer.

The permissible load of each material could be estimated from the material deformation properties and a stated limit value for the permanent deformation. The deformation properties are evaluated either from triaxial tests (Alternative A) or from experience data in a database (Alternative B). The limit value depends on the desired road standard or road category.

Estimation of the permissible load

Alt. A For an *unknown* (alternative) material the permissible load is estimated by means of cyclic load triaxial tests. The tests are performed at several stresses and at different water contents. The results are reported in a graph, where accumulated permanent strain is plotted as a function of the stress and water content (Figure 5).

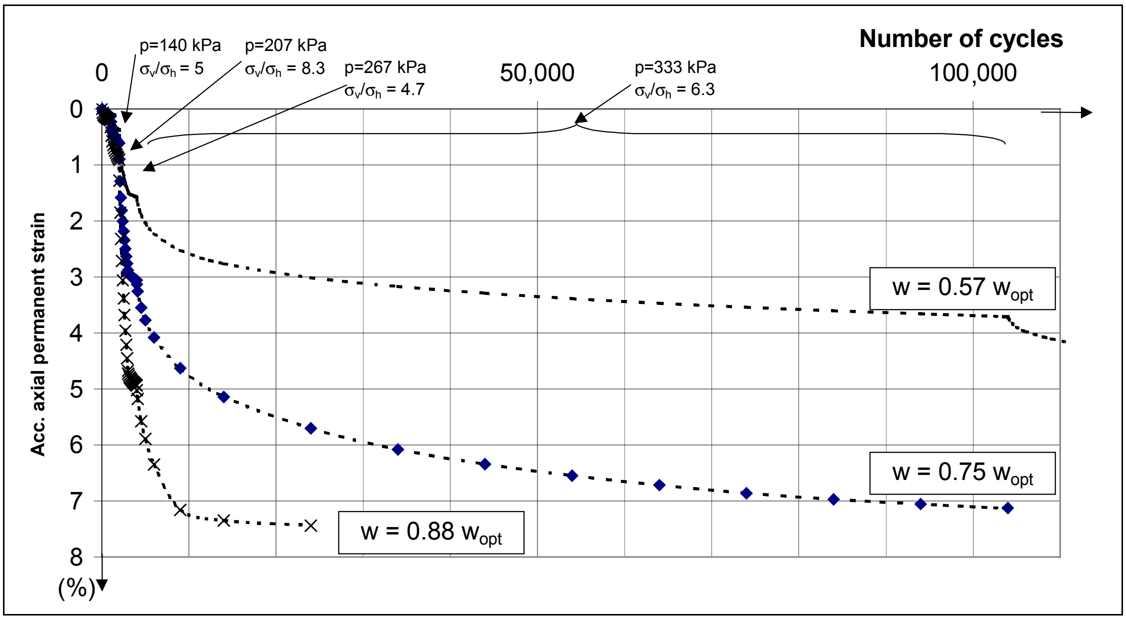


Figure 5. ***Example of water susceptibility of permanent strain. Well-graded crushed granite with a particle size of 0–20 mm and a fines content of 8%.***

From this graph a choice is made between the highest stress that gives a 'constant' permanent strain <u>or</u> the stress level that is closest to the stipulated limit strain value. Interpolation could be a possibility.

The permissible load could be calculated automatically from the test data via strain rate observation, either as the stress state where the deformation curve levels off completely or as the stress state where strain rate is lower than a certain value.

Alt. B For a *well-known* (conventional) material the load is estimated by means of triaxial tests performed earlier on similar or almost similar material. Apart from the origin of the material, three properties must be given to describe the material. The data are then compared with data on material already tested and registered in the database. The three properties needed are

1. Particle size distribution: Either the whole curve or a few descriptive parameters, such as maximum particle size, d_{max}, fines content and uniformity coefficient, c_u, or another indicator of the curve shape.
2. Particle shape (crushed, uncrushed, perhaps also flakiness index).
3. Water content or 'Drainage degree' expected for the material in the proposed pavement (according to ATB VÄG).

Alternative B requires the data from previous tests to be stored in a suitable way in the database.

Design of an alternative pavement where the permissible load is not exceeded

This action is performed through an iterative procedure where the layer thicknesses are varied and the stress at certain depths is calculated. M_r for the material is changed according to calculated stress.

- A pavement is chosen with the new material in one of the unbound layers.
- The M_r of the material is evaluated for different stress states if this has not already been done.
- The 'real' stress state is calculated at different depth in the unbound layers of the pavement. This is done using software that allows stress-dependent moduli. (It is also possible with a

sub-layering technique in order to account for the non-linearity of the materials.) The M_r from the triaxial tests described above is used for the alternative material. For other materials, moduli from a database are used.

