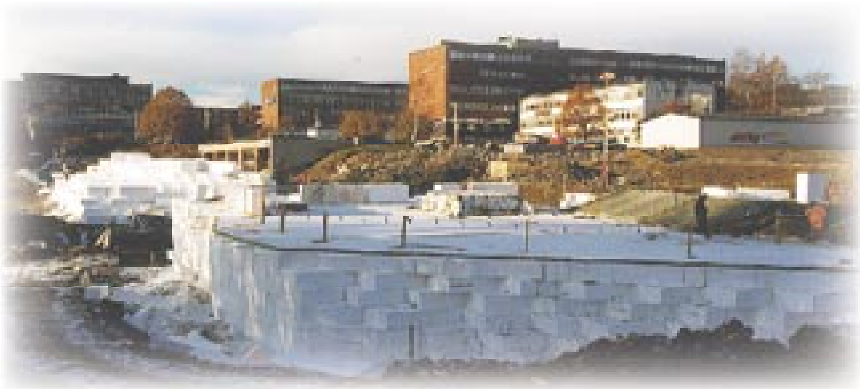
no. 100

Publication

Lightweight filling materials for road construction



Norwegian public Roads Administration

Road Technology Department

Publication no. 100

Lightweight filling materials for road construction

Directorate of Public Roads

Road Technology Department

Oslo, December 2002

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Preface

In Norway the Public Roads Administration has a long tradition in applying various types of llightweight filling materials for road construction purposes. During the last 50 years wooden materials like sawdust and bark residue from the timber industry have been applied for such purposes. Also waste materials from the production of cellular concrete blocks and LWA (Light Weight Clay Aggregate) have been widely used. Since 1972 blocks of Expanded Polystyrene have been used extensively in road projects for a variety of applications also including blocks produced from re-circulated EPS material. In all cases monitoring programmes have been initiated in order to investigate the long term performance of these materials.

In the present publication experience gained over some 30 years in Norway with the use of EPS as a lightweight filling material in roads is presented based on observations and recorded data from monitoring programmes. We now see both a wider use of this material on a global scale and the introduction of a number of different design applications. In addition to reduced vertical loads, advantages from using EPS may also include reduced horizontal loads, simplified designs, foundations placed directly on EPS blocks and increased speed and ease of performing construction activities. The two papers on EPS were presented at EPS Geofoam, 3rd International Conference, Salt Lake City, December 2001.

Also preliminary results from monitoring lightweight fills with granulated foamed glass produced by re-circulating waste glass are presented. This is a fairly new option and the material may be produced in various densities. When placed and compacted in a drained fill the unit density of the lightest version will be some 300-350 kg/m^3 depending on the compaction machinery and compaction efforts. The mechanical strength of the material require some special handling in order to prevent excessive crushing during placement and compaction. The paper on foamed glass was presented at an International Workshop on Lightweight Geomaterials, Tokyo, March 2002.

Norwegian Road Technology Department
Oslo, December 2002

Contents

Long term performance and durability of EPS as a lightweight filling material

Tor Erik Frydenlund and Roald Aabøe

Paper presented at EPS Geofoam 3rd International Conference, Salt Lake City, December 2001.

ABSTRACT

Some 30 years of experience with Expanded Polystyrene (EPS) as a lightweight filling material in Norway has brought about both a wider use on a global scale and the introduction of a number of different design applications. In addition to reduced vertical loads, advantages from using EPS may also include reduced horizontal loads, simplified designs, foundations placed directly on EPS blocks and increased speed and ease of performing construction activities. This document describes practical experiences in Norway with long term performance and durability of EPS as a fill material based on observations and recorded data from monitoring programmes.

1 Introduction

In Norway the Public Roads Administration has a long tradition in applying various types of lightweight filling materials for road construction purposes. During the last 50 years wooden materials like sawdust and bark residue from the timber industry have been applied for such purposes. Also waste materials form the production of cellular concrete blocks and Leca (Light Expanded Clay Aggregate) have been widely used. In this connection also monitoring programmes were initiated in order to investigate the long term performance of these materials. Presently a new option is being investigated involving the use of granulated foamed glass produced by re-circulating waste glass.

When a major research project on frost action in soils was carried out in Norway during the period 1965 to 1973 this included the investigation of various insulation materials for frost protection of roads like 50 to 100 mm thick boards of foamed glass, extruded polystyrene (XPS) and expanded polystyrene (EPS). In this connection also fatigue tests were performed. It was then concluded that EPS material could sustain the repetitive stresses occurring in a road structure and the idea of applying EPS in greater layer thickness than boards emerged.

In 1972 the Norwegian Public Roads Authorities adopted the use of EPS as a super light filling material in road embankments. The first project involved the successful reconstruction of road fills adjacent to a bridge founded on piles to firm ground. Prior to reconstruction the fills, resting on a 3 m thick layer of peat above 10 m of soft marine clay, experienced a settlement rate of more than 200 mm per year. By replacing 1 m of ordinary fill material with two layers of EPS blocks, each layer with 0,5 m thickness, the settlements were successfully halted. When placed the EPS blocks had a density nearly 100 times lighter than the replaced materials.

Since then authorities in several countries have also found the method advantageous for building roads across soft ground and for other construction purposes where low loads are essential. In addition to reduced vertical loads, advantages from using EPS may also include reduced horizontal loads, simplified designs, foundations on EPS and increased speed and ease of performing construction activities. The method is now in common use in several countries in Europe, Asia and North America. At present more than 350 road projects involving EPS fills have been completed in Norway with a volume of totalling some 500,000 m^3 of EPS blocks.

EPS does of course not represent the only solution to bearing capacity and settlement problems. Other lightweight filling materials should also be considered together with other alternatives such as replacement or displacement of the weak soil or soil improvement, piled foundations etc. Availability and cost are important factors in this connection, but in many cases the use of EPS will prove advantageous and in some cases represents the only practical solution. In a book published (1997) by PIARC, the World Road Association, lightweight filling materials in common use are presented together with case histories.

2 Monitoring programme

Expanded polystyrene is a very stable compound chemically and no material decay should be expected when placed in the ground and protected according to the present design guidelines. Still, since the first road insulation project with EPS was performed in 1965 and the first EPS light weight embankment was constructed in 1972, EPS fills have also been monitored for long term performance along the lines followed for other lightweight filling materials used in road construction in Norway.

Figure 1: An EPS embankment with vertical walls for a city tramline in Oslo as an alternative to a bridge.

The monitoring programme has focused on the following material qualities:

- Material behaviour
  - *Compressive strength*
  - *Water absorption*
  - *Decay*
- Deformation
  - *Total fill deformation and deformation in EPS layers*
  - *Creep effects*
- Stress distribution
- Reduced lateral pressure
- Bearing capacity.

3 Testing frequencies

Since 1972, several tests have been carried out in order to monitor possible material changes. In this connection test samples have been retrieved from existing fills to be checked for possible changes in strength and unit density. Also variations in water absorption for blocks placed in drained, submerged or semi-submerged positions have been observed. In order to determine the stress distribution within blocks and fills both laboratory and field tests have been performed. Finally load creep effects have been observed both in the laboratory and on existing fills.

In Norway test samples have been retrieved from a total of five fills. The testing frequency is shown in Table 1.

Table 1: Testing frequencies of EPS embankments.

Fill location	**Constructed year**	**Test samples retrieved, no. of years after construction**			
National road 159 Flom bridges	1972 / 73	0	7	12	24
National road 154 Solbotmoan	1975	4	9	21	
County road 91 Lenken	1978	6			
County road 26 Langhus	1977	7			
National road 610 Sande - Osen	1982	9			

Figure 2: Excavation of the first EPS embankment at Flom bridge.

4 Material behaviour

Material strength

According to Norwegian specifications the design compressive strength of EPS blocks have been set to be at least σ = 100 kPa when not otherwise specified. In actual practice a shipment of blocks may be accepted if the average strength of tested blocks is σ = 100 kPa. The average value for test specimens from one block (6 tests) should be σ = 90 kPa and no single tests should show values σ < 80 kPa.

One major indicator of possible deterioration of blocks with time would be a decrease in the material strength. The strength tests performed on retrieved samples from fills having been in the ground for up to 24 years are shown in Figure 4 as a function of dry unit density and compressive strength. Bearing in mind the criteria mentioned above for accepting blocks to be placed in a fill, all test results give values of compressive strength above σ = 100 kPa except for one test.

Figure 3: Excavation of a 24 years old EPS block from the first EPS embankment at Flom bridge.

This one test was performed on samples taken from the first fill shortly after it was completed in 1972, and is more an indication of variations in material quality of EPS with the production process used at that time. Still the observed value is within the accepted statistical variations in material strength.

From Figure 4 it may also be observed that the majority of tests show values of compressive strength in relation to unit density above that of a normal quality material. Although it is of course impossible to make exact comparisons between material strength at the time of construction and some time afterwards since tests cannot be performed on the same specimen twice, the results indicate clearly that there are no signs of material deterioration over the total time span of 24 years. If a change tendency is to be noted, this would go towards a slight increase in material strength. If this is the case, such an increase could be explained by a continued chemical reaction leading to material hardening during the first few weeks after production. There is also a tendency that the material strength is slightly higher in the middle of the block than towards the outer sides. Furthermore there is no sign of variation in material strength whether the retrieved specimens are tested wet or dry. This indicates that water pickup over years in the ground will not affect material strength.

Unit density

The only change in design rules that have been introduced in Norway since the first fill in 1972 is that the design unit weight for EPS blocks placed in a drained position is reduced from $\gamma = 1.0$ kN/m^3 ($\rho = 100$ kg/m^3) to $\gamma = 0.5$ kN/m^3 ($\rho = 50$ kg/m^3) when stability and settlement calculations are performed. For blocks placed in a submerged or semi-submerged position the value of $\gamma = 1.0$ kN/m^3 ($\rho = 100$ kg/m^3) is maintained. The change mentioned above is based on tests data from existing fills.

EPS placed in the ground will absorb water in two ways. One is by water entering possible voids between spheres due to water pressure or capillary rise. Since water vapour may diffuse through the polystyrene when there is a temperature gradient, the water vapour will condense in the spheres if there is a drop in temperature below the dew point. However, in an EPS block of 500 mm thickness or an EPS fill of greater thickness the temperature difference over the block or fill will be very small. Possible water absorption due to water vapour diffusion is therefore expected to be small.

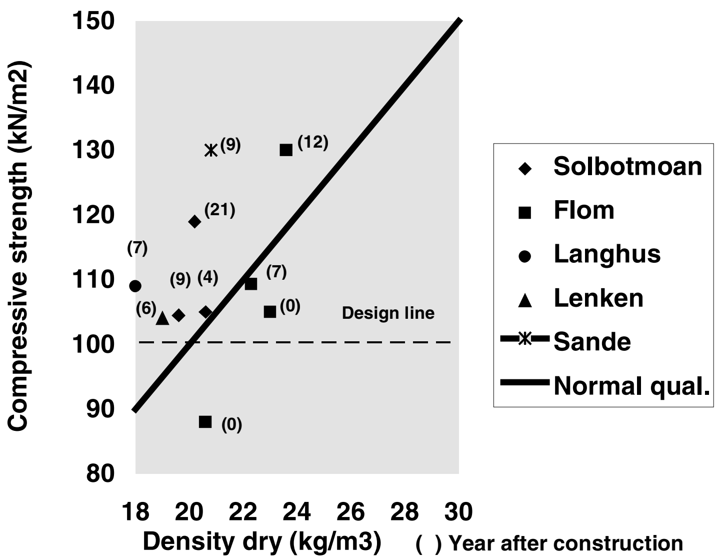

Figure 4: Compressive strength on retrieved samples from EPS fills.

Water absorption due to water pressure or capillary rise depends on unit density and how well the spheres are welded together. A number of tests, mainly on small samples in laboratories, have been performed in several countries in order to study water absorption effects. Both the quality of these tests and the results vary somewhat. Tests performed on samples retrieved form existing fills in Norway are in agreement with some of the laboratory tests.

Tests performed on samples retrieved from three EPS fills placed in a drained position, i.e. blocks are located above the highest groundwater or flood level, all show water contents below 1 % by volume after more than 20 years in the ground (Figure 5).

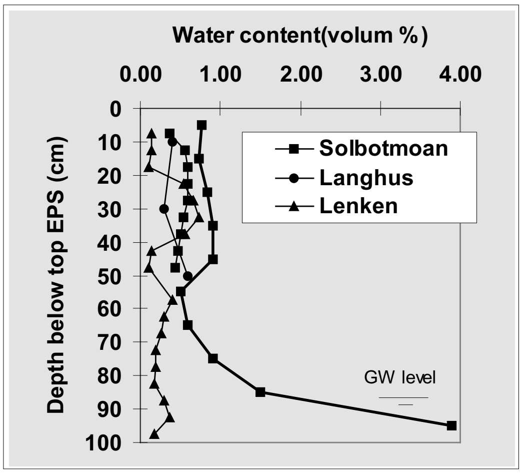

Figure 5: Typical drained situation from 3 EPS fills.

Furthermore there is hardly any change in the water content with time. Samples retrieved from the outer parts of blocks facing the surrounding soil, may have a higher water content as may be seen from Figure 6. But only 500 mm further into the block the water content is again below 1 % by volume. So the average density of drained fills therefore has values of $\rho < 30$ kg/m^3. This is well below the specified design value for such fills.

In blocks, which are periodically submerged, water contents of up to 4 % by volume have been measured. In permanently submerged blocks measured water contents have reached values close to 10 % by volume with some increase over the years, Figure 7. Further increases above 10 % by volume are, however, not to be expected. For submerged fills the average density is therefore of the order of $\rho = 90\text{-}95$ kg/m^3 after some 20 years in the ground. The water content decreases rapidly above the water table and show values for drained conditions only some 200 mm above the highest water level.

