DESIGN PARAMETERS FOR EPS GEOFOAM

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ABSTRACT

Over the past 30 years, design with geofoam has been based on either factored strength or limit strain approaches. Geofoam parameters for design have been derived from unconfined compression testing of small laboratory samples. Closer examination of performance observations indicate extrapolation of small sample laboratory results can lead to misleading interpretation of field results. The potential for creep deformations is exaggerated and design modulus values are underestimated when based on small sample laboratory tests. Possible reasons for these shortcomings, in reference to field observations, are examined on the basis of creep tests on small samples, uniaxial loading of large samples, compression tests using tactile pressure sensors and review of enlarged images of geofoam surfaces. Creep deformations in geofoams under uniaxial loading remain mainly in primary stages where strain rates continually decrease. Modulus values for design that are derived from small sample laboratory tests are about half of the values that were estimated from field observations. Accordingly, the suggestion is made to increase small sample based modulus values from laboratory tests for design applications.

Key words: compression, creep, deformation, elastic, geofoam, modulus, polystyrene, strain, stress, tactile sensor (IGC: K14/M9)

INTRODUCTION

The first documented use of EPS (expanded polystyrene) geofoam as lightweight fill occurred in Norway over 30 years ago. The reconstruction of the approach fill to Flom Bridge, near Oslo, is a significant milestone in lightweight embankment construction (Frydenlund, 1987). The motivation for the application was to better respond to the cycle of settlements and grade adjustments of a roadway supported on very compressible soil foundation. The top surface of the geofoam fill was covered with polyurethane spray and the overlying pavement structure was only 0.5 m in thickness. Under estimated daily traffic of some 15,000 vehicles, the road section performed well and settlements were gradually arrested. Geofoam blocks installed in 1972 were recovered and reused when the roadway was reconstructed by 1996. Visual inspection and laboratory testing of samples from the exhumed EPS blocks further confirmed the durability of geofoam in service and under conditions of periodic flooding. The design approach developed in Norway for this project and many more that followed continues to influence practice around the world. Mainly, the allowable surcharge load over geofoam fill is restricted to 30% of the compressive strength at 5% strain as determined by laboratory testing of small size samples at a strain rate of 10% per minute (Frydenlund and Aabøe, 1996). The most commonly used density variety has been 20 kg/m³. The nominal design strength at 5% strain for 20 kg/m³

density geofoam is taken as 100 kPa to provide up to 30 kPa allowable stresses from surcharge loadings. This design method constitutes a factored strength approach based on correspondence between density and strength. To allow for weight increases due to moisture pick up in service, design densities are increased to 50 and 100 kg/m³ for installations expected to remain above and below ground water, respectively. These density adjustments are adequate to compensate for weight increases due to moisture absorption under field conditions (Esch, 1995; Frydenlund and Aabøe, 1996). For buoyancy considerations, the actual 20 kg/m³ is used without adjustment or the geofoam weight is neglected. A friction coefficient of 0.5 has been assumed for geofoam to geofoam and geofoam to soil or concrete interfaces. The practice of providing the equivalent of two double sided timber fasteners per block between geofoam layers was introduced in Norway and has become wide spread. For purposes of pavement design, a modulus of 5 MPa has been assigned for geofoam subgrades. Actual modulus values from project specific tests have not been required for design or product qualification.

In the rapid and innovative development of geofoam applications in Japan since 1985, a design process has evolved wherein stress at a limit 1% strain has become a working load criterion (Miki, 1996). The behavior of geofoam below 1% strain has been considered to be linear elastic. The design standard adapted in Japan provides a 50 kPa allowable compression load for 20 kg/

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m³ density geofoam and thus the same modulus of 5 MPa as was previously assumed in Norway. Allowable stresses to a 1% strain limit remain below half of the respective compression strengths at 5% strain. However, as the stresses under a limit strain criteria account for both surcharge and live loading, the two design approaches developed in Norway and subsequently in Japan are comparable (Negussey and Sun, 1996). In the latter method, the essential correspondence is between density and modulus. Modulus values of the order of 10 MPa and Poisson's ratios generally in the range of 0.1 and lower have been assumed in analyses for geofoam of 20 kg/m³ density (Hotta, 2001). A slightly higher friction coefficient of 0.6 has been in common use in Japan for geofoam to geofoam and geofoam to soil interfaces. Also, larger (150 by 150 mm) double-sided fasteners are used instead of the smaller (100 by 100 mm) size types earlier introduced in Norway. The design procedure introduced in Japan enabled consideration of deformation or performance criteria.