- The stresses are checked against the permissible load for each material and the pavement is adjusted according to the results. This step is repeated until the limits are no longer exceeded. Adjustments could be done either by decreasing the load or by increasing the strength of the material. The load could be decreased by choosing a thicker or stiffer bound surfacing layer or by placing the material further down in the pavement. The material strength could be increased by means of stabilisation of the material.

5 MATERIAL DATABASE

When a material has been tested in a cyclic load triaxial test, the test results should be stored for future comparison together with other material data, such as origin, particle size distribution, particle shape (uncrushed/crushed) and water content. When a new material is to be assessed, a comparison can be made by checking origin, particle size distribution and particle shape.

In previous projects, the Swedish National Road and Transport Research Institute (VTI) has systematically investigated the effects of different parameters on the deformation properties of natural aggregates (Ydrevik, 1996; Arm, 1996b, 1997, 1998). The following parameters have been investigated.

- Particle size distribution
- Particle shape (crushed–uncrushed, flaky–rounded)
- Water content
- Compaction/density

In addition, different rock types (granite, gneiss, limestone) have been studied.

The findings in these investigations regarding permanent deformation at loading are described briefly below. For the purpose of this paper, the results have been translated into permissible load as described in Section 4.3 Alternative A.

5.1 Impact of particle size distribution on permissible load

Figures 6, 7 and 9 show the particle size distribution for a number of materials that have been tested in cyclic load triaxial tests at VTI. In Figures 7 and 9 the materials have the same origin, crushed fine-grained granite. On each curve there is the estimated permissible load in kPa. In those cases where two or three values are given, the material has been tested at several water contents. The difference shows how water-susceptible the deformation properties of the material are.

For natural sand and gravelly sand the permissible load decreases with increasing sand content and also with a decreasing c_u (Figure 6).

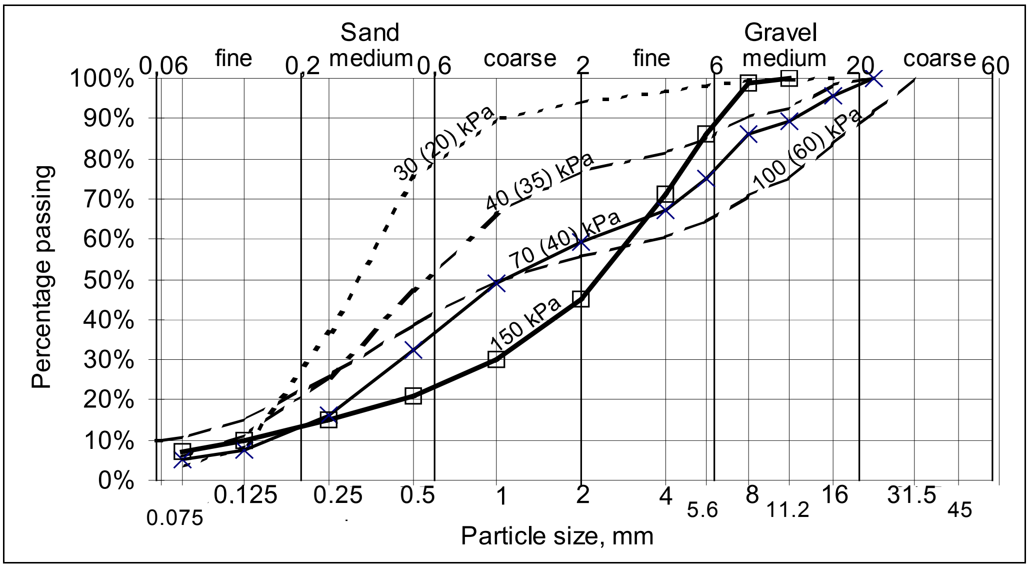


Figure 6. ***Comparison of permissible load for four natural sand materials plus one crushed granite material 0–8 mm (bold line). Water content of the sand 0.6 w_{opt} (0.8 w_{opt}) and accumulated strain limit 2%.***

Crushed granite with a particle size of 0–8 mm can bear a relatively high load due to its angular particles (150 kPa in Figure 6). Note that the materials in Figure 6 have been tested at low stresses using the 'capping layer test'.

All gradings in the approved zone for Swedish base courses (Figure 1a) produced about the same permissible load, approximately 400 kPa, with decreasing values in wet conditions for gradings with a higher fines content (Figure 7). A gap-graded material could not bear an equally high load while a 'sand-bump' material behaved surprisingly well.