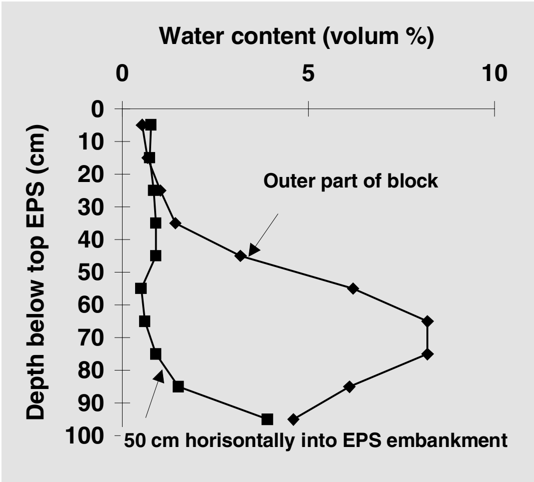

Figure 6: Horizontal gradient of water in EPS.

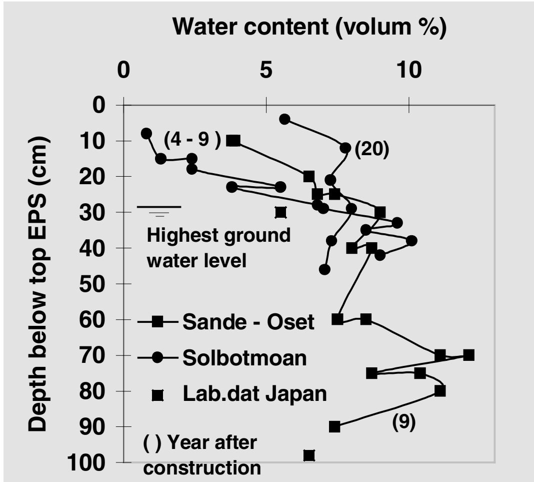

Figure 7: Typical water content in submerged EPS blocks.

5 Deformation and creep effects in EPS structures

Both full scale and laboratory tests have been performed related to material creep and stress distribution in the material. In general only about 30 % of the material strength is utilised for supporting dead loads, i.e. $q_{dw} < 30$ kPa for normal strength blocks ($\sigma = 100$ kPa). In some special cases higher stress related to dead loads have been used.

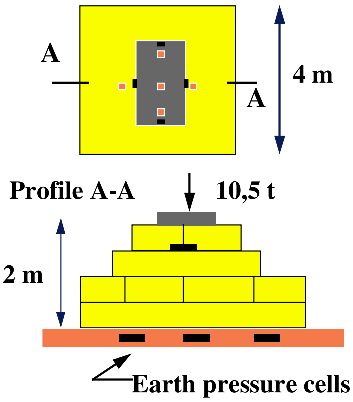

Figure 8: EPS test fill at the Norwegian Road Research Laboratory (now the Road Technology Department)

In a laboratory test at the Norwegian Road Research Laboratory (now the Road Technology Department) a test fill of height 2 m with normal size blocks and a compressive strength σ =100 kPa has been loaded to a value of $q_{dw} = 52.5$ kPa and the resulting deformations observed over a period of 3 years (Figure 8). The results are shown in Figure 9 together with calculated deformations to be expected according to the theories introduced by Magnan & Serratrice.

As may be seen the observed deformations are only about half of the calculated values and creep deformations with time are also much smaller.

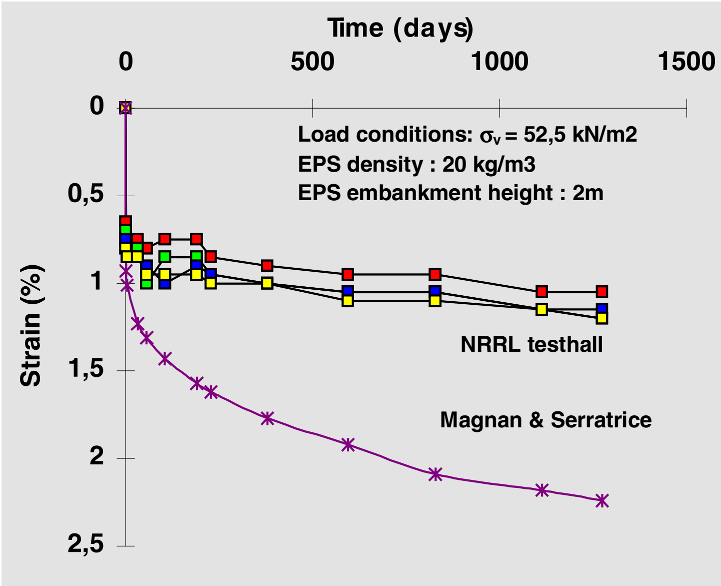

Figure 9: Deformation / creep in the test fill.

Full scale test at Løkkeberg bridge founded on two EPS embankments, long term monitoring of deformation, creep and stress distribution

The Løkkeberg bridge is a single lane Acrow steel bridge with a single span of 36.8 m crossing road E6 close to the Swedish border. The bridge was built in 1989 in order to temporary (3-5 years) improve traffic safety until the completion of a new motorway between Norway and Sweden.

Figure 10: Construction of one abutment on the EPS embankments at Løkkeberg bridge.

Due to low bearing capacity and expected large settlements, light weight fill materials (EPS) were considered in the embankments adjoining the bridge. The project provided an opportunity to place the bridge foundation directly on top of the EPS fills (height 4.5 m and 5 m) on both sides as an alternative to placing the bridge abutment on piled foundations. Since the bridge was a temporary solution and possible deformations could be adjusted during the period of operation, it was decided to carry out the project as a full scale test.

Three different qualities of EPS material strength have been used with design strengths of σ = 240 kPa in the upper layer directly below the bridge abutment, σ =180 kPa in the remaining layers halfway down the fill and σ =100 kPa in the bottom half. In the upper layer only 25 % of the material strength has been utilised while in the bottom layer the corresponding figure is 60 %. Construction details are shown in longitudinal profiles in Figures 11 and 12.

The bridge is today still in operation 12 years after completion. No signs of cracks or uneven deformation have been observed. The bridge support has been lifted 30 cm on one side due to subsoil settlements in accordance with the theoretical calculation.
Løkkeberg bridge has provided a good opportunity for monitoring long time performance such as creep and stress distribution of an EPS embankment.

After 12 years in service measurements show only small deformations 6 cm (1,3 % of the EPS height) in the EPS embankment. Most of the deformation occurred during the construction period and only minor creep effects have been measured. Creep deformations as an average and creep deformations for the lowest EPS layer (6,5 % of the layer thickness) are shown in Figure 13 for a period of 10 years.

Observed deformations after 10 years in operation are plotted in Figure 14 together with data from the laboratory test and theoretical values according to Magnan & Serratrice calculated for various stress levels

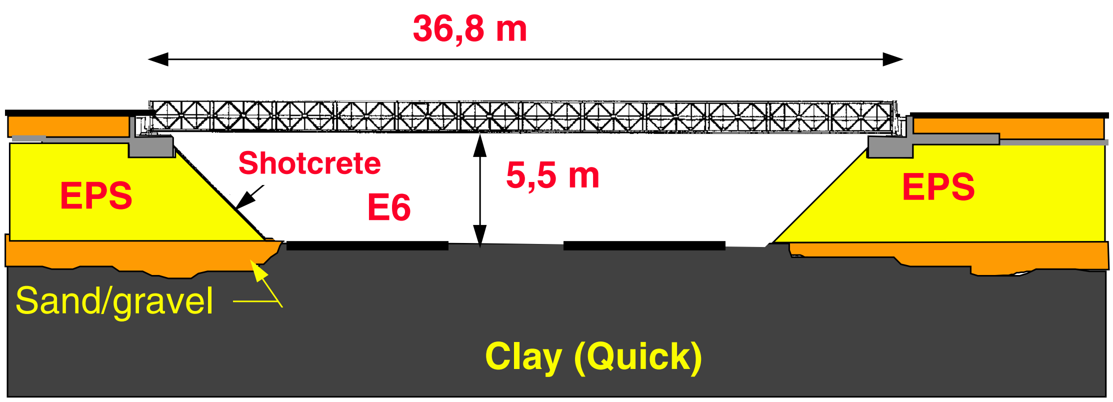

Figure 11: Longitudinal profile of Løkkeberg bridge.

The figure clearly shows that the average deformation at the Løkkeberg bridge is small and slightly over 1 % of the total fill height. Also observed creep effects are almost negligible for the total fill although deformations in the bottom block layer was 4 % initially and later creep effects amount to further 2.5 %. Creep deformations in the bottom layer correspond with the theoretical values the first 5 years but has later slowed down to almost zero.

Two similar structures, a 3 span bridge at Hjelmungen and a 3 span pedestrian bridge has been built in 1994 and 1995 with abutments founded on EPS fills. Observations from these bridges correspond with the measurements at Løkkeberg and shows that this can be a promising method for supporting bridge abutments.

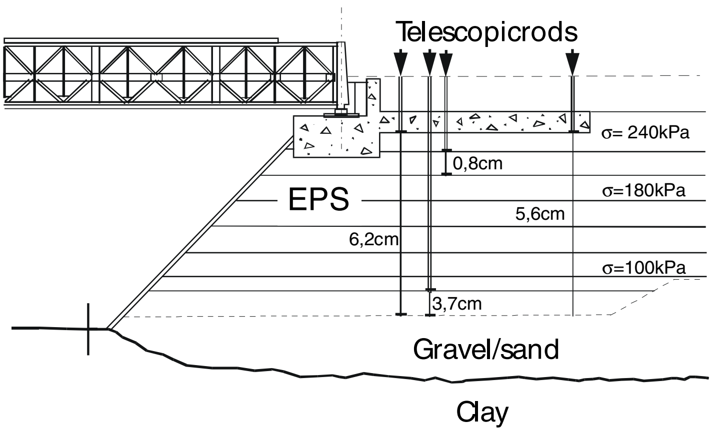

Figure 12: Deformation in EPS embankment at Løkkeberg.

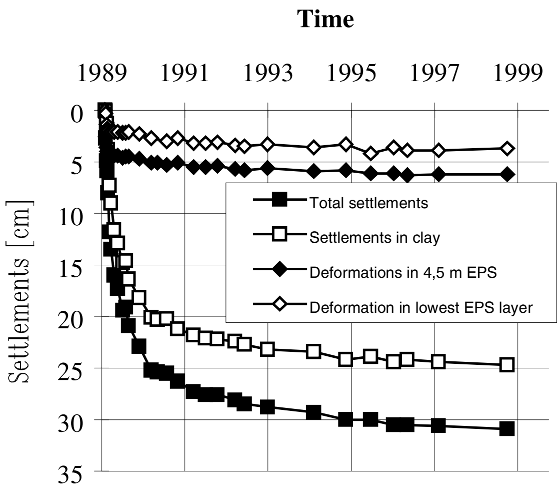

Figure 13: Creep deformation at Løkkeberg.

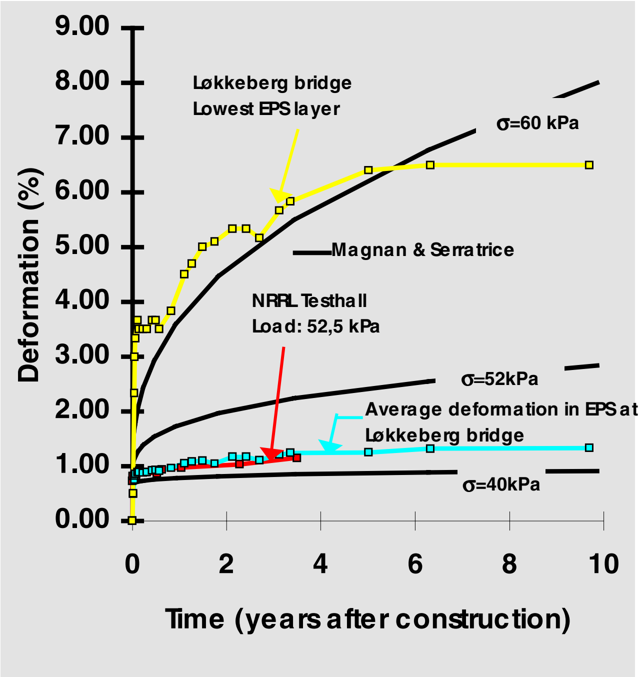

Figure 14: Creep deformations in EPS.

6 Stress distribution

In order to observe the stress distribution in the EPS material below the bridge abutment at Løkkeberg bridge during construction and on a long term basis, 10 hydraulic earth pressure cells have been placed in different levels in the fill including 3 cells in the sand layer below the EPS fill. In Figure 15 the measured stress level after 10 years in service is indicated with red figures.

Observations indicate that cell boundary effects may have influenced the stress results, especially in the first loading stage, probably due to poor interaction between EPS and the steel casings for the earth pressure cells.

Long term measurements from 3 earth pressure cells below the fill and one situated 2 m higher up have been plotted in Figure 16. During the first year of operation a stress decrease of 15 - 30 % was observed. Later only small variations with time have been observed. Measured stresses corresponds well with the theoretical vertical load in the lower part of the EPS fill.

In the upper part of the fill (with a higher material strength) lower stress than expected has been measured in a zone under the central part of the embankment. One explanation could be some kind of arching effect due to large subsoil settlements in the middle of the embankment. This can clearly be seen in Figure 15 where the measurements from the settlement tube are shown. In the lower part of the fill it is difficult to explain load concentration and deformation in the lowest EPS layer and likewise the low stress values in the upper part of the EPS fill. It may therefore be concluded that there still may be load distribution mechanisms in EPS fills that are not fully understood.