Shear demand at geofoam interfaces from live loading due to vehicle stopping and acceleration is low (Sheeley, 2000). Although static geofoam to geofoam interface shear strengths are higher than residual interface strengths, their use in practice is not common. Residual friction coefficient values of 0.5 to 0.7 have so far been adequate. However, why geofoam interfaces develop higher friction coefficients as compared to geotextiles and geomembranes is not well understood. Although reported to be unnecessary (Sheeley and Negussey, 2000), double sided timber fasteners and similar proprietary devices continue to be installed between geofoam layers.

Capping of geofoam fills with concrete slab for load distribution and protection continues to be a common practice. Apart from minor differences in sample sizes and shapes, design practices in other countries generally follow the state of practice developed in Norway and Japan. The aforementioned geofoam properties have been used for different geotechnical applications (BASF Plastics, 1998; Negussey, 1997). Poisson's ratio values attributed to geofoams, the influence of confining pressures and effects of temperature on geofoam performance have not been closely investigated. The latter properties and effects are generally not important for common near surface and below ground installations of geofoam. Geofoam design parameters and behavior regarding strengths and moduli with density as well as creep phenomena are of more practical significance and are considered herein.

UNCONFINED COMPRESSION AND DENSITY

The geofoam base material or resin price closely follows the trend of oil futures and the cost of EPS blocks increase with density. For large volume applications there can be considerable savings from specifying a lower density geofoam. As shown in Fig. 1, it is clear that both the strengths and moduli of geofoam increase with

Fig. 1. Unconfined compression of 50 mm geofoam cubes of 12 and 33 kg/m³ densities

Fig. 2. Strength gain at 1, 5 and 10% strain levels with increasing geofoam density

Fig. 3. Modulus values for different densities of 50 mm cube samples of geofoam

density. Relationships between limit stresses at 1, 5 and 10% strain levels and geofoam density are as shown in Fig. 2. Differences between strengths at 5 and 10% strain criteria have been used in factored strength design procedures. Initial moduli to 1% strain and with increasing geofoam densities are shown in Fig. 3. The data from different sources are for 50 mm cube samples tested at 10% strain per minute. These results indicate modulus values for geofoam, as a function of density, determined by testing specimens of the same nominal dimensions, at the same strain rate are reasonably reproducible within $\pm 10\%$. Provided the manufacturing process and fusion quality are good, density is a consistent index property for strength and modulus as design parameters.

Fig. 4. Creep strain developments in three geofoam samples at different stress levels

CREEP BEHAVIOR OF GEOFOAM

The main rationale for limiting allowable static pressures to 30% of strength at 5% strain or total stresses to 1% limit strain has been to minimize potential creep deformations. The behavior of geofoam is assumed to be elastic and deformations to be tolerable for working stress levels consistent with these criteria. Figure 4 presents results of creep tests from three 50 mm cube samples of nominal 20 kg/m³ density geofoam. Each test sample was subjected to almost 2 years of sustained unconfined compression loading of 29, 49 or 68 kPa. These load levels correspond to about 30, 50 and 70% of the unconfined compression strength at 5% strain for specimens of the same nominal dimensions and density (Sheeley, 2000). Initial strain of 0.5, 0.8 and 1.5% developed on application of loading at the respective stress levels. Additional deformation of about 0.5, 0.6 and 20% developed over the 2 years of creep loading period. The extent of creep deformation was especially acute for the higher loading at 70% of the unconfined compression strength. Both the 30 and 50% loading conditions resulted in creep strains in addition to the initial or immediate deformations. The results suggest geofoams develop both elastic and plastic strains under loading to 1% limit strain. For 70% loading or what would amount to a stress state above the presumed proportional limit, 5% strain developed in 3 days and up to 20% strain over 2 years of constant loading. Such large creep deformation observations encouraged the practice of setting strict limitation on allowable working stress levels in geofoam design.