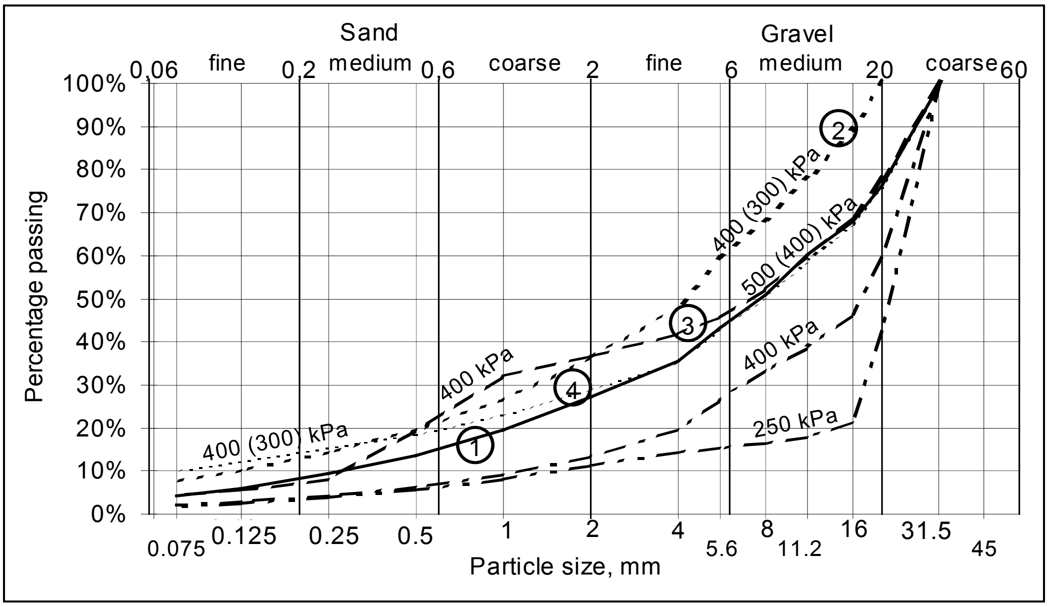


Figure 7. ***Comparison of permissible load for six crushed fine-grained granite materials. Water content 0.6 w_{opt} (0.8 w_{opt}) and accumulated strain limit 2%.***

5.2 Impact of downscaling

When a material is too coarse-grained to be tested in the triaxial test equipment, i.e. when the maximum particle size is larger than five times the specimen diameter, scaling down the material before testing could be a solution. The following experiences regarding downscaling were gained in the previous tests (Arm, 1998). When scaling down a material the M_r was decreased in all cases, which coincides with the findings of Thom (1988) and Kolisoja (1997). However, the change in the permanent deformation behaviour depended on the new particle size distribution achieved. If the scaling-down action resulted in a steeper curve, the scaled-down material had poorer stability (larger permanent deformation) than the original material and vice versa. The conclusion was that if the intention is to retain the properties as far as possible the curve shape for instance the c_u should be maintained. However, attention must also be paid to the change in the fines content.

5.3 Impact of water content on permissible load

The impact of water content on permissible load was found to vary for different materials (Arm, 1996b). The fines content and the origin were of vital importance (as expected). For well-graded materials and materials containing a relatively high proportion of fines, the permissible load was decreased with increased water content, as shown in Figures 6 and 7. In all tests the change in water content was achieved through admixing certain amounts of water before testing.

The impact of water content on accumulated permanent strain in four of the materials in Figure 7 is shown in Figure 8. In addition, there is data on a fifth material, which is crushed limestone with the same grading as material No 1. Linear curve fits have been made for materials 2, 4 and 5.