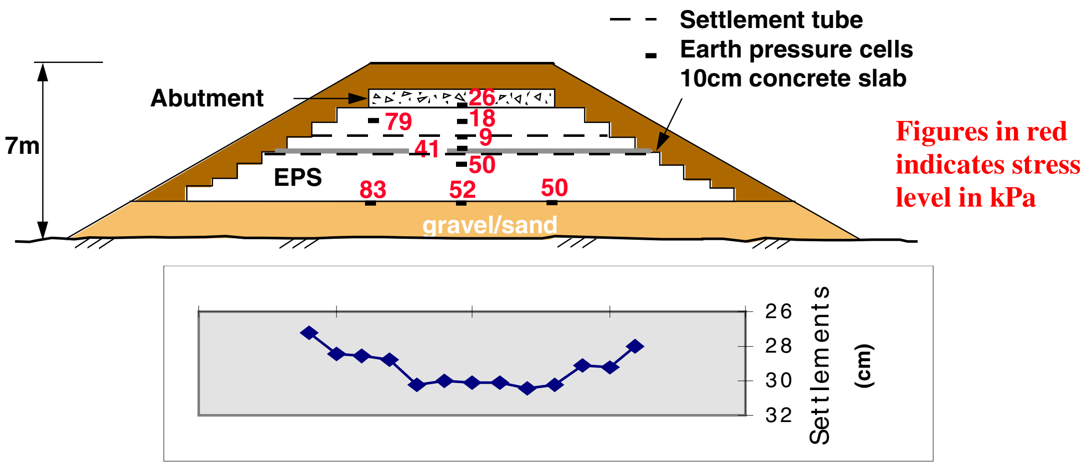

Figure 15: Løkkeberg bridge. Observed stress distribution and settlements in the cross section after 10 years in operation.

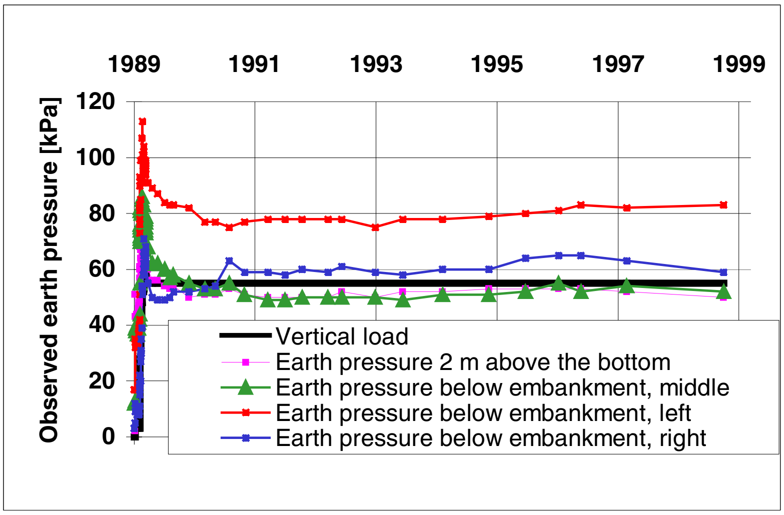

Figure 16: Long term measurements of earth pressure at Løkkeberg bridge.

Another test was performed to check the stress result. A dumper with a weight of 33 tons was placed at different distances from the abutment. In the case when it was placed directly upon the abutment an increase of 6 kPa was expected. The measured stress increases in the various fill levels with the additional load from the dumper correspond well with the stress distribution without the dumper. The increase in stress levels from the dumper is shown in Figure 17. The same tendency to reduced vertical pressure in a zone beneath the abutment was also observed here. After unloading the abutment, the pressure cells immediately returned to the initial stress levels.

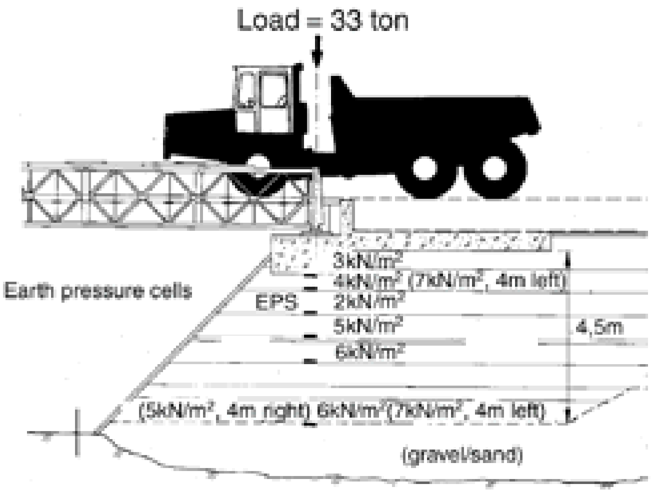

Figure 17: Stress distribution from additional load.

Attempts have been made to evaluate the stress distribution in EPS blocks based on stress observations from the test hall experiments and stress observations at the Løkkeberg bridge.

Stresses between blocks are of course difficult to measure and the results obtained vary quite a bit. In general measured stresses are, however, relatively low and indicate that the outer perimeter of the stress bulb may lie within a slope with a gradient 2:1 measured from the outer edge of the loading area. However, depending on the stiffness of the loading area and possible load eccentricities, local stress concentrations may occur. In Figure 18 the stress level at the bottom of the fill with distribution gradient 2:1 is indicated with a black line and may be compared to the stress levels of cells 2, 3, 4 and 5.

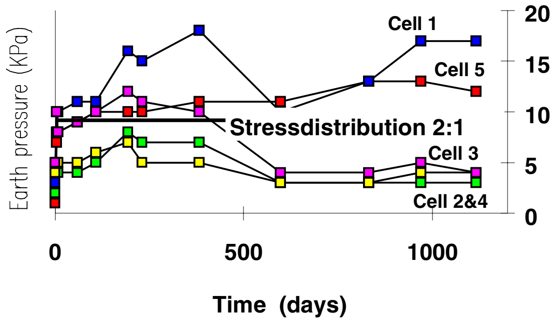

Figure 18. Stress distribution measured in the test hall (see Figure 8).

Repair of Hjelmungen bridge

Hjelmungen bridge is a three span, 54 m long continuous concrete deck bridge completed in 1992 with abutments and pillars founded on concrete piles to firm ground. The 5 m high fills adjoining the bridge consisted partly of ordinary filling materials partly of waste material from the production of Leca building blocks. The fills rested on subsoil consisting of some 10-14 m of soft sensitive marine clay, partly quick and with a high water content.

Some 2 years after completion it became evident that the bearing capacity of the soil beneath the abutments had been exceeded as excessive settlements occurred and the abutments inflicted damage to the bridge deck. Deformation monitoring was initiated and it soon became clear that immediate repair measures had to be initiated. Since settlement caused by the approach fills was the main problem, it was decided to reduce the load on the subsoil by some 30-40 kN/m^2 in order to re-establish the initial subsoil stress conditions. This involved replacing parts of the fills with EPS blocks and supporting new bridge abutments directly on the EPS. The repair design is indicated in Figure 19.

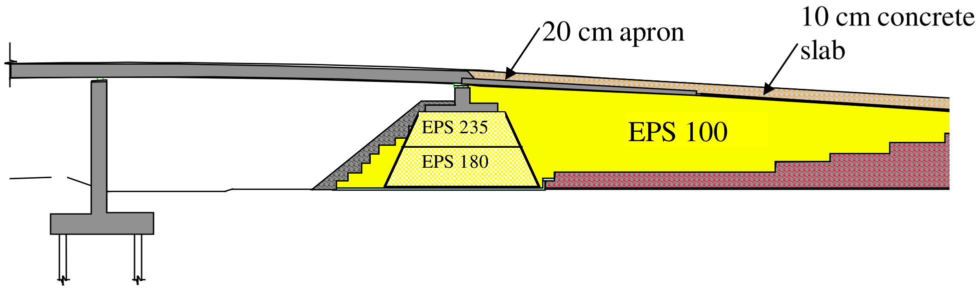

Figure 19. Supporting bridge abutments directly on EPS, Hjelmungen bridge, Norway.

Repair works were initiated in December 1995 and completed in the spring of 1996. One abutment was treated at a time while the bridge deck was provided with a temporary support as shown in Figure 20. Thickness and densities of the original filling materials were recorded as they were removed in order to have accurate data for control of load and settlement calculations. After removing the old abutments, the concrete piles were inspected regarding possible damage before being cut at ground level. No pile damage was observed. Construction of the EPS fills could then start. In this case three different qualities of EPS were utilized. In the zone directly beneath the bridge abutment, as indicated by the trapezoidal shaped lines in Figure 19, a material quality of σ =235 kPa was specified for the first three block layers beneath the bottom slab of the abutment. Further down a material quality of σ =180 kPa was specified within the indicated zone. For the rest of the EPS fill a material quality of σ =100 kPa was used. These quality requirements have been decided based on evaluation of stress distribution in the material in order to keep the stress level for dead loads within 30 % of the material strength. Stricter geometric requirements than normal were also enforced related to block dimensions in order to obtain an even fill and reduce initial deformations when the load from the bridge deck was transferred to the new abutment.

Behind both abutments a 10 m long and 200 mm thick concrete apron was specified to be cast above the EPS fill as a friction plate in order to take up horizontal forces on the abutment. On the rest of the EPS fill a concrete slab of 100 mm thickness was specified. To complete the road pavement 400 mm of pavement material was placed on top of the concrete slab.

Figure 20: Temporary support of abutments at Hjelmungen.

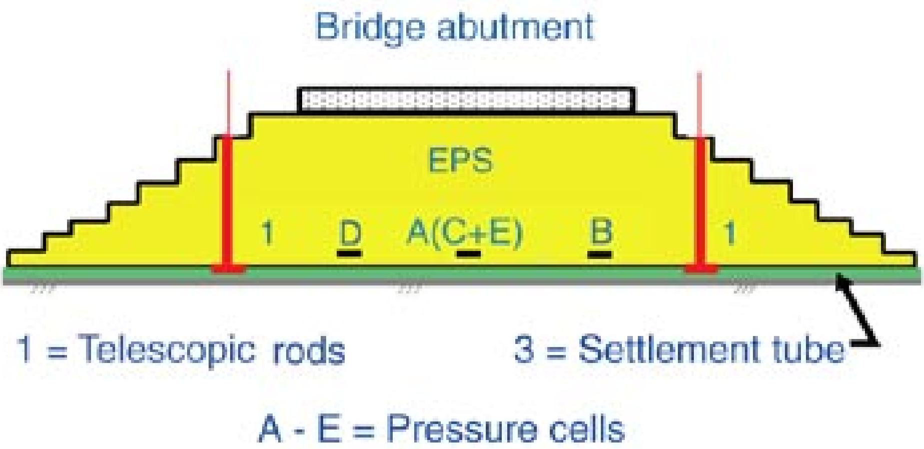

Figure 21: Cross section indicating location of monitoring equipment.

In order to monitor the behaviour of the reconstructed bridge both settlement and stress gauges have been installed. The different types of gauges and their locations in relation to the bridge abutment are indicated on the cross section in Figure 21. Prior to reconstruction the settlement rates of the adjoining fills were observed to be 100 mm/year and constant. After reconstruction the settlements have nearly been halted as shown on Figure 22.

Observed stresses beneath the EPS fill indicate a higher stress under the central part of the abutment than under the edges as shown in Figure 23. Calculated loads on the abutment are indicated by the heavy line drawn in the diagram. Problems associated with providing enough lifting force when jacking up the bridge deck may, however, indicate that reaction forces from the bridge deck are somewhat higher than calculated.

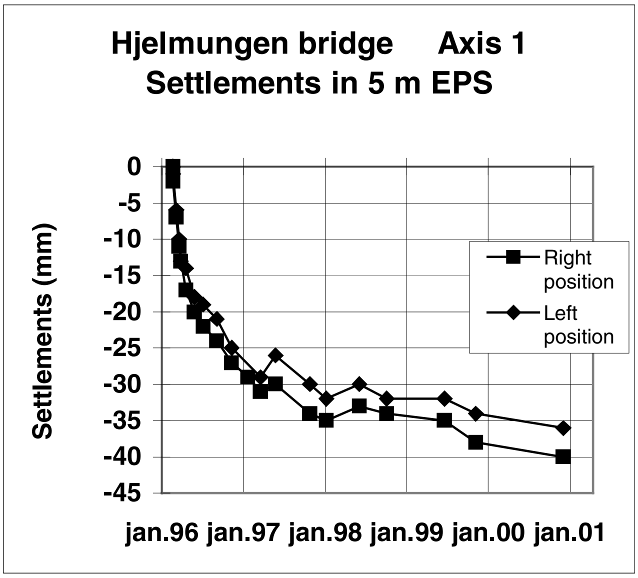

Figure 22: Measured settlements at Hjelmungen.

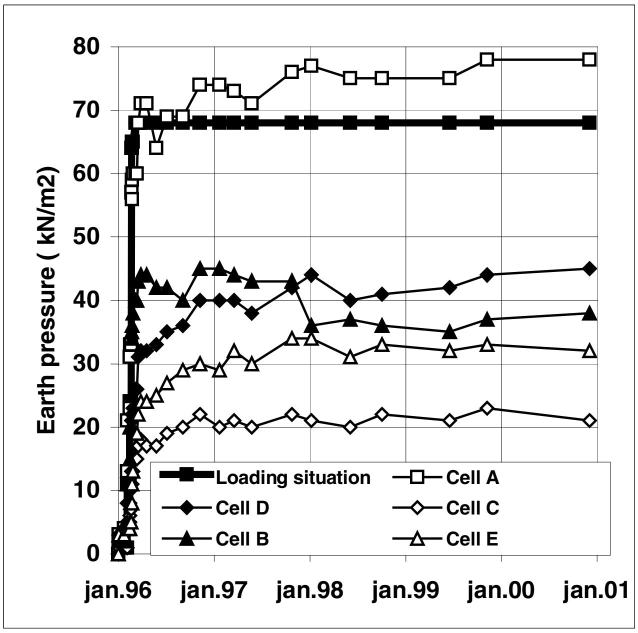

Figure 23: Earth pressure below EPS layer at Hjelmungen.