Figure 5 presents the creep strain rates associated with the test results shown in Fig. 4. Depending on the sustained load level, common materials deform in stages of primary, secondary and tertiary creep to ultimate rupture. In the primary stage, strain rates continue to decrease while in the secondary, the strain rates become constant. With time, secondary creep can transit to a tertiary stage of increasing strain rates and ultimate rupture. Reducing creep strain rates in the primary stage imply continuing stability as in one dimensional compression loading of soils. Secondary creep is a transition stage while tertiary is a precursor to ultimate instability. With

Fig. 5. Creep strain rates for geofoam samples of 20 kg/m³ density and typical creep stages

Fig. 6. Unconfined compression loading and unloading of 30 kg/m³ density geofoam

reduction in loading or stress intensity, secondary and even tertiary stages can reverse back to primary. The creep deformations of geofoam remained in primary or decreasing strain rate mode for all loading levels even as accumulated strains exceeded 10%, as in the case for the 70% loading. Continually decreasing strain rates imply strain increments diminish with time. This is further substantiated by results shown in Fig. 6 where, under unconfined compression, geofoam continues to stiffen and gain strength with plastic strain instead of approaching rupture. On unloading and re-loading, the proportional limit increases with accumulated strain. The results suggest controlled pre-stressing of geofoam fill can be beneficial in reducing initial deformations while improving the allowable working stress range. This observation is for a higher, 30 kg/m³, density geofoam and lower density geofoams tend to develop softer reloading modulus (Elragi, 2000) but continue to strain harden and stiffen. The laboratory test results suggest creep deformations occurred at all load stages but will tend to eventually cease without inducing shear rupture, regardless of the amount of accumulated strain (Negussey and Jahanandish, 1993; Elragi, 2000).

FIELD OBSERVATIONS

Field monitoring of high geofoam fills at the I-15 reconstruction project in Salt Lake City (Negussey et al., 2001; Bartlett et al., 2001), the NRRL (Norwegian Road Research Laboratory) test embankment and accom-

Fig. 7. Loading and deformation of a large geofoam embankment (Negussey et al., 2001)

panying field monitoring (Frydenlund and Aabøe, 2001) indicate development of limited creep settlements. Figure 7 presents data from the long-term monitoring program at the 100 South Street crossing of the I-15 project (Negussey and Stuedlein, 2003). Two duplicate sensor sets of base pressure cell and extensometer column at 14 m separation represent the North and South instrument arrays. The pressure cells installed below the geofoam fill registered the construction loading. Deformations were monitored with magnet extensometer plates located between the natural subgrade and the geofoam fill and also at different elevations between geofoam block layers. The top three curves represent the loading history on the basis of the two pressure cell readings and estimates from the record of surcharge placement over the geofoam fill. The corresponding bottom two curves represent settlements of the geofoam fill at the respective extensometer positions. The settlements are total deformations of 9 and 9.5 layers of geofoam at the South and North array locations. The nominal thickness of each layer was 0.83 m, so that the initial geofoam heights were about 7.5 m and 7.9 m at the South and North arrays, respectively. A large part of the deformations registered by the extensometers during construction were inferred to be mostly a result of gap closure between block layers (Negussey et al., 2001; Stuedlein et al., 2004). The installed geofoam blocks were not trimmed and tight dimensional tolerance criteria was not enforced. Post construction creep settlements, beyond approximately 300 days, are minimal if not evident even though estimated and recorded stress levels exceeded 30% of strength at 5% strain for the nominal 20 kg/m³ density geofoam.