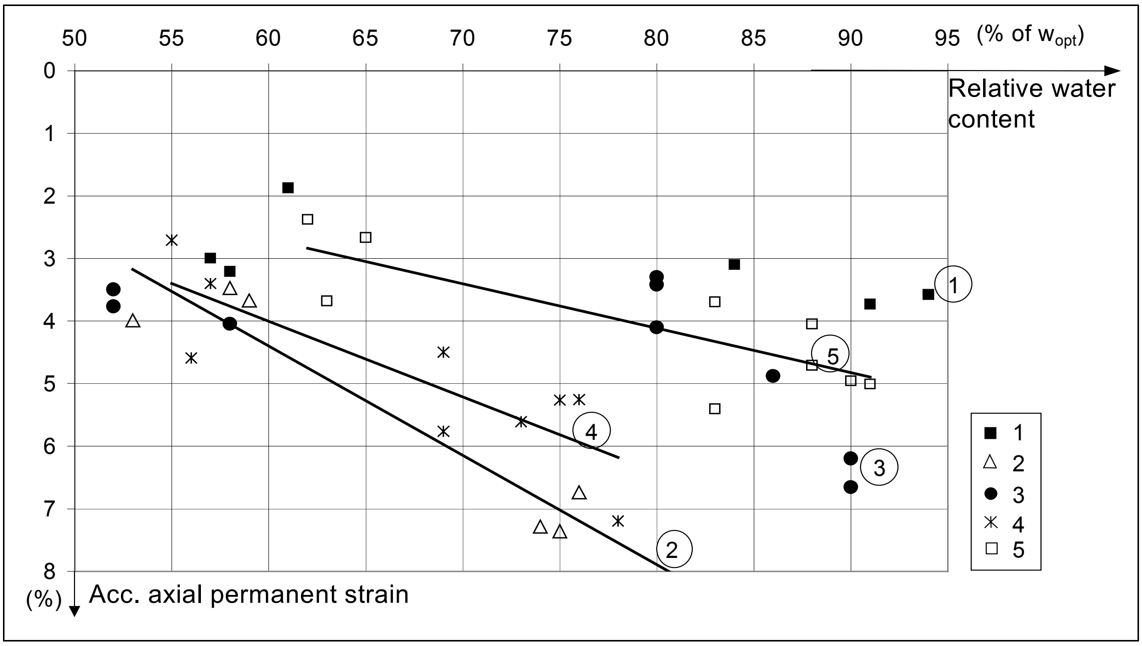


***Figure 8.** Impact of water content on accumulated axial permanent strain during a cyclic load triaxial test. Crushed granite with gradings according to Figure 7 and crushed limestone (5) with the grading 1. After 104,000 loads with σ_v = 120–620 kPa and σ_h = 60–120 kPa. (After Arm, 1996b).*

An interesting observation from Figure 8 is the large difference between the two approved gradings, (1) = 'Centre of zone' and (2) = 'Upper limit'. The gradings (3) = 'Sand-bump' and (4) = 'High fines content' are not approved according to the present specifications.

The combined effect of water content and particle size distribution was also obvious when three types of sub-base materials were compared (Figure 9).

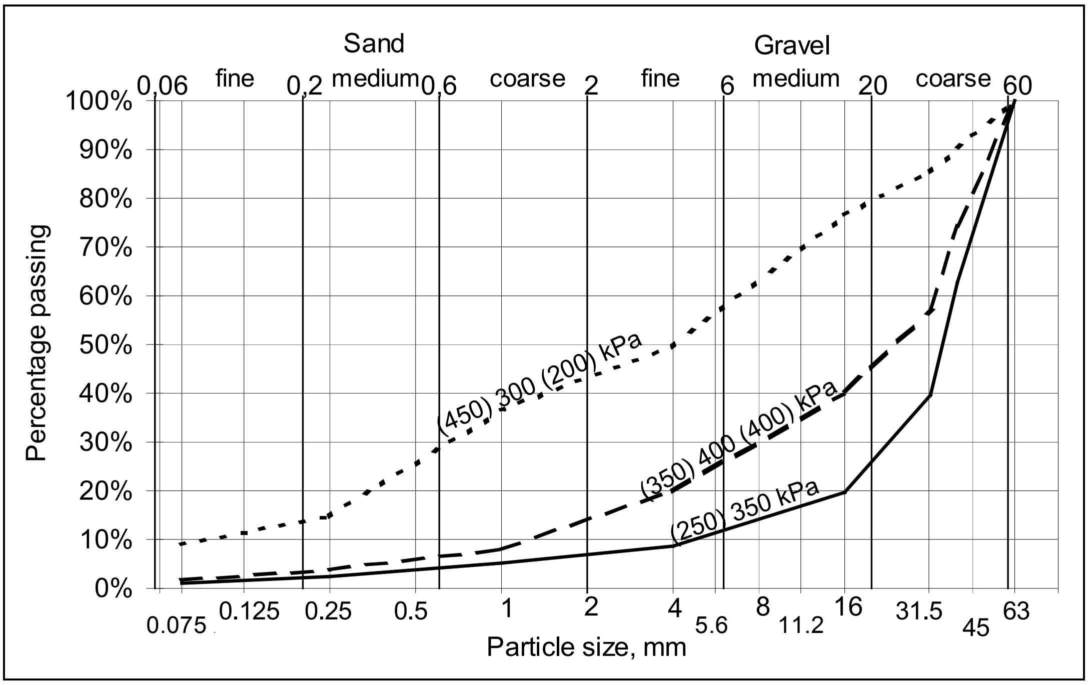


Figure 9. Permissible load in kPa for three sub-base materials. Water content (0.3 w_{opt}), 0.6 w_{opt}, (0.9 w_{opt}) and accumulated strain limit 2%.
NB. The results are from tests with a specimen diameter of 300 mm. Comparisons have shown a somewhat lower permissible load in tests with a smaller specimen diameter.