7 Durability

Although polystyrene is a stable chemical compound it may dissolve when exposed to petrol agents. When placed in a road fill the EPS blocks are therefore protected either by a concrete slab on top of the blocks also serving as a load distributing layer, or a high density polypropylene sheet. Although it is possible that a petrol tanker may overturn and spill petrol on the road surface at the location of an EPS fill, the statistical likelihood that such an event should occur is very small and the precautions mentioned above should be sufficient to protect the EPS. Also it will take some time for the petrol fluid to percolate through the soil on the side slopes, allowing time for corrective measures. Still, if a petrol spill should find its way to the EPS, only the outer blocks are likely to be effected and repair should be easy to perform. During the nearly 30 years that have passed since the first EPS fill was placed and the accelerated use of the EPS method on a worldwide scale in later years, no such spill incident has been reported.

Failures

Of the many EPS projects now completed in many parts of the world, only five known failures have been reported. Two failures are associated with water fluctuations and buoyancy forces. The other three are caused by fires.

On the 16^{th} of October 1987 Northern Europe experienced exceptionally strong storms with high wind velocities and high rainfall intensities. Norway was also exposed to major floods, and in the Oslo area the first EPS fill built in 1972 floated off as did an adjacent section of motorway constructed some years later. What was wrong? Had the dangers of buoyancy forces not been considered? Yes, such calculations had been performed, but the highest possible flood level predicted at the design stage in 1972 was 0.85 m lower than the flood level that occurred in October 1987. So it was rainfall and flood level predictions in 1972 that were misleading.

Also the second failure reported from Thailand involved an unexpected high water level causing a completed road fill to be washed away. So it should be duly noted that the dangers of buoyancy forces should be carefully studied when considering the design of an EPS fill. Often soft subsoils are located in lowland areas subjected to flooding. In such cases accurate predictions of the highest possible water level are essential in order to obtain a safe and lasting road structure.

Ordinary polystyrene is a combustible material and will burn when set on fire. For this reason some precautions should be taken when constructing EPS fills using normal quality material. Such precautions may include fencing in any stockpiles at the construction site and provide guards round the clock, or place the blocks directly in the fill when they arrive on site, working round the clock if necessary. Alternatively a self-extinguishing quality of EPS may be used at approximately 5 % increase in production costs. However, once the EPS is covered by the pavement material on top, and soil on the slopes, there will not be sufficient oxygen available to sustain a fire.

Two failures due to fires have occurred in Norway, and both were caused by welding activities on bridge abutments adjacent to EPS fills, during the construction phase. In the first case 1500 m^3 of EPS were transformed into black smoke in a matter of some 10 minutes. The concrete bridge abutment was also damaged due to the heat developed with concrete spalding from the reinforcing bars. Since the fire was initiated by sparks from welding activities on the bridge, the contractor responsible for the welding had both to repair the bridge abutment and replace the EPS fill at his own expense. A similar incident occurred in 1995 and again the repair costs had to be covered by the contractor responsible for the welding activities. In a third case a stockpile at a construction site was set on fire, probably due to children playing with matches. So the fire potential should not be overlooked and in some counties in Norway the local highway offices are only using self-extinguishing material at a somewhat higher cost in order to exclude fire hazards.

8 Conclusions

From the observations discussed above it may be fair to conclude that no deficiency effects are to be expected from EPS fills placed in the ground for a normal life cycle of 100 years. This should hold true provided possible buoyancy forces resulting from fluctuating water levels are properly accounted for, the blocks are properly protected from accidental spills of dissolving agents and the applied stress level from dead loads is kept below 30-50 % of the material strength. The observed performance of the many projects designed and constructed on these principles around the world so far supports this conclusion.

9 References

Frydenlund T. E. and Aabøe R.: *Expanded Polystyrene The Light Solution*. International symposium on EPS construction method. Tokyo 1996.

Aabøe, R.: *Deformasjonsegenskaper og spenningsforhold i fyllinger av EPS* (Deformation characteristics and stress conditions in fills of EPS). Intern rapport nr.1645. Public Roads Administration, Road Technology Department, Oslo1993.

Skuggedal H. and Aabøe R.: *Temporary overpass bridge founded on expanded polystyrene.* In: Proceedings XECSMFE, Florence May 1991, Volume 2.

Magnan & Serratrice: Propriétés méchaniques du polystyréne expansé pour ses applications en remblai routier. Bulletin LCPC 1989, No. 164, p.25-31.

Comparison of existing EPS-block geofoam creep models with field measurements

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Abstract

An estimate of the long-term vertical creep of EPS (expanded polystyrene)-block geofoam is required to predict the total vertical deformation that may occur in embankments and bridge approaches that utilize EPS-block geofoam as lightweight fill. This paper compares long-term vertical deformations from case histories with creep models that have been suggested for EPS blocks to investigate the accuracy of existing creep models. These comparisons show that current creep models do not provide reliable estimates of creep effects and are used to present other techniques for estimating the long-term creep strains of EPS-block geofoam in lightweight fill applications. Recommendations for future study of time-dependent stress-strain behavior of EPS block are also presented.

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1 Introduction

Two time-dependent stress-strain (creep) models that have been suggested for predicting the vertical strain or deformation of EPS (expanded polystyrene) blocks that occurs under an applied stress include the general power-law equation and the Findley equation. An initial overview of the theory and application of both equations is presented. The total vertical strain predicted by these two equations consist of two components as shown below.

$$\varepsilon = \varepsilon_o + \varepsilon_c \tag{1}$$

where ε = total strain after time t after application of the stress
ε_o = immediate strain upon stress application
ε_c = time-dependent strain (creep) after time t after application of the stress.

Based on the assumption that ε_o is linear-elastic and based on empirical relationships established through laboratory creep-test data, the Laboratoire Ponts et Chaussess (LCPC) derived the following General Power-Law equation for the total strain of EPS blocks (Horvath 1998; Magnan and Serratrice 1989):

$$\varepsilon = \left(\frac{\sigma}{E_{ti}}\right) + 0.00209\left(\frac{\sigma}{\sigma_p}\right)^{2.47}\left(t^{\left\{-0.9\log_{10}\left[1-\left(\frac{\sigma}{\sigma_p}\right)\right]\right\}}\right) \tag{2}$$

where ε = total strain at some time period t after stress application (in decimal form, not as percent),
σ = applied stress in kPa,
σ_p = plastic stress of EPS in kPa, which is defined as the stress corresponding to the onset of yielding (Horvath 1995),
E_{ti} = initial tangent modulus in kPa, which is defined as the average slope of the compressive stress-strain relationship at a strain between 0 and 1%, and
t = time in hours after stress application.

The LCPC established the following two empirical relationships based on laboratory testing to facilitate use of Equation (2):

$$\sigma_p = 6.41\rho - 35.2 \tag{3}$$
$$E_{ti} = 479\rho - 2875 \tag{4}$$

where σ_p = plastic stress in kPa,
E_{ti} = initial tangent modulus in kPa, and
ρ = EPS-block geofoam density in kg/m^3.

However, during this study it was found that Equation (4) yields values of initial tangent modulus that are higher than typically reported in the literature. The consequence of using Equation (4) to estimate the initial tangent modulus is discussed subsequently. The following relationship, based on averaging other published relationships by Horvath (1995) can also be used to estimate E_{ti}:

$$E_{ti} = 450\,\rho - 3000. \tag{5}$$

The Findley equation (Findley 1960; Findley and Khosla 1956) is also used to predict the total time-dependent vertical strain of geofoam. The Findley equation has been modified by (Horvath 1998) based on creep test results that extend for nearly 19000 hours (2.2 years) as shown below:

$$\varepsilon = 1.1\sinh\left(\frac{\sigma}{54.2}\right) + 0.0305\sinh\left(\frac{\sigma}{33.0}\right)(t)^{0.20} \tag{6}$$

where ε = total strain at some time t after a stress application (in percent)
σ = applied stress in kPa
t = time in hours after stress application.

Equation (6) is based on three tests performed on 50 mm cube-shaped EPS specimens with a density of 20 kg/m^3 at stresses of 30, 40 and 50 kPa. Therefore, the modified Findley equation, i.e., Equation (6), is applicable to EPS block with a density of 20 kg/m^3 subjected to stresses between 30 and 50 kPa. The applicability of Equation (6) at stress levels not between 30 and 50 kPa is investigated herein to determine the potential benefit of refining Equation (6) so that it can be used for other stress levels.

Both the general power-law and modified Findley equations will be compared with laboratory measured results on full-size EPS blocks to assess their accuracy.

2 Laboratory creep tests

A review of published creep test results (Duskov 1998; Horvath 1998; Magnan and Serratrice 1989; Negussey and Jahanandish 1993; Public Works Research Institute 1992; Sun 1997; van Dorp 1988; Wu 1996; Zou and Leo 1998) for this study revealed a lack of a standard test method for geofoam. Therefore, a qualitative, not quantitative, comparison is made between published laboratory creep test results and the calculated strain values derived from the general power-law and modified Findley equations to assess the accuracy of these equations.

It is recommended that a standard test method be developed for performing creep tests on EPS-block geofoam so creep models can be developed and reliably evaluated. The primary variables that need to be considered for creep tests are: test specimen shape, test specimen dimensions, test specimen age at the start of testing, confinement of the test specimen, test duration, applied stress level, and ambient temperature in the laboratory where the test is performed (Stark et al. 2000). Specimen shapes that have been reported in the literature include a cube, right-circular cylindrical, and disc. Cube-shaped specimens are typically 50 mm cubes. Right-circular cylindrical specimens with heights of 38, 50, 200 and 300 mm and diameters of 76, 50, 100 and 150 mm, respectively, have been utilized. Disc-shaped specimens typically replicate the dimensions of oedometer (one-dimensional consolidation) test specimens of soil (i.e. 25 mm thick and 65 mm ± in diameter). Figure 1 shows creep test results from three different specimen sizes with a density of 20 kg/m^3 tested at stresses of 20 kPa. These results as well as comparisons made from specimens tested at stresses of 30 and 50 kPa indicate that disc shaped specimens may yield higher creep strains than cylindrical specimens.

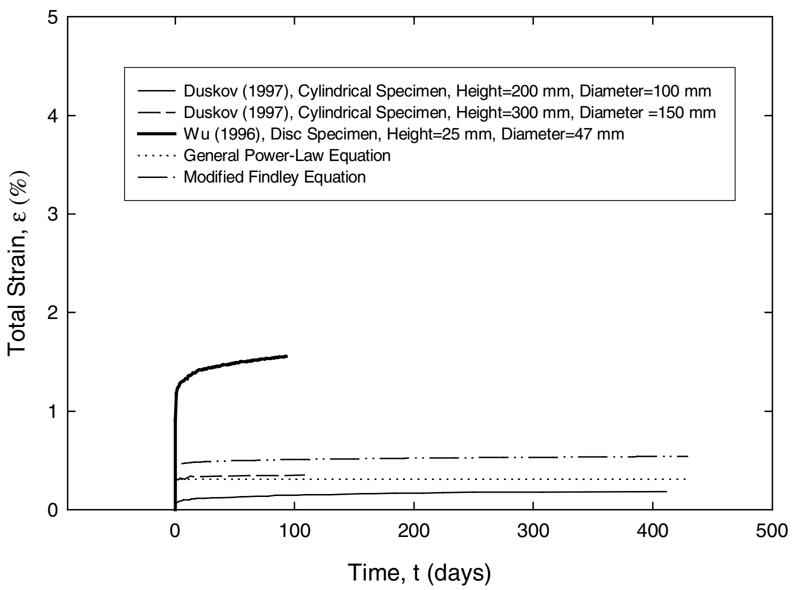

Figure 1: Comparison of Laboratory Compression Creep Test Data for an EPS Density of 20 kg/m^3 and Applied Stress of 20 kPa and Calculated Values.

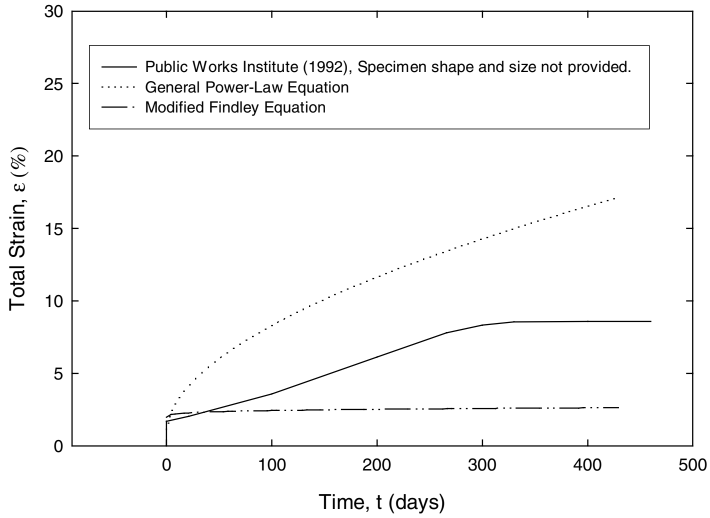

Figure 2: Comparison of laboratory compression creep test data for an EPS density of 20 kg/m^3 and applied stress of 70 kPa and calculated values.