The self weight of the geofoam fill was negligible as compared to the surcharge pressure of the pavement structure. Estimates of applied pressures and pressure cell readings at the base of the geofoam fill were comparable. Figure 8 shows average stress strain relationships based on the field observations presented in Fig. 7. Typical results from unconfined compression testing of a standard 50 mm cube sample of geofoam installed at I-15 are also included in Fig. 8 for comparison. The curves for the field results manifest an initial lag stage and stiffening with further strain like the compression test behavior of the small size test specimen. The base line reading for the settlement monitoring with extensometer arrays was

Fig. 8. Geofoam deformations from lab result and field observation (Negussey et al., 2001)

taken after the 150 mm thickness concrete load distribution slab was poured but prior to placement of the 1.2 m pavement section. As construction of the pavement over the load distribution slab began, settlements due to gap closure between geofoam block layers occurred. The initial lag evident in the stress-strain results derived from field observations are associated with such gap closures. Whereas the typical initial lag of the stress-strain curve for the small size specimen under uniaxial compression between rigid metal platen is commonly attributed to non-uniform contact interfaces or seating error at the early stages of loading. Beyond gap closure and seating error effects and as loadings progressed, the highest estimates of tangent modulus from the field results are 11.6 and 13.1 MPa compared to 4.1 MPa for the small sized laboratory test specimen. The dashed line in Fig. 8 represents the average of the maximum secant modulus for the two field curves of about 4 MPa. This secant modulus value is closer to what normally is considered to be the elastic modulus derived from corrected stress strain data from laboratory tests on small size specimens. The trend of the field observations suggest the tangent moduli, at stages beyond initial gap closure and for assessing incremental deformations and fatigue life; should be at least twice the maximum modulus derived from the laboratory testing of small specimens. The higher modulus values suggested by the field observations would be more appropriate for evaluating responses due to transient loadings in post construction.

SAMPLE SIZE EFFECTS

Previous studies of geofoam engineering behavior by Duškov (1997), Elragi et al. (2000), O'Brien (2001) also suggest a higher modulus and a linear range less than the normally assumed 1% corrected strain limit. Further small strain modulus evaluation on the basis of compression wave velocities with bender element tests (Sivathayalan et al., 2001) suggest upper bound estimates of geofoam elastic moduli of 14 to 22 MPa and higher than the widely accepted value of 5 MPa (as from results shown in Fig. 3) for 20 kg/m³ density geofoam. An alternative approach for evaluating modulus from small beam deflection testing also indicates higher modulus values (Anasthas, 2001).

Fig. 9. Unconfined compression of 50 and 600 mm cube geofoam samples

Fig. 10. Geofoam modulus with density, samples size and estimate from field observation (Elragi et al., 2000)

When compared to typical design values associated with different types of soils, 5 MPa is in a range normally associated with very soft to soft clays (Das, 1998). Whereas the over 10 MPa modulus implied by the field data indicates 20 kg/m³ density geofoam compares better with stiffer clays. Even when the higher unit cost can be justified, the apparent equivalence of geofoam modulus to very soft to soft clay soils can be ambiguous for practitioners less familiar with prior geofoam applications. Figure 9 presents results and comparison of unconfined compression tests on 50 and 600 mm cube samples. Both the small and large size cube samples are of nominal 15 kg/m³ density geofoam and were tested at 10% per minute strain rate. However, the stress strain curve for the large sample is based on deformations measured over the middle third of the height and away from possible edge effects at the loading platen boundaries. The results show the large sample based modulus can be double that for the small sample of the same density. However, the strengths at either 5 or 10% strain criteria are about the same for both sample sizes. A similar difference in modulus values over a range of densities was reported by Elragi et al. (2000) and is shown in Fig. 10. Also included in Fig. 10 are the modulus estimates from field observations and laboratory testing shown in Figs. 3 and 8. The modulus of about 10 MPa for 600 mm cube samples of 20 kg/m³ density geofoam is in better agreement with modulus estimates from field observations, provided gap closure effects are viewed separately. The larger height of 600 mm cube samples is closer to the thickness of common full size EPS blocks. Modulus values derived

Fig. 11. Details of a tactile pressure sensor (adapted from Tekscan, 2002)

from laboratory tests on small size samples are too small and rather unrealistic to be used in design. Beyond adjustments for seating error, the reason for the noted significant difference in modulus obtained from small and large sized samples was assumed to be due to end effects.