The old type of sub-base material with a high fines content and also a small 'sand-bump' performed best in dry conditions (0.3 w_{opt}) and even better than the two other materials. At medium and high water content (0.6 w_{opt} and 0.9 w_{opt}) the situation was the reverse.

The technique for translating from water content to 'Drainage degree' can be discussed. In the triaxial tests performed so far, material with a water content of w = 0.6 w_{opt} has been called 'naturally moist'. It is therefore suggested here that 0.6 w_{opt} corresponds to 'Drainage degree' 2 on a scale of three used in the present design guidelines ATB VÄG (SNRA, 2003). This means that if good drainage can be guaranteed, the right-hand part of Figure 8 does not need to be considered.

5.4 Impact of compaction and particle shape on permissible load

Compaction of a well-graded material increases its stability. Previous triaxial tests on well-graded crushed rock with a particle size of 0–32 mm showed that an increase in the compaction degree from 92% to 97% halved the permanent deformation (Ydrevik, 1996). Compaction degree was defined as actual density related to the maximum density achieved in modified proctor tests.

The compactibility is affected to a great extent by the particle shape and the relative water content. The impact of particle shape on resilient and permanent deformation was reported by Ydrevik (1996). Crushed fine-grained granite and natural gravel with the same particle size distribution, both with the fraction 0–32 mm, were tested and the gravel obtained more than twice as large a deformation as the crushed material (Figure 10). It was also noticed that cubic particles caused greater instability than flaky particles.

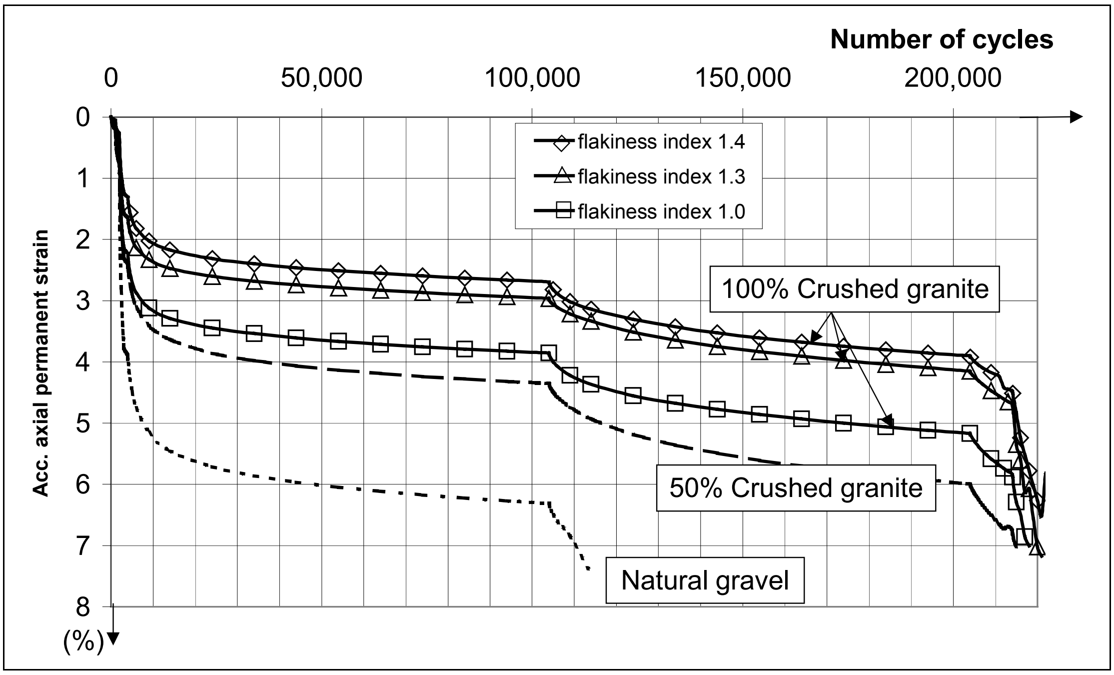


Figure 10. Impact of particle shape on permanent deformation of five granite materials with the same grading.

6 DISCUSSION

Many parameters have an impact on the deformation properties of an unbound material (load, relative water content, particle size distribution, particle strength, particle shape, organic content, degree of compaction/density). In the present Swedish specifications these are taken into account and are controlled by prescribing a certain value or a certain permitted range for each property. In a cyclic load triaxial test all these parameters contribute to the results.