Figures 1 and 2 provide a qualitative comparison between various size EPS specimens with a density of 20 kg/m^3 at stresses of 20 kPa and 70 kPa and the calculated results based on the general power-law and the modified Findley equations. The laboratory test results shown in these figures are limited to specimens with a density of 20 kg/m^3 and to stress levels of 20 kPa and 70 kPa because this is the density and stress range of EPS blocks that are used in the full-size block and full-scale model tests. Laboratory test data utilized in deriving the general power-law and modified Findley equations are not shown to provide non-bias comparisons. At the lower stress level of 20 kPa, both equations predict strains that are in agreement with the measured values from cylindrical EPS specimens. However, the modified Findley equation predicts slightly larger strains than the general power-law equation. Neither equation predicts strains near the measured values obtained on a disc-shaped specimen. A disc-shaped specimen is usually used when creep testing is performed with an oedometer, which is typically used to simulate one-dimensional compression of soils in the laboratory. At the higher stress level of 70 kPa (Figure 2), the power-law equation predicts and the modified Findley equation predicts larger and smaller total strains, respectively, than the measured values.

The general power-law equation indicates a relationship between the time-dependent behavior of EPS and the plastic stress and initial tangent modulus, see Equation (2). Therefore, it is recommended that compressive strength tests be performed on similar specimens that will be used for creep testing so values of plastic stress and initial tangent modulus can be obtained from the same test sample. It is also recommended that the elastic-limit stress be determined from compressive strength tests because, as will be discussed later, the elastic-limit stress may be a useful guide for estimating the onset of significant creep effects (Horvath 1995). The elastic-limit stress is defined as the measured compressive normal stress at a compressive normal strain of 1% (Horvath 1995). It is also recommended that axial strain data be obtained immediately upon stress application and frequently for the first hour after load application to better estimate the immediate strain, ε_o, (Horvath 1998). A good estimate of ε_o is critical to estimating the total strain because ε_o contributes more to the total strain than the creep-induced strain, ε_c.

3 Full-size EPS block creep test

A full-size block with a density of 20 kg/m^3 and dimensions of 1.5 m by 1 m by 0.5 m was loaded under a stress of 71 kPa for 61 days (Aabøe 1993). A stress of 27 kPa was initially applied for four days. An additional stress of 19 kPa (total stress equal to 46 kPa) was applied for seven days and an additional stress of 25 kPa (total stress equal to 71 kPa) was applied for 50 days. The stress at the bottom of the block was measured using seven pressure cells and an average pressure of 34, 55 and 79 kPa was measured in the pressure cells for days 1 through 5, 5 through 12, and 12 through 62, respectively. These average stresses are used in calculating the vertical strains using the power-law and modified Findley equations.

Figure 3 shows a comparison between the calculated and measured total strains for compressive stresses of 34, 55 and 79 kPa. At the initial stress levels of 34 and 55 kPa, both the general power-law and modified Findley equations predict total strains that are in agreement with the measured strains. At the largest stress of 79 kPa, the power-law equation significantly overestimates the measured strains and the modified Findley equation underestimates the measured strains. However, the modified Findley equation provides the best agreement with the measured values especially as the time, t, increases.

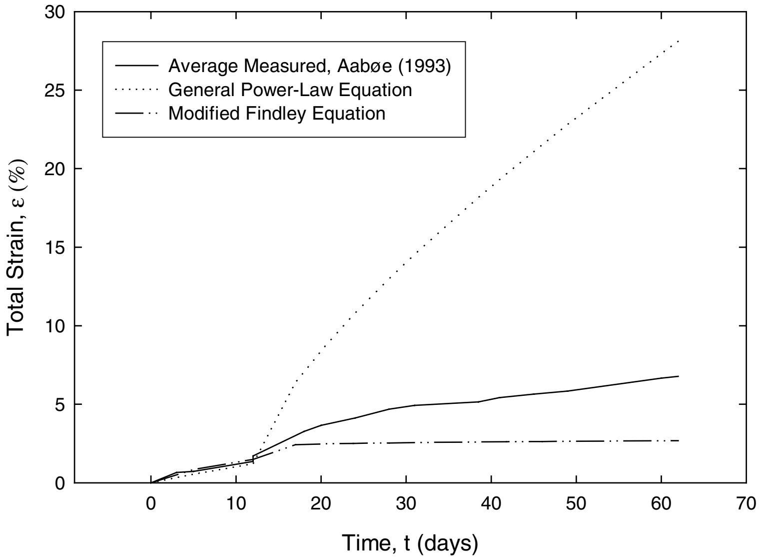

Figure 3: Comparison of full-size EPS block creep test data and the creep equations for an EPS density of 20 kg/m^3 and an applied stress of 34 kPa for days 1-5, 55 kPa for days 5-12, and 79 kPa for days 12-62.

4 Full-scale model creep test

A full-scale model creep test was performed at the Norwegian Road Research Laboratory (Aabøe 1993; Aabøe 2000) to investigate the time-dependent performance of EPS-block geofoam. The test fill had a height of 2 m and measured 4 m by 4 m in plan at the bottom of the fill decreasing in area with height approximately at a ratio of 2 (horizontal) to 1 (vertical) to about 2 m by 2 m at the top of the fill. A load of 105 kN was applied through a 2 m by 1 m plate at the top of the fill resulting in an applied stress of 52.5 kPa. The fill consisted of four layers of full-size EPS blocks with dimensions 1.5 m by 1 m by 0.5 m and densities of 20 kg/m^3.

The stress at the bottom of the fill was measured using four pressure cells. An average pressure of 7.8 kPa was measured in the pressure cells during the 1270 day test. Based on this average pressure measured at the bottom of the test fill and the stress of 52.5 kPa applied at the top of the fill, the stress distribution within the EPS fill was approximately 1 (horizontal) to 1.8 (vertical). This is in agreement with a stress distribution of 1 (horizontal) to 2 (vertical), which is typically assumed in design calculations incorporating EPS-block geofoam structures. The measured stress distribution is slightly wider but still in agreement with 1 (horizontal) to 2 (vertical). Thus, the measured stress with depth is slightly less than the typically assumed stress distribution, which results in a slightly conservative design. Therefore, it is recommended that the assumed 1 (horizontal) to 2 (vertical) stress distribution be utilized in design calculations for EPS-block geofoam embankments.

Figure 4 shows a comparison of the total strain measured in the EPS blocks of the full-scale test fill and the calculated total strains based on the power-law and modified Findley equations. In calculating the total strains, the fill was divided into the same number of horizontal layers as EPS block layers used, four. The total strain of each layer was determined based on the average stress calculated at the middle of each block using the measured 1 (horizontal) to 1.8 (vertical) stress distribution. Thus, the stress used for each layer from top to bottom was 36.2, 20.4, 13.1, and 9.1 kPa. As indicated in Figure 4, both the general power-law and modified Findley equations underestimate the strains measured in the full-scale test fill. The power-law predictions are lower than the modified Findley predictions and thus the Findley equation provides the best agreement.

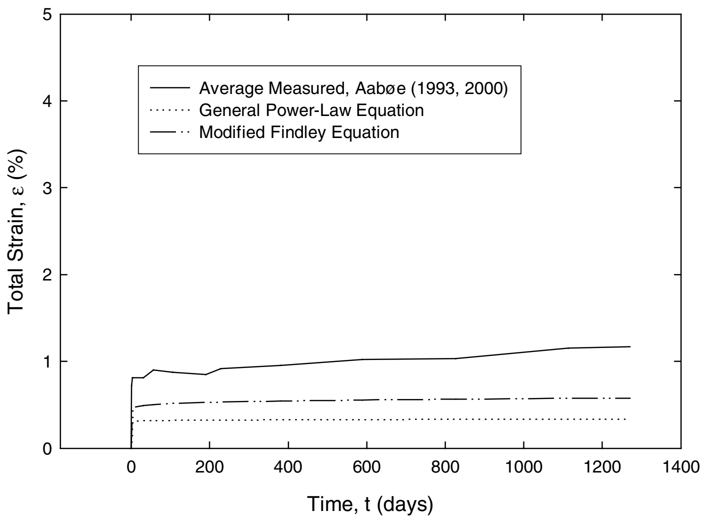

Figure 4: Comparison of full-scale model creep test data and the creep equations for an EPS density of 20 kg/m^3.

5 Full-scale field monitoring

A field monitoring program was implemented as part of the Løkkeberg bridge project built in Norway in 1989 (Aabøe 1993; Aabøe 2000). EPS blocks were used to construct a bridge approach embankment and to support the bridge foundation. Pressure cells were installed at various locations within the embankment and settlement monitoring rods were installed at four locations to measure the total settlement of the embankment and the vertical strains at various depths in the embankment. The height of the embankment is 4.5 m. EPS blocks with an unconfined compressive strength of 240, 180, and 100 kPa, were used in the top 1.2 m, middle, and bottom 2.1 m of the embankment, respectively. A 10 cm concrete slab was placed between the 180 and 100 kPa blocks to further distribute the stresses within the 100 kPa blocks.

Figure 5 shows the total vertical strain measured in the lowest block layer. The density of the bottom row of EPS blocks is 20 kg/m^3 and the original thickness of the EPS blocks is 0.6 m. Three pressure cells were installed below the first row of blocks. An average pressure of 67 kPa was recorded in the three pressure cells during the period that the vertical strain was being obtained from the settlement rods. As shown in Figure 5, the power-law and modified Findley equations significantly overestimate and underestimate the measured total strains, respectively. However, the total strains predicted by the modified Findley equation are again in better agreement with the measured values than the power-law equation.

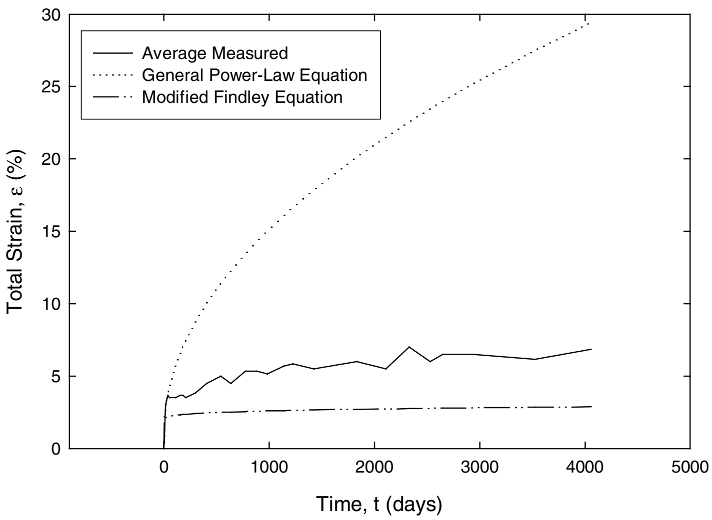

Figure 5: Comparison of total vertical strain measured in the lowest EPS block layer of the field test fill and the general power-law and modified Findley equations.

6 Summary of comparison of measured and calculated values of total strain

For stresses between 10 and 55 kPa, both the power-law and modified Findley equations yield total strain values similar to or less than the measured values obtained on the full-size block and full-scale creep test fills. In general, the power-law equation predicts total strains smaller than the modified Findley equation for compressive stresses between 10 and 55 kPa. A similar observation was made by Horvath (Horvath 1998). Horvath (Horvath 1998) suggests that the power-law equation predicts smaller total strains than laboratory measured values, especially for short time durations, because the test specimens used by the LCPC to derive the power-law equation yield larger values of initial tangent modulus than for other test specimens reported in the literature. This is apparent by comparing Equations (4) and (5). Horvath (Horvath 1998) suggests that the values of E_{ti} obtained from the LCPC relationship in Equation (4) are approximately 40 % larger than the values from Equation (5), which is based on averaging other published relationships. In summary, the modified Findley equation is recommended to predict total vertical strains for compressive stresses between 10 and 55 kPa. Further refinement of the modified Findley equation for stresses outside the 30 to 50 kPa stress range that was used in developing the equation may result in better predictions.

At larger compressive stresses of 67, 70 and 79 kPa, the total strains determined by the power-law equation and the modified Findley equation significantly overestimate and underestimate the measured full-size block and full-scale test fill values. The modified Findley equation provides better agreement than the power-law equation, especially as the elapsed time increases. Further refinement of the modified Findley equation for stresses outside 30 to 50 kPa stress range that was used in developing the equation may result in better predictions. As noted by Horvath (Horvath 1998), the power-law equation may provide unusually high strain values at large compressive stresses, especially at longer durations of applied stress, because the power-law equation was developed from creep tests of insufficient duration. This results in greater strains because the total strains decrease with increasing elapsed time as shown in Figures 1-5.

The time-dependent behavior obtained on one layer of blocks in the full-scale field test is similar to the behavior obtained during the full-size block test. After a time equal to 1440 hours (60 days), the difference in total strain measured was approximately 3.2%, with the full-size block test producing the larger total strain because the average total measured stress in the full-size block test was 79 kPa compared to 67 kPa for the full-scale field test. Therefore, it appears that creep tests based on a full-size EPS block may provide reasonable predictions of total vertical strain with time for projects utilizing EPS-block geofoam as lightweight fill. This reduces the need for constructing full-scale model test fills to develop time-dependent data. Therefore, a standard test method could be developed either using a full-size block or comparing the results from smaller specimens with the results of full-size blocks.

7 Summary

At present general power-law and modified Findley equations do not provide a reliable estimate of the time-dependent total strains. Further research is required to either refine these expressions or develop new expressions based on other creep models. In particular, the power-law equation should be refined to include results from specimens with lower values of E_{ti} and tests of longer duration. The modified Findley equation should be refined to include test results from compressive stresses outside the 30 to 50 kPa stress range that was used to develop the relationship.