Conditions of uniformity in both stress and deformation cannot be imposed simultaneously at the interface of two materials that have contrasting stiffness. In a laboratory test, the loading platens impose uniform deformation across the section area of test samples. Therefore, the stress non-uniformity at the relatively rigid loading platen and flexible but cohesive geofoam sample was inferred to produce higher stresses toward the edges in the manner discussed by Taylor (1948). This reasoning was assumed for interpreting results of stacked samples (Elragi et al., 2000) where the average deformation near the rigid loading platen was shown to be higher than deformations across geofoam to geofoam interfaces. As a result of higher edge stresses over the top and bottom end faces of geofoam test samples, crushing and local yielding were envisioned as likely mechanisms for the low modulus and exaggerated creep deformation of small size samples. With development of tactile pressure sensors, an alternative means for sensing and investigation of interface pressure distributions became possible.

INTERFACE PRESSURE DISTRIBUTION

Tactile pressure sensors consist of two flexible sheets each containing thin conducting strips over semi-conducting coated surface. Two sheets are mated to form a very thin sandwich of semi-conducting surface between conducting strips. In forming the sandwich, the upper and lower conducting strips are oriented orthogonally to form a grid pattern or matrix of pressure sensing elements called sensels, Fig. 11. A row of 44 strips is mated with a column of 44 strips to form a matrix of 1936 sensels. Standard sensing pad sizes are available and specific sizes and shapes can be custom made. The spacing of the conducting strips is varied to match the desired size of the sensing area. The semi-conductor layer between the conducting strips changes in resistance in response to applied pressures. Resistances measured at each grid point are highest when no external pressure exists. The

Fig. 12. Comparison of average pressures from a load cell and tactile pressure sensors

conducting strips are protected by flexible polyester sheets and connect to a handle. The handle connects to a data acquisition board in a computer via a cable. The overall sensor thickness can be less than a millimeter. The thickness of the semi-conducting layer controls the sensor operating capacity, which can vary from a low of 0 to 14 kPa to a high of 0 to 175 MPa. Pressure sensor calibration studies indicate errors in accuracy to be less than 10 to 20% depending on the pressure range. Sequential scanning of the pressure field is displayed and stored in real time. The average sensor output is calibrated against a known force or pressure to establish an adjustment factor. Force and pressure time histories of each sensing point can be further analyzed by exporting the data to other software. The sensor manufacturer provides both the hardware and software for the system. Tactile sensor technology is about 25 years old and is now used in other fields and industries (Paikowsky and Hajduk, 1997; Sodhi, 2001).

A 56 by 56 mm pressure sensing pad, like the one shown in Fig. 11, was placed at the top and bottom of a 50 mm cube geofoam test specimen. There were 1936 sensels over the full sensor pad. Pressure states at about 1600 locations over each sample end surface contact area were detected. Thus the area coverage per sensel was approximately 1.6 mm². The tactile pressure pads had a range of 0 to 250 kPa. Standard unconfined compression tests at a strain rate of 10% per minute were performed on geofoam samples of 15 kg/m³ nominal density. Pressure distributions between geofoam to geofoam interfaces were also examined by testing two stacked 50 mm cube samples with tactile sensors placed in between the samples and also at the interface of the lower block and base pedestal. A load cell and a displacement transducer recorded the applied vertical force and resulting deformation synchronously.