Cyclic load triaxial tests have been used in research for many years (Lekarp et al., 2000). Common objections to the method are that it is difficult, time-consuming and expensive. This is valid for the early use of the method, which was only for modelling purposes. However, the way that VTI has used the method, i.e. for comparing and ranking unbound materials, does not have all these drawbacks. In recent years, similar ideas of use have arisen at the Norwegian research institute SINTEF (Hoff, personal communication, 2002) and also at the University of Dresden (Werkmeister, personal communication, 2002). It has therefore been possible to introduce the method in the European pre-standard for cyclic load triaxial tests (prEN 13286-7) in addition to the original draft method from France and Portugal. In the pre-standard, the VTI method of loading, where the same specimen is exposed to several stress levels, is called multi-stage testing.

The first draft of the CEN standard for cyclic load triaxial tests on unbound mixtures contained a French suggestion for a classification graph, which was based on a combination of the so-called characteristic resilient modulus, E_c, and the characteristic permanent axial strain, ε_c. Apart from the test description, the present draft of the standard contains an informative annex for material ranking procedures based on either resilient behaviour or permanent deformation behaviour. The procedure for resilient behaviour is a modification of the first French suggestion, with an E_c defined as the resilient modulus at stress values p = 250 kPa and q = 500 kPa (which corresponds to a vertical stress of 583 kPa and a horizontal stress of 83 kPa) and an ε_c calculated as the permanent axial strain after conditioning (20,000 cycles with a certain stress state). In the ranking procedure based on permanent deformation behaviour, three ranges of behaviour are discussed: stable deformation behaviour, failure at a high number of load cycles and failure at a low number of load cycles. Furthermore, a model including the ratio between vertical stress and horizontal stress is suggested for finding the border between the different ranges. This ratio has also been used in the VTI evaluation procedure (Ydrevik, 1996).

There are many ways of presenting results from cyclic load triaxial tests. It is important that the results are understandable and useful, otherwise the method will continue to be a researcher's project and not a tool for characterising and comparing materials. In this paper all permanent deformations and strains are plotted downwards. There are two reasons for this. Firstly, it coincides with the natural feeling for compression of a pavement layer. Secondly, this is the traditional way to plot results from settlements and compressions in soil mechanics.

Furthermore, vertical stress, σ_v, and horizontal stress, σ_h, are easier to imagine than mean normal stress, $p = (\sigma_v + 2 \bullet \sigma_h)/3$, and deviator stress, $q = (\sigma_v - \sigma_h)$.

It has been observed that the ratio between vertical and horizontal stress has a considerable impact on the deformations. The higher the ratio the less stress-dependency, probably due to crushing and subsequent alteration to more fine-grained material. A higher stress ratio could be expected in base courses under a surfacing layer of 70 mm asphalt concrete compared with under a bituminous spray seal or 40 mm of asphalt concrete. In the VTI tests the stress ratio ranged between 4 and 12. An extended range is desirable. The SHRP Protocol 46 (FHA, 1989) formed model for the test procedure, although the stresses used in the protocol were corrected since they were found to be too low for Swedish pavements. Kolisoja (1997) drew the same conclusion about the stresses in the SHRP P46.

In South Australia, stress states are expressed using p and an inverse stress ratio of vertical and horizontal stress (DoT SA, 1997). This is more intuitive than the CEN method using p and q. Furthermore, the 'inverse' is not necessary (Figure 11).

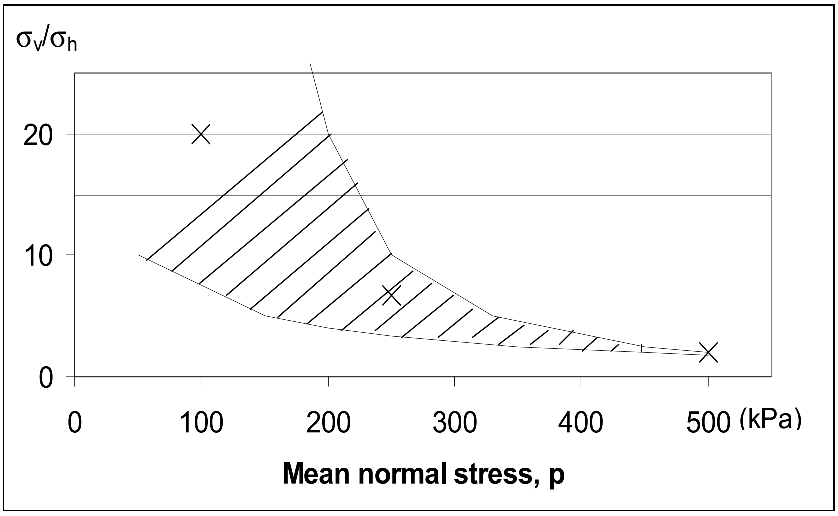


Figure 11. Stress space for unbound granular base courses. The crosses show, from left to right, a pavement with 70 mm asphalt concrete, with 35 mm asphalt concrete and with a bituminous spray seal only. (Modified from DoT SA, 1997)