The results of the full-scale model test conducted at the Norwegian Road Research Laboratory indicates that the typically assumed 1 (horizontal) to 2 (vertical) distribution of compressive stresses through a geofoam embankment is reasonable, albeit slightly conservative because the measured stress showed a stress distribution of 1 (horizontal) to 1.8 (vertical), for design calculations. A comparison made during this study indicates that the measured total strains obtained on one layer of blocks in the full-scale field test fill is similar to the strains obtained from the full-size block test. Therefore, creep tests based on a full-size block may provide reasonable predictions of total vertical strain with time for projects utilizing EPS-block geofoam as lightweight fill. Currently, there is a lack of comparable test data on small laboratory specimens and full-size blocks. It is recommended that creep tests be performed on both full-size blocks and small specimens cut from similar full-size blocks to establish a correlation between these two specimen sizes. If a correlation is established, future creep testing can be performed on small laboratory specimens instead of full-size blocks.

Published test results are not sufficient to refine the existing creep models or to develop new models because testing procedures are not standardized and sufficient information about the testing procedures are not available. It is recommended that creep testing be standardized so the necessary information for refining creep models becomes available. Recommendations on standardizing creep testing procedures are provided herein. In addition to using traditional creep testing procedures, consideration should be given to using time-temperature superposition procedures or a combination of both conventional testing procedures and time-temperature superposition procedures (stepped isothermal methods) to measure creep behavior. These alternate methods have been used to study creep behavior of other geosynthetic materials (Sandri et al. 1999) and can accelerate acquisition of meaningful creep data. The resulting creep data could be used to develop a stress-strain-time-temperature mathematical model for EPS block. Such a model would enable better predictions of creep strains at temperatures other than the conventional laboratory ambient conditions.

The current state of practice for considering creep strains in the design of EPS block embankments and bridge approaches is to base the design on laboratory creep tests on small specimens cut from the same type of EPS block that will be used in construction or to base the design on published observations of the creep behavior of EPS, such as:

- If the applied stress produces an immediate strain of 0.5% or less, the creep strains, ε_c, will be negligible even when projected for 50 years or more. The stress level at 0.5% strain corresponds to approximately 25% of the compressive strength at a compressive normal strain of 1% or 33% of the yield stress.
- If the applied stress produces an immediate strain between 0.5% and 1%, the geofoam creep strains will be tolerable (less than 1%) in lightweight fill applications even when projected for 50 years or more. The stress level at 1% strain corresponds to approximately 50% of the compressive strength or 67% of the yield stress.
- If the applied stress produces an immediate strain greater than 1%, creep strains can rapidly increase and become excessive for lightweight fill geofoam applications. The stress level for significant creep strain corresponds to the yield stress which is approximately 75% of the compressive strength.

In summary, the compressive stress at a vertical strain of 1%, i.e., the elastic-limit stress, appears to correspond to a threshold stress level for the development of significant creep effects and the field applied stresses should not exceed the elastic-limit stress until more reliable creep models are developed (Horvath 1995). Based on these observations, it is concluded that creep strains within the EPS mass under sustained loads are expected to be within acceptable limits (0.5% to 1% strain over 50 to 100 years) if the applied stress is such that it produces an immediate strain between 0.5% and 1% (Horvath 1995; Stark et al. 2000).

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9 References

Aabøe, R.: *Deformasjonsegenskaper og spenningsforhold i fyllinger av EPS* (Deformation and stress conditions in fills of EPS). Intern Rapport Nr. 1645, Public Roads Administration. Oslo 1993.

Aabøe, R.: *Long-term performance and durability of EPS as a lightweight fill.* Nordic Road & Transport Research, 2000, 12(1), 4-7.

Duskov, M.: *EPS as a Light-Weight Sub-Base Material in Pavement Structures*. Thesis: Doctor of Engineering, Delft University of Technology. Delft 1998.

Findley, W. N.: *Mechanism and mechanics of creep of plastics*. SPE Journal, 1960, 16(1), 57-65.

Findley, W. N. and Khosla, G.: *An equation for tension creep of three unfilled thermoplastics.* SPE Journal, 1956, 12(12), 20-25.

Horvath, J. S.: *Geofoam Geosynthetic*, Horvath Engineering, P.C., Scarsdale, 1995.

Horvath, J. S.: *Mathematical Modeling of the Stress-Strain-Time Behavior of Geosythetics Using the Findley Equation: General Theory and Application to EPS-Block Geofoam.* CE/GE-98-3, Manhattan College, Bronx. N.Y. 1998.

Magnan, J.-P. and Serratrice, J.-F.: *Propriétés mécaniques du polystyréne expansé pour ses applications en remblai routier.* Bulletin Liaison Laboratoire Ponts et Chaussées 1989, (164), 25-31.

Negussey, D. and Jahanandish, M.: *Comparison of some engineering properties of expanded polystyrene with those of soils*. Transportation Research Record, 1993, no 1418, 43-48.

Public Works Research Institute: *Design and Construction Manual for Lightweight Fill with EPS*. The Public Works Research Institute of Ministry of Construction and Construction Project Consultants, Inc., Japan 1992.

Sandri, D., Thornton, J. S. and Sack, R.: *Measuring geosynthetic creep: three methods.* Industrial Fabrics Association International, Roseville, MN. Geotechnical Fabrics Report, 1999, v.17, no 6, p 26-29.

Stark, T. D., Horvath, J. S., Arellano, D. and Leshchinsky, D.: *Guidelines for Geofoam Applications in Embankment Projects, Interim Report*. National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C. 2000.

Sun, M. C.-W.: *Engineering Behavior of Geofoam (Expanded Polystyrene) and Lateral Pressure reduction in Substructures.* M.S., Syracuse University. Syracuse 1997.

van Dorp, T.: *Expanded Polystyrene Foam as Light Fill and Foundation Material in Road Structures, preprint paper*. The International Congress on Expanded Polystyrene: Expanded Polystyrene- Present and Future, Milan, Italy.

Wu, Y.: *An Investigation of Long-Term Deformation Behavior of EPS Block Under Static & Repeated Loads*. M.S., South Dakota School of Mines and Techology. Rapid City, SD 1996.

Zou, Y. and Leo, C. J.: *Laboratory Studies on the Engineering Properties of Expanded Polystyrene (EPS) Materials for Geotechnical Applications*. In: 2nd International Conference on Ground Improvement Techniques, 8-9 October 1998, Singapore. P. 581-588.

Use of waste materials for lightweight fills

Tor Erik Frydenlund and Roald Aabøe

Paper presented at International Workshop on Lightweight Geomaterials, Tokyo, March 2002.

ABSTRACT

In Norway the Public Roads Administration has a long tradition in applying various types of llightweight filling materials for road construction purposes. During the last 50 years wooden materials like sawdust and bark residue from the timber industry have been applied for such purposes. Also waste material from the production of cellular concrete blocks and LWA (Light Weight Clay Aggregate) have been widely used. Since 1972 blocks of Expanded Polystyrene have been used extensively in road projects for a variety of applications also including blocks produced from re-circulated EPS material. In this connection monitoring programmes have been initiated in order to investigate the long term performance of these materials. Presently a new option is being investigated involving the use of granulated foamed glass produced by re-circulating waste glass. The maximum grain size will be some 50 mm with angular edges and the material may be produced in various densities. When placed and compacted in a drained fill the unit density of the lightest version will be some 300-350 kg/m^3 depending on the compaction machinery and compaction efforts. On a recent road project deformations and possible variations in moisture content, unit density and grain size distribution will be monitored. This paper presents results from this monitoring programme.

1 Introduction

Sawdust and bark residue

The Norwegian Public Roads Administration has for many years employed various lightweight filling materials to overcome load and settlement problems in connection with road construction on soft subsoils [1], [2], [3]. Some 50 years back sawdust, a waste material from sawmills, was used as a lightweight filling material. The unit density employed for design purposes was 10 kN/m^3. Similarly when the timber industry started to strip trees of bark at centrally located barking stations in the 1960ties, vast amounts of bark residue were accumulated. In both cases this waste material could be obtained at practically no cost except for the cost of transportation from the waste dumps to the construction sites. The bark was classified according to degree of decomposition and the less decomposed material was used in road construction on secondary roads with a design density equal to saw dust, i.e. 10 kN/m^3. As long as the saw dust or bark was kept moist in a submerged position or covered by layers of clay, no appreciable decay or road subsidence were observed. However, when the timber industry started to make use of saw dust and bark as a combustible material for drying newly cut planks at the saw mills, the cost aspect changed dramatically and these types of lightweight filling materials could no longer compete with alternative material having emerged in the meantime.

Foamed concrete materials

Such materials were waste from the production of cellular concrete blocks or slabs for house building purposes, in this country sold under the trade name of Siporex or Ytong. Again being a waste product these materials could be obtained at fairly low cost. Being a rather weak material, several tests were performed regarding compressive strength and unit density including the affinity to absorb water. Also monitoring programmes were initiated testing for possible changes in particle sizes due to crushing during placement and repetitive loading in a road structure, and also changes in water content when stored in the ground for some appreciable period of time. From the result of these monitoring programmes a design unit weight for design purposes was established at 10 kN/m^3 [2]. When the demand for such materials in the building industry receded also the amount of available waste materials diminished and today such materials are seldom used.

Light Weight Clay Aggregate - LWA

Instead the focus was placed on another type of lightweight aggregate, LWA (Light Weight Clay Aggregate). By sintering clay in a special furnace, hard spheres of various sizes are formed (generally grain size 0-32 mm). These spheres are used for producing building blocks and slabs. At the start in the late 1950ties waste products from this block production were used as a lightweight filling materials, but it was also found suitable to use the LWA grains in fills directly. The unit density of such material based on monitoring programmes has been determined to be 6 kN/m^3 when positioned in a drained condition in the fill or 7 kN/m^3 when periodically submerged [2] [3]. In addition it has thermal insulation effects and may therefore also act as a frost-insulating layer if having sufficient thickness. LWA material is in common use as a lightweight filling material for road construction in Norway today and will compete with other lightweight materials like Expanded Polystyrene blocks EPS.

Expanded Polystyrene

Since 1972 when the first road EPS fill was placed in Norway, the use of blocks of EPS as a super light filling material has seen a major increase both in volume and types of application on a worldwide scale. Initially blocks were only foamed using fresh raw materials. Incorporation of recycled EPS material has also been introduced in the block production and in some cases whole blocks have been reused in another location. As the use of EPS as a filling material now has a fairly broad literature coverage internationally, most recently in the proceedings from the EPS Geofoam Conference in Salt Lake City December 2001, further information on this topic may be obtained here [4].

Other waste materials in Norway

Another waste material of some concern is used car tyres. Most countries are now accumulating appreciable amounts of used car tyres and one possible reuse of these tyres are in fills as a lightweight material. Various approaches have been made in different countries, but in general shredded tyres are most commonly used for this purpose, but in some cases whole tyres have also been applied. In Norway only a few projects have been completed using shredded tyres including a noise barrier with a pile of whole tyres at the core.

In this country attempts have also been made to produce a lightweight insulation and filling material by foaming common household garbage and industrial waste. This requires various processes like sorting, grinding, kneading, cleaning and purification prior to foaming and drying. Small scale testing is presently being performed in order to evaluate the production procedures and resulting material qualities. No further information can therefore be given at this stage.

2 Foamglass

Concept

In the western hemisphere vast heaps of glass products are also accumulating. This comprises various sorts of glass waste originating from light bulbs and other lighting fixtures like mercury lamps, bottles, windowpanes, car windshields etc. and industrial waste. At the same time as being a waste product it also constitute a raw material for possible reuse. Some of the glass waste may be used directly in the production of bottles and other products but some of the glass waste also contains toxic materials that need to be removed in the recycling process. In this connection a production process has been initiated based on the recycling of waste glass in the middle region of Norway. The resulting product is a lightweight foamglass material and it has been given the trade name of Hasopor. This product has now been used as a lightweight filling material on some road projects in Norway and the Norwegian Public Roads Administration has initiated a monitoring programme in order to evaluate the material properties and performance in this connection. Hasopor is known to have been produced in Switzerland for some time and it is also reported to have been used in road structures. Regarding material quality and behaviour in this connection only sparse information has been obtained.

In Norway about 4 million mercury lamps are used every year and the aim is to recycle about 40 % amounting to an annual production of some 50 000 m^3 of Hasopor.

Production process

Foamglass is produced using an environmentally friendly recycling technology for contaminated and toxic waste ranging from mercury lamps, industrial slag and flyash, PC- and TV-tubes, and laminated glass to batteries. The process is based on the concept of transforming finely grinded glass powder from different glass sources mixed with an activator like silica carbide into glassfoam. In the grinding process heavy metals are separated out and recycled to metal melting plants.

The powder is spread on a steel belt conveyor running through high temperature ovens whereby the powder expands above 4 times, to leave the oven as solid glass foam material. When the product leaves the oven it will crack and separate into smaller units due to the temperature shock. Normal grain size will be in the range of 10-60 mm. Figure 1 shows a typical particle.

The production process is free of dust and any harmful gases and does not need water at any stage.

The principles behind this system is very simple:

- To separate, and
- Clean the waste in fractions for further treatment down the process line.

During this process the toxic components are reduced below the detecting limits. In this connection a certificate has been obtained for the material confirming that possible leaching products from a fill will have toxic contents well below normal environmental requirements (see Figure 2.).

Figure1: Typical foamglass particle.

Foamglass generally consists of 8 per cent of glass by volume and 92 per cent gas bubbles. A thin impervious glass wall encloses each bobble. Material qualities listed by the producer is as follows

- Low unit bulk density, the product is delivered in two qualities: 180 kg/m^3 and 225 kg/m^3
- High thermal insulation qualities
- High material strength, 60-120 kN/m^2
- Low moisture absorption
- Chemically and thermally stable.

These material qualities have been measured in laboratory tests performed by certified laboratories.

The production process may be visualised as indicated in Figure 2.