Figure 12 shows a plot of vertical stress against strain for a geofoam sample based on data from the tactile sensors and the load cell output. The lower curve represents the unadjusted average output from the top and base tactile sensor pads. With adjustment or calibration, the average stress strain curve for the tactile sensors matched with the load cell based stress strain results reasonably well. All three curves show the characteristic bedding error stage at initial loading, presumed elastic

Table 1. Maximum, average and minimum pressure states at 1% corrected strain in 25 clusters over the base platen and geofoam interface: Each cluster contains 64 sensels

		8 sensels in each of columns A to E				
		A	B	C	D	E
8 sensels in each of rows 1 to 5	1	153	86	158	72	108
		29	11	17	16	17
		0	0	0	0	0
	2	230	89	81	132	114
		39	24	23	37	33
		0	0	0	0	0
	3	Max	57	101	114	117
		Ave	17	21	24	32
		Min	0	0	0	0
	4	170	120	98	81	72
		37	28	21	22	28
		0	0	0	0	0
	5	120	68	89	89	77
		34	20	19	20	20
		0	0	0	0	0

region followed by apparent yield and plastic deformation. The corrected modulus of about 2.5 MPa, strength at 5% strain of about 60 kPa indicated by results from the tactile sensors and also the load cell are in good agreement with corresponding values for 15 kg/m³ nominal density geofoam, shown in Figs. 2 and 3. There is good correspondence and consistency between the stress strain curves and moduli derived from the record of tactile sensors and load cell observations, as shown in Fig. 12.

Individual sensel pressure states of the tactile sensor pads were captured and stored for further analyses and graphic output. In play back mode, different stages of the test were viewed separately, frame by frame, to observe pressure distribution patterns over the geofoam test sample end areas. At the beginning of loading and in the seating error stage, the applied load induced contact pressure on only about half of the sample end areas. With continued loading, the contact areas enlarged to the size of the sample end surface. A 3D representation of pressure patterns over the geofoam test specimen and base platen interface showed scattered spikes and load free sensels. A summary of pressure states for all sensel positions, as in Fig. 11, at a corrected strain of 1% and average adjusted pressure of 25 kPa (at approximately 2% vertical strain in Fig. 12) is presented in Table 1. Each of columns A to E contains 8 sensel columns and each of rows 1 to 5 contains 8 sensel rows. Thus clusters 1A through 5E each represent 64 sensels for a total of 1600 potential load bearing sensels at the base platen and geofoam interface. Maximum, average and minimum of 64 sensel pressure states within each of the 25 clusters are presented in Table 1. Maximum pressures vary from 230 to 57 kPa with an average of 110 kPa. Maximum pressure states in all clusters, except 3B, exceed 60 kPa or the strength of the geofoam at 5% strain. In several clusters, maximum pressure states exceed twice the 60 kPa refer-

Table 2. Contact pressure states at 1% corrected strain for 64 sensels of cluster 3C, the middle cluster in Table 1

Row	Column							
	C.1	C.2	C.3	C.4	C.5	C.6	C.7	C.8
3.1	29	24	0	17	9	12	27	9
3.2	8	36	0	9	48	36	20	0
3.3	0	8	0	0	60	29	0	0
3.4	0	33	45	0	0	0	15	0
3.5	12	20	0	0	0	0	50	29
3.6	0	29	101	0	0	50	56	0
3.7	41	21	32	33	32	89	0	0
3.8	41	32	33	38	27	0	80	36

ence strength for setting an allowable working stress. Average pressure states in the same 25 clusters range from 39 to 11 kPa with a mean of 25 kPa. The mean of the average pressures is the global average of all 1600 sensels and is equal to the average pressure derived from the load cell record at 1% corrected strain. The 25 kPa global average pressure at 1% corrected strain constitutes the allowable working stress for the test sample geofoam density. However, maximum pressures of up to one order of magnitude higher than the allowable working stress developed at discrete positions over the end contact area. Initial contact and loading started along the left edge of the sample. The highest contact pressures continued to develop on the left side as loading progressed but zero pressure states existed in all 25 clusters and at all stages of loading and strain levels. Pressure states for all 64 sensels of the center cluster 3C at 1% corrected strain are shown in Table 2. Maximum, average and minimum pressures for cluster 3C were 101, 21 and 0 kPa, respectively. Both the maximum and average pressures of 101 and 21 kPa were below the mean of maximum and average pressures of 110 and 25 kPa for the 25 clusters. Over 1/3 of the sensels in cluster 3C were under zero pressure while about 42% of the sensels were at pressure states greater than the allowable working stress of 25 kPa. Approximately 6% of the sensels in cluster 3C were at pressure states equal or greater than the geofoam strength of 60 kPa at 5% corrected strain.