In the stress space in Figure 11, the stresses in unbound granular base courses in pavements with different surfacing types, are indicated with large crosses. From left to right they represent 70 mm asphalt concrete, 35 mm asphalt concrete and bituminous spray seal. The origin of Figure 11 is derived from South Australia, where a project running over several years on characterising unbound pavement materials took place at the Department of Road Transport (DoT SA, 1997). The recommendations regarding permanent deformation behaviour of base course material from that project were based on both the maximum vertical strain and the strain rate. It was suggested that the strain after 50,000 cycles at p = 150 kPa and an inverse stress ratio of 0.17 and the permanent strain rate at 3,000 cycles under a stress regime of p = 150 kPa and inverse stress ratio of 0.15 should be used.

In Figures 6, 7 and 9 the permissible loads are written directly on the grading curves. This is a pedagogical manner of presentation, influenced by the Finnish road design manual from 1985 in which the E-moduli of different unbound materials were presented as figures in MPa in this way (TVH, 1985). However, an objection could be made that the figures only show the results from a few chosen stress paths. For other stress paths there are other permissible loads. For comparison purposes, however, this is of no importance when all materials are tested in the same way.

Crucial points in the comparison procedure are: how similar must the parameters be to be regarded as similar and what parameter is the most important? Guidance is already given in the present database and through the knowledge of how the different parameters affect the deformation. More experience is gained for every new test performed.

The methodology for estimating permissible load can be developed further and could include translation into a suitable depth in the pavement. The depth required depends mostly on the traffic load and the stiffness and thickness of the bound layers. Table 2 shows an attempt regarding application.

Table 2. Suitable depth from the surface for certain materials tested with regard to permissible load. Pavement with 40 mm of asphalt concrete. Stress calculation by means of BISAR software. (After Arm, 2000b)

	Suitable depth from surface (cm)					
Material/grading	**If 2% perm. strain**			**If 5% perm. strain**		
Crushed material 0–64 mm	**0.3 w_{opt}**	**0.6 w_{opt}**	**0.8 w_{opt}**	**0.3 w_{opt}**	**0.6 w_{opt}**	**0.8 w_{opt}**
‘Old sub-base’, c_u 53	10	14	19	4	7	14
‘Centre of zone’ (ATB VÄG), c_u 23	14	10	10	7	4	4
‘Open-graded sub-base’, c_u 6	14	10		10	7	
Crushed material 0–32 mm						
‘Upper base course limit’ (ATB VÄG)		10	14		0–4**	8
‘Centre of zone’ (ATB VÄG)		7	10		0–4**	0–4**
‘Lower base course limit’ (ATB VÄG)		10			4	
Uncrushed material 0–32 mm						
‘Centre of zone’ (ATB VÄG)		14	—		7	—
(Crushed granite 0–8 mm, c_u 25)		25			25	
45% sand, c_u 61		33	53		28	33
47% sand, c_u 23		47	72		33	39
54% sand, c_u 14		47	72		33	72
70% sand, c_u 7		72	72		62	72
91% sand, c_u 3		85	108		85	85

(w_{opt} = optimal water content, c_u = uniformity coefficient of the grading)
**depends also on resistance to wear and degradation.

In Table 2 the first column could be renamed ’Material type’ and the different water contents could be called ’Drainage degree’ 1, 2 and 3.

Of course, the specific deformation values obtained in the cyclic load triaxial tests cannot be expected to appear in the field. The laboratory results must be compared with field performance. Such a validation is about to be carried out in Sweden by means of instrumented test sections loaded by a Heavy Vehicle Simulator, HVS. The HVS tests include test sections with crushed and uncrushed granite as well as crushed granite and gneiss with different mica content. Small *in situ* methods, such as dynamic cone penetrometer, DCP, are also used.

Future action includes calculation of the difference in design life and calculation of the cost of exceeding the permissible load. Development of deformation due to degradation of the protecting bound layer could also be calculated. This equation should include the vertical stress in, and water ratio of, the unbound layer.

Finally, the permissible load assessed from triaxial tests should be part of a 'first' characterisation of a new material. A characterisation that should form the basis for certification and technical approval, like the European CE mark or the Dutch KOMO mark (CROW, 1999). Production control or quality control could take place as described in Section 4.3 Alternative B in order to check that the material in question complies with the certified material.