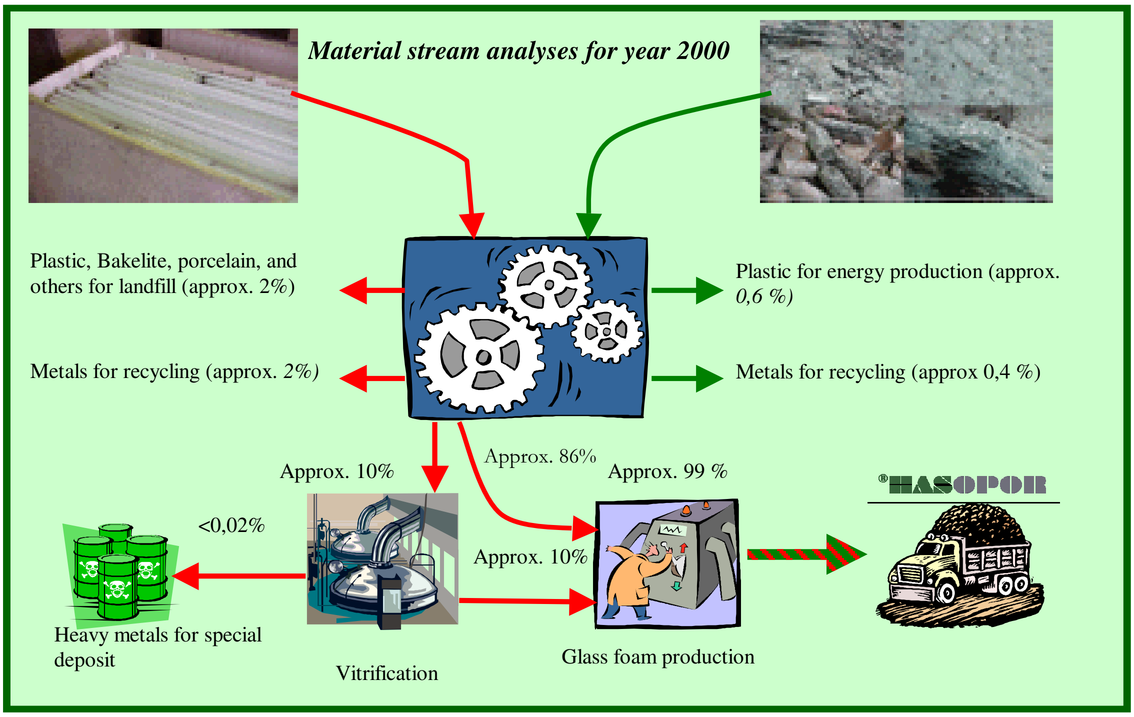

Figure 2: Recycling process.

3 Use in road projects

National road No. 17 - repair of slope failure

Due to erosion along the riverbed a 10 m high road slope failed over a distance of some 30 m. At the river level and in the slope the subsoil consisted of quick clay of medium strength. In order to re-open the road and prevent further erosion along the riverbank and more slides, it was necessary to quickly implement repair measures. Erosion protection was established using blasted rock before filling the slide area with foamglass of the Hasopor type. The material was delivered on site by large trucks and side tipped into slide area. Volumes of up to 100 m^3 could be transported in one haul. The foamglass was then distributed and placed in layers of 0.5 m thickness by a 30 tonnes crawler mounted excavator and compacted by 3-4 passes by the crawler belts giving a compaction ratio of 1.4. In this way a total volume of 300 m^3 was placed within the time span of 4 hours, see Figure 3.

Figure 3: Placing of foamglass with excavator.

The composition and thickness of the road pavement placed on top of the foamglass fill was equivalent to the pavement on the adjoining road sections.

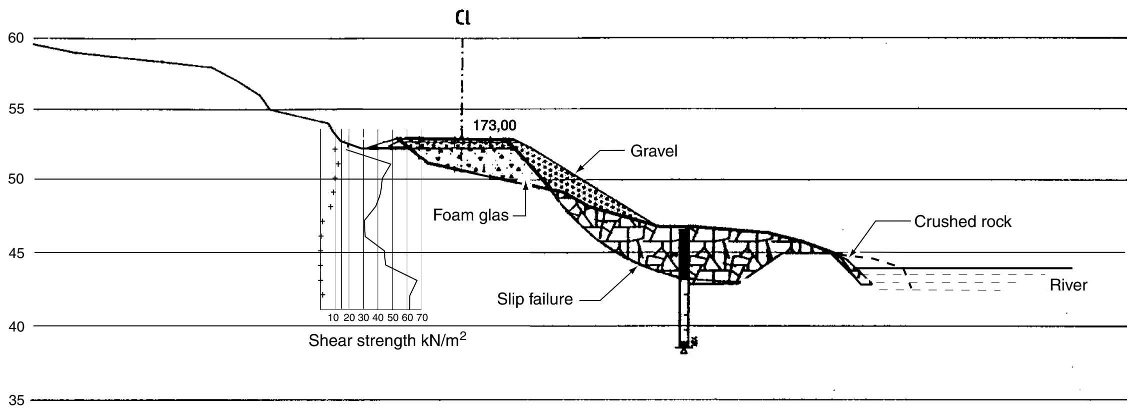

Figure 4: Cross section showing repair measures in the slide area.

The slide occurred not far from the processing plant for foamglass in the middle region of Norway and at the time in question foamglass had already been used in pilot projects for testing its thermal properties as a frost insulating layer in road structures and other road related properties. Based on the preliminary experiences from this project a decision to use foamglass to repair the slide area could quickly be made. Due to the short construction period no on site density tests were performed other than estimating the compaction ratio based on the delivered volume divided by the theoretical volume.

Experience gained form this project was that foamglass was easy to handle and therefore resulting in a very short reconstruction period. The internal stability was also satisfactory on sloping subsolil surfaces. The cost of foamglass delivered on site was \$ 35 per m^3 in 1999.

Visual observations 2 years after the slope was re-established indicate that the road is performing well without any pavement cracks or settlement. Test pits are planned for checking grain size distribution and density in the future.

Pedestrian/cycle path in Lodalen

In connection with the construction of a pedestrian/cycle path on a slope with low stability against failure, foamglass was used to construct the fill. The lightest quality of foamglass was employed, i.e. $\rho = 180$ kg/m^3. Since no additional load could be placed on the slope it was decided to use a fully compensated solution by replacing some of the natural subsoil with foamglass. Totally a volume of 1200 m^3 foamglass was used.

The foamglass was placed in layer thickness up to 2 m. The top surface was levelled off by the excavator. Compaction was performed by the crawler belts of the excavator, see Figure 5. On the slopes light compaction was performed with the excavator bucket. The natural slope angel for uncompacted foamglass seems to be 45°.

Figure 5: Distributing foamglass material using a crawler mounted excavator (Hitachi EX60, 8 tonnes). For internal transport on site a shovel dozer (CAT 923E) was used.

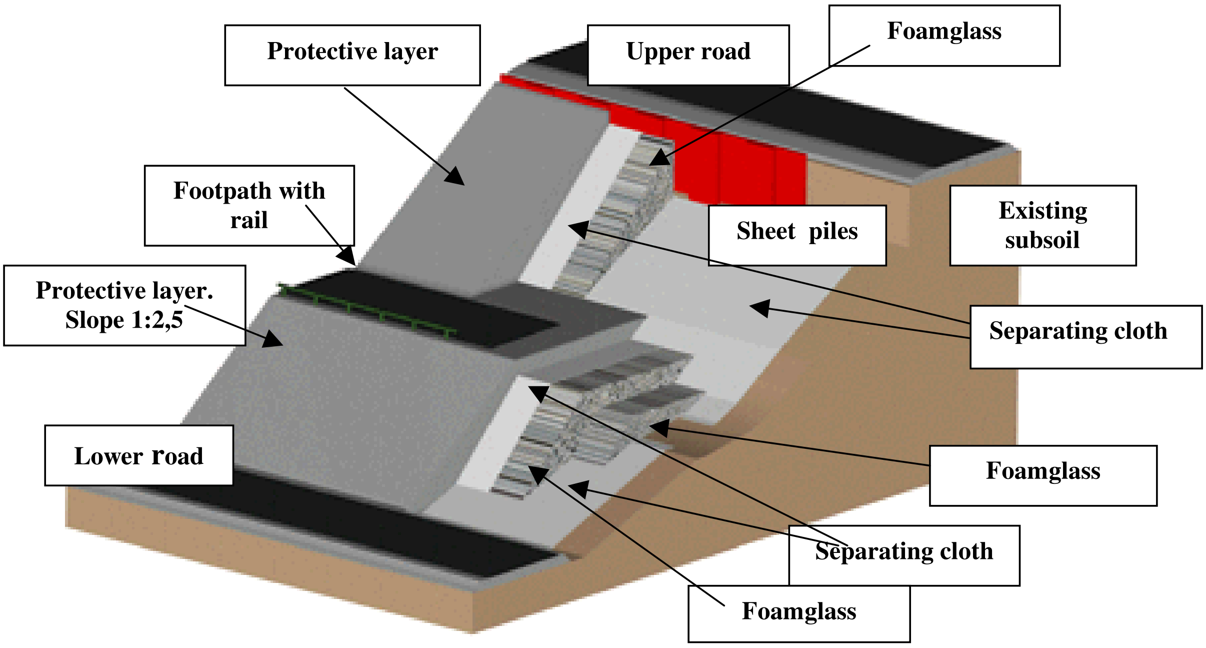

Figure 6: Isometric view of foamglass fill at Lodalen.

For layer thickness less than 2 m compaction may be performed after placing the road base. The thickness of the roadbase layer should then be at least 20 cm.

Field observations

In order to monitor deformations in the material, settlement tubes have been installed at the bottom and at the top of the foamglass layer at two sections, see Figure 7 and 8.

The deformations of the foamglass layer may then be evaluated from the settlement differences of the tubes, see Figure 8.

According to the measurements shown in Figure 8 the foamglass layer deformed about 12 cm after the roadbase materials were placed. It is important to note that the deformations in the outer area occurred immediately after the foamglass layer was loaded.

Figure 7: Monitoring long term deformations with tube settlement gauges.

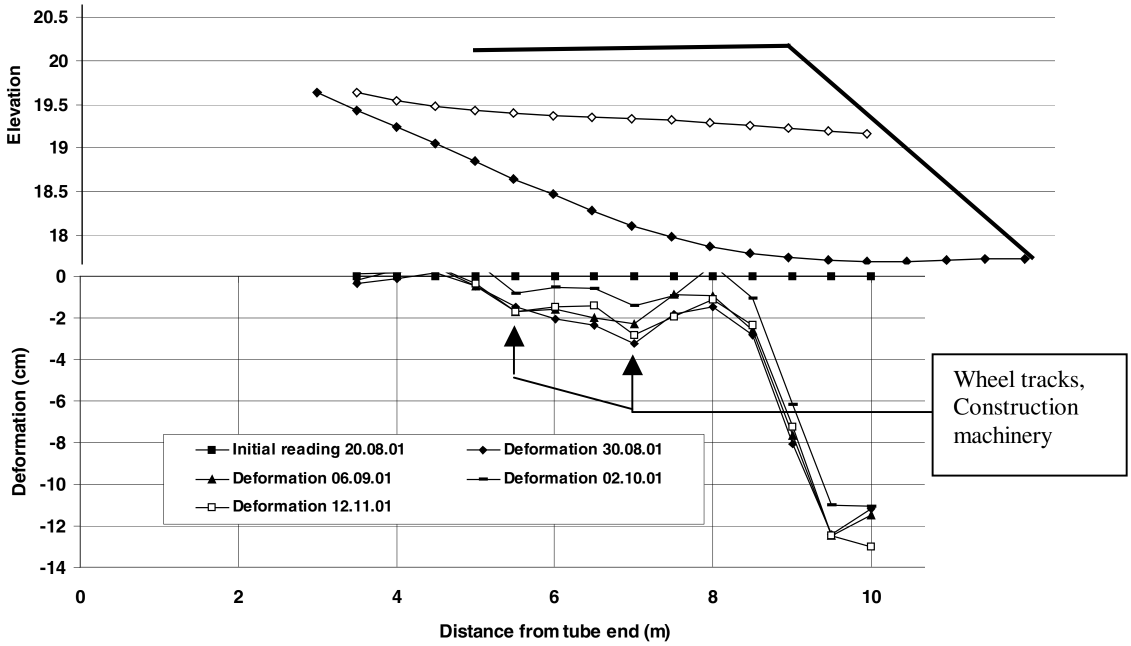

Figure 8: Deformations measured in foamglass fill at Lodalen.

Design density

The design unit density used for foamglass in this fill was calculated to be $\rho = 300$ kg/m^3 based on the following criteria:

- ► Dry density (light quality) 200 kg/m^3
- ► Compaction factor 1.3
- ► Increase due to moisture 15 kg/m^3
- ► Material factor $\gamma_m = 1.1$.

At two locations a thin walled steel tube with diameter 570 mm was pressed 270 mm into the foamglass fill until the top was level with the top of the foamglass layer, see Figure 9. One test site was located in the open and the other underneath a bridge crossing over the foot/cycle-path. The particles contained within the steel tube was removed and the excavated material weighed wet and dry. Also the void left in the tube was lined with a thin plastic sheet and filled with water from a measuring glass in order to determine the excavated volume, see Figure 10. Just before the samples were taken, heavy rain occurred.

Figure 9: Steel tube with material removed.

Figure 10: Water used to measure excavated volume.

Results from the two test sites are as follows:
Test site 1: Measured density $\rho = 384$ kg / m^3 (w = 15.2 %)
Test site 2: Measured density $\rho = 294$ kg / m^3 (w = 2.8 %).

The test shows that the amount of accumulated water is somewhat higher than expected. Test site 2 was located under the bridge and this may explain the difference in water content. Also the material was compacted more than anticipated and the resulting density tend to be somewhat above the design density.

This site will be followed up with more tests over time in order to monitor changes in deformations, water content and density.