Frames containing pressure states for all sensels at corrected strain levels of 1 and 5% were extracted to further review the pattern of stress distributions. Figure 13 shows the percentile of the contact areas that were subjected to different tactile pressures at the top and bottom of geofoam to rigid plate interfaces. At the 1% strain level, about 10% of the end areas were subjected to pressures at or above the average pressure associated with 5% global strain. On the low side, up to 30% of the end areas were under zero pressure at 1% strain. About 10% of the end areas were at contact pressure states below yield when the global strain reached 5%. With yielding and pressure redistribution in the vicinity of local maximum pressure areas, creep rates would have

Fig. 13. Sensel pressure states at 1 and 5% corrected vertical strain for tactile sensors at top and bottom geofoam and loading platen interfaces **Fig. 14. Contact pressures between geofoam/geofoam and geofoam/end platen surfaces**

decreased in the manner evident in the long duration constant loading results shown in Fig. 5. As local stress states greater than yield began to develop at the earliest stages of loading and co-existed with states of zero pressure throughout the test period, elastic and plastic conditions remained current at a loading interface at all times.

Figure 14 shows comparisons of pressure distributions between a geofoam/sensor/geofoam interface and at a geofoam/sensor/base loading platen for a test on two stacked 50 mm cube geofoam samples. The interface at the bottom tactile sensor pad was between the metal base plate and the lower geofoam block end area. The top tactile sensor pad was located at the geofoam to geofoam interface between the two samples. While the trend of contact pressures for the upper and lower sensor pads was similar, the upper geofoam to geofoam interface experienced a lower maximum pressure profile. Maximum contact pressures between geofoam surfaces should even be lower, if detected with a more flexible tactile sensor pad to closely approximate a geofoam to geofoam interface. This is because the sensor acts to bridge voids to inhibit cushioning of small area stress concentrations. The interlocking of intact cells and void spaces is a likely cause for observed good interface strengths at geofoam to geofoam interfaces. Figure 14 also shows that peak pressure states exceeding levels corresponding to the strength at 5% corrected strain began to develop near zero corrected strain and at initiation of loading. Such high stresses tend to influence laboratory test results but would not develop to the same extent to significantly

Fig. 15. Resin beads, pre-puffs and a cut surface exposure of a geofoam block

Fig. 16. Magnified (X600) image of pre-stressed cells in a pre-puff

affect field performance.

CONTACT SURFACE DETAILS

A photograph of resin beads (the feed stock for geofoam production), pre-expanded beads or pre-puff together with a small cut sample from a geofoam block is shown in Fig. 15. The spherical resin beads, on the lower left, are generally 0.5 to 1 mm in diameter and contain tiny impregnated gas bubbles, pentane, as a blowing agent. The gas bubbles are contained in a polystyrene matrix. In the pre-expansion stage, the resin beads are subjected to steam and pressure. The polystyrene matrix softens to allow the gas bubbles to expand in forming the pre-expanded beads or pre-puffs, on lower right, that acquire diameters of 3 to 5 mm. The resin type and extent of pre-expansion generally determine the density grade of the geofoam block. As neighboring gas cells act to contain the expansion of adjacent cells, the spherical bubbles within a resin bead acquire polyhedral shapes within the pre-puff, Fig. 16. The field of view in this scanning electron microscope image (about X600 magnification) covers an area of about 0.05 mm². The individual cell units within a pre-puff are too small to be detected in isolation by a sensel of about 1.6 mm² area.

The pre-expanded beads are poured in a mold and are subjected to steam and pressure under all around confinement to form a geofoam block. On further expansion and fusion while in the mold, the pre-expanded beads or pre-puffs become immobilized in a matrix of discontinuous air voids. This is shown in Fig. 17 where individual pre-expanded