7 CONCLUSIONS

- Cyclic load triaxial tests can be used for comparing and ranking unbound road materials.
- The methodology suggested has clearly shown the impact of particle size distribution, particle shape, water content and compaction on deformation properties.
- The methodology can be introduced as part of performance-related material specifications.
- The quality requirements for materials could be more differentiated and thus offer the potential to use a wider range of materials than at present. As an example, higher requirements are needed for materials placed directly under a thin asphalt surfacing than under a thick surfacing.
- It is anticipated that wider use of cyclic load triaxial tests for characterising unbound road materials, both conventional and alternative, will result in 'suitability for purpose' use and thus more efficient use of natural materials and more sustainable resource management.

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REFERENCES

Alavi, S., Merport, T., Wilson, T., Groeger, J. & López, A., (1997) *LTPP Materials characterization program: Resilient modulus of unbound materials (LTPP Protocol P46) Laboratory startup and quality control procedure.* Federal Highway Administration, USA.

Arm, M., Svensson, J. & Ydrevik, K., (1995) *Deformationsegenskaper hos finkornig jord.* VTI meddelande 770. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Arm, M., (1996a) *Dynamic testing of fine-grained subgrade soils.* In: Correia (edited) Flexible Pavements. Balkema. pp 135–142.

Arm, M., (1996b), (1997), (1998), (2000a) *Obundna vägmaterials mekaniska egenskaper. Bärförmåga hos obundna vägmaterial bestämd med hjälp av dynamiska treaxialförsök.* (*Mechanical properties of unbound road materials. Permissible load of unbound road materials evaluated from cyclic load triaxial tests.)* Annual project reports to the National Road Administration. Väg- och transportforskningsinstitutet, Linköping. (In Swedish)

Arm, M., (2000b) *The properties of alternative aggregate materials – with special reference to MSWI bottom ash, crushed concrete and blast furnace slag.* Licentiate thesis. TRITA-AMI LIC 2063. Royal Institute of Technology, Stockholm. (In Swedish with an extensive summary in English)

AS, (1995) *Determination of the resilient modulus and permanent deformation of granular unbound pavement materials.* AS 1289.6.8.1 - 1995. ISBN 0 7262 9515 9. Standards Australia.

CROW, (1999) *Nationale beoordelingsrichtlijn voor het KOMO-attest-met productcertificaat voor AVI-bodemas voor ongebonden toepassing op of in de bodem in grond- en wegenbouwkundige werken.* BRL 2307 d.d. 1999-07-23. CROW.

DoT SA, (1997) *Performance comparisons of granular basecourse materials using repeated load triaxial testing.* MTRD Report no MT 16-10. Department of Transport, Walkley Heights, South Australia.

FHA, (1989) *SHRP Protocol 46. Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils.* Publ no FHWA-RD-96-176. Federal Highway Administration, USA.

Kolisoja, P., (1997) *Resilient Deformation Characteristics of Granular Materials.* Dissertation. Publication 223. Tampere University of Technology.

Lekarp, F., Isacsson, U. & Dawson, A., (2000) *Permanent strain response of unbound aggregates.* Journal of transportation engineering, Jan/Feb 2000, pp 76–83.

Pihl, K. & Milvang-Jensen, O., (1996) *Bundsikringslag af forbraendingsslagge.* Vejteknisk Institut Rapport 78 1996. Vejdirektoratet, Roskilde.

prEN 13286-7, April 2003. *Unbound and hydraulically bound mixtures. Part 7: Cyclic load triaxial test for unbound mixtures.* CEN.

Schmith, N.B., (1984) *Incinerator Residue as a Pavement Material III.* Notat 176. Statens Vejlaboratorium, Roskilde.

SNRA, (2003) *ATB VÄG. Allmän teknisk beskrivning för vägkonstruktioner (General Technical Specifications for Roads).* Vägverket (Swedish National Road Administration), Borlänge. (In Swedish)

Sweere, G.T.H., (1990) *Unbound granular bases for roads.* Dissertation. Delft University of Technology.

Thom, N.H., (1988) *Design of road foundations.* PhD thesis, University of Nottingham.

TRL, (2000) *ALT-MAT Draft final report for publication.* Report No WP6.TRL.001. www.trl.co.uk/altmat/index.htm. Transport Research Laboratory (TRL), Crowthorne.

TVH, (1985) *Teiden suunnittelu.* TVL:n ohjeet. Tie- ja vesirakennushallitus (TVH), Finland.

Ydrevik, K., (1996) *Dynamic triaxial testing of roadbase and lower sub-base materials.* VTI notat nr 4-1996. Swedish National Road and Transport Research Institute, Linköping. (In Swedish, but partly translated into English)

Hoff, Inge (2002): SINTEF, Trondheim. Personal communication.

Werkmeister, Sabine (2002): University of Dresden. Personal communication.

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