Embankment on National highway No 120

In an area some 50 km north of Oslo with subsoils consisting of soft sensitive clay it was decided to use foamglass as a lightweight filling material in competition with LWA. The cross section of the road is in cut on one side and fill on the other. The height of the fill is up to 4 m. No appreciable settlements are expected, but lightweight filling materials were selected due to stability concerns.

Figure 11: Transport to site in large trucks.

A total volume of 3950 m^3 of Hasopor type foamglass, light version with unit density 180 kg/m^3 was used in layer thickness up to 3 m. The following procedures were adopted in co-operation with the contractor:

- Placing of separation cloth on subsoil
- Transport to site by large trucks and internal transport by dumper, see Figure 11
- Distribution and compaction with crawler mounted dozer, (belt pressure 56 kN/m^2, belt width 61 cm), see Figure 12
- Placement in min 1 m layers. Compaction from min 3 passes by dozer
- Separation cloth used on fill sides and top
- Road sub-base and base placed on top of separating cloth.

Field observations

As in the previous case at Lodalen settlement tubes have been installed at the bottom and top of the foamglass layer at two locations in order to monitor possible deformations in the material. The results from measurements taken so far are shown in Figure 13.

Also measurements of density and water content was performed in the same way as described for the Lodalen site. The accuracy of this testing method is believed to be fairly good. Errors may occur if the plastic sheet bridges some of the voids within the steel tube and possibly some compaction may occur when pressing the tube into the material. The overall error is, however, not expected to exceed 1-2 %.

Two test sites were selected, one underneath the tracks of the transport trucks and other traffic on the fill while the other was located towards the edge of the fill less influenced by traffic other than for compaction purposes.

Figure 12: Distribution and compaction with dozer.

Results from the two test sites are as follows:
Test site 1: Measured density $\rho = 653\ kg / m^3$ (w = 17.3 %)
Test site 2: Measured density $\rho = 429\ kg / m^3$ (w = 17.0 %).

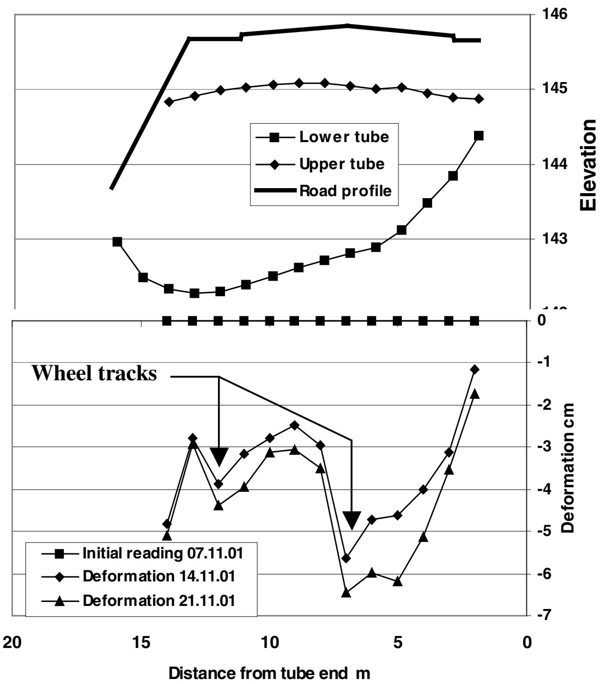

Figure 13: Deformations measured in foamglass fill on National road No. 120.

Again a higher water content than expected was found and also a considerably higher unit density than anticipated was measured.

Test pits were also excavated below wheel tracks for visual inspection. In this connection clear indications of major crushing were detected down to a depth of 700 mm below the surface. Outside the wheel tracks and in areas with little on site traffic substantially less crushing was observed.

During construction heavy machinery was used to a great extent for placing and compacting the foamglass and the material was distributed in thin layers. In addition the fill was also used as an access road for the construction site in general. A compaction factor of 1.37 is calculated by the contractor based on actual volume of foamglass delivered on site compared to the theoretical volume.

The results of sieving tests performed on excavated material are shown in Figure 14. Here it may be observed that the grain size distribution curves show considerable amount of fines below the typical particle size 10-60 mm that results from the production process.

This site will also be followed up with more tests over time in order to monitor possible changes in deformations, water content and density.

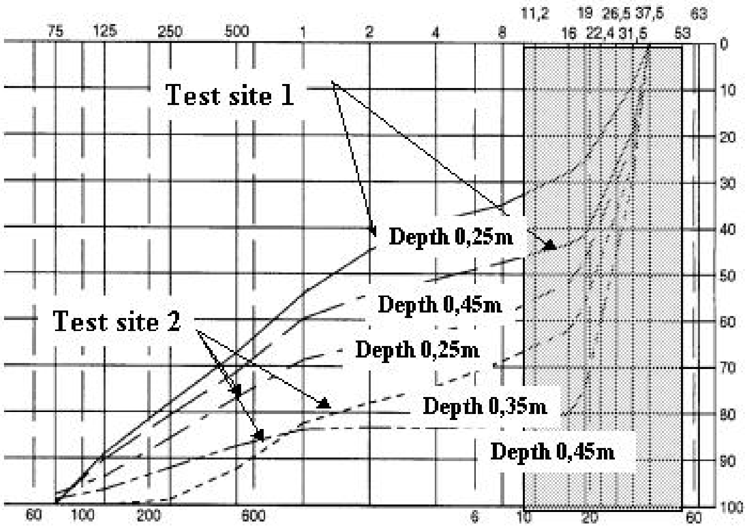

Figure 14: Grains size distribution of excavated material.

4 Conclusions

Foamglass of the type Hasopor is believed to be an interesting lightweight filling material that may be applied in road construction projects if the price is economically favourable compared to competitive products. Consisting of glass the material is believed to be fully resistant to possible chemical degrading agents in a road structure. The mechanical strength of the light quality material may require some special handling in order to prevent excessive crushing. Observations so far indicate that fairly light machinery should be used on site for placement, distribution and compaction of this light quality. In this respect the denser quality may be preferred since the in place unit density of the light quality tend to be at least as high as that to be expected for the denser quality. In this connection both specifications for determining design densities and construction procedures may have to be revised.

With the angular shaped particles the foamglass has a high internal stability facilitating transport with heavy trucks directly on the material. This will, however, cause some breakage and crushing of individual grains resulting in a higher unit density underneath the tracks.

5 References

[1] Aabøe R.: *Lette fyllmasser i vegbygging* (Lightweight filling materials in road construction), Intern rapport 954. Norwegian Road Research Laboratory. Oslo 1980.

[2] Aabøe R. et al.: *Vegbygging på bløt grunn* (Road construction on soft subsoils). Text in Norwegian. Handbook 188 (Code of Practice). Norwegian Public Roads Administration. Oslo 1995.

[3] *Vegbygging* (Road Construction). Text in Norwegian. Handbook 018 (Standard Specifications). Norwegian Public Roads Administration. Oslo 1999.

[4] Frydenlund T. E. and Aabøe R.: *Long term performance and durability of EPS as a lightweight filling material.* In: Proc. EPS Geofoam 2001 (CD-ROM), Salt Lake City, Utah, USA, 2001.

Publications from Road Technology Department (NRRL)

40. R. S. NORDAL. Vormsund Forsøksveg. Del 2: Instrumentering (Vormsund Test Road. Part 2: Instrumentation). 38 p. 1972.
41. K. FLAATE and R. B. PECK. Braced Cuts in Sand and Clay. 29 p. 1972.
42. T. THURMANN-MOE, S. DØRUM. Komprimering av asfaltdekker (Compaction of Asphalt Pavements). Hurtige metoder for komprimeringskontroll av asfaltdekker (Rapid Methods for Compaction Control of Asphalt Pavements). 39 p. 1972.
43. Å. KNUTSON. Dimensjonering av veger med frostakkumulerende underlag (Design of Roads with a Frost accumulating Bark Layer).

    K. SOLBRAA. Barkens bestandighet i vegfundamenter (The Durability of Bark in Road Constructions).

    G. S. KLEM. Bark i Norge (Bark in Norway). 32 p. 1972.
44. J. HODE KEYSER, T. THURMANN-MOE. Slitesterke bituminøse vegdekker (Characteristics of wear resistant bituminious pavement surfaces).

    T. THURMANN-MOE, O. E. RUUD. Rustdannelse på biler (Vehicle corrosion due to the use of chemicals in winter maintenance and the effect of corrosion inhibitors).

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    O. E. RUUD, B-E. SÆTHER, F. ANGERMO. Understellsbehandling av biler (Undersealing of vehicles with various sealants). 38 p. 1973.
45. Proceedings of the International Research Symposium on Pavement Wear, Oslo 6th-9th June 1972. 227 p. 1973.
46. Frost i veg 1972. Nordisk Vegteknisk Forbunds konferanse i Oslo 18-19 sept. 1972 (Frost Action on Roads 1972. NVF Conference in Oslo 1972). 136 p. 1973.
47. Å. KNUTSON. Praktisk bruk av bark i vegbygging (Specifications for Use of Bark in Highway Engineering).

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    K. G. FIXDAL. Snøskredoverbygg (Snowsheds).

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57. R. S. NORDAL, E. HANSEN (red.). Vormsund forsøksveg. Del 3: Observasjoner og resultater (Vormsund Test Road, Part 3: Observations and Results). 168 p. 1984.
58. R. S. NORDAL, E. HANSEN (red). The Vormsund Test Road. Part 4: Summary Report. 82 p. 1987.
59. E. LYGREN, T. JØRGENSEN, J. M. JOHANSEN. Vannforurensing fra veger. I. Sammendragsrapport. II. Veiledning for å

håndtere de problemer som kan oppstå når en veg kommer i nærheten av drikkevannforekomst (Highway pollution). 48 p. 1985.

60. NRRL, ASPHALT SECTION. Surfacings for low volume roads.

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Appendix: Case histories 1-12. 60 p. 1987.

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67. STEINMATERIALKOMITEEN. Steinmaterialer (Aggregates). 20 p. 1993.

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69. J. VASLESTAD. Stål- og betongelementer i løsmassetunneler (Corrugated steel culvert and precast elements used for cut and cover tunnels).

J. VASLESTAD. Støttekonstruksjoner i armert jord (Reinforced soil walls). 56 p. 1993.

70. SINTEF SAMFERDSELSTEKNIKK. Vegbrukers reduserte transportkostnader ved opphevelse av telerestriksjoner (Redused transportation costs for road user when lifting axle load restrictions during spring thaw period). 144 p. 1993.

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75. P. SENSTAD. Sluttrapport for etatsatsingsområdet «Bedre utnyttelse av vegens bæreevne» («Better utilization of the bearing capasity of roads, final report»). 48 p. 1994.

76. F. FREDRIKSEN, G. HASLE, R. AABØE. Miljøtunnel i Borre kommune (Environmental tunnel in Borre Municipality).

F. FREDRIKSEN, F. OSET. GEOPLOT – dak-basert presentasjon av grunnundersøkelser (GEOPLOT – CAD-based presentation of geotechnical data). 48 p. 1994.

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R KOMPEN, G. LIESTØL. Spesifikasjoner for sikring av armeringens overdekning (Specifications for ensuring cover for reinforcement). 40 p. 1995.

79. The 4th international conference on the «Bearing capasity of roads and airfields» – papers from the Norwegian Road Research Laboratory. 96 p. 1995.

80. W. ELKEY, E. J. SELLEVOLD. Electrical resistivity of concrete. 36 p. 1995.

81. Å. KNUTSON. Stability analysis for road construction. 48 p. 1995.

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83. Sluttrapport for etatsatsingsområdet «Teknisk utvikling innen bru- og tunnelbygging» («Technical development – bridge and tunnel construction, final report»). 20 p. 1996.

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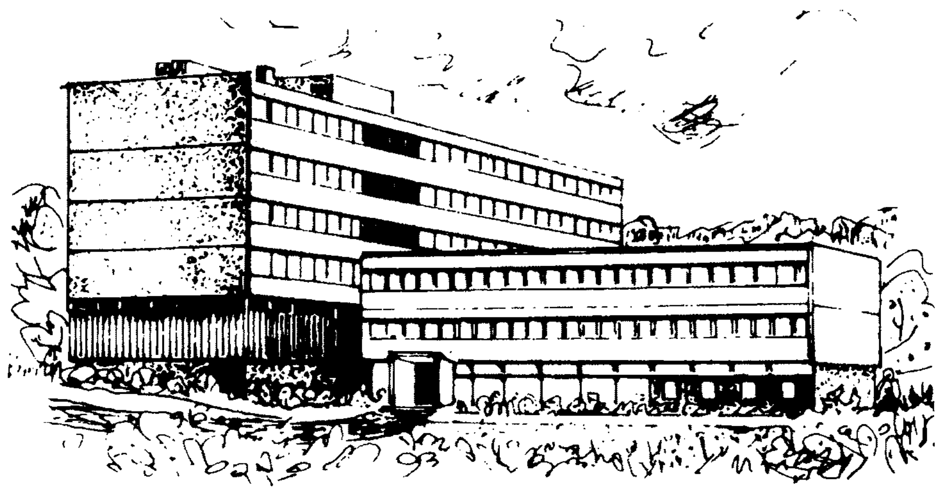

The Norwegian Directorate of Public Roads, Road Technology Department (NRRL)

Organization
The Norwegian Road Research Laboratory (NRRL) was established in 1938. After merging with the Operations Technology Department a new Road Technology Department within the Directorate of Public Roads was created on the 1th of March 1998. The new Department is organized in 6 technical subdivisions:

Pavement Division, Soil Mechanics Division, Concrete Division, Geology and Tunnel Division, Production Technology Division and International Division.

Fields of Operation
Activities are directed towards Research and Development, Information and Tuition, Consulting, Specifications and Guidelines, Testing and Design Approval within the fields of material testing and highway construction and maintenance.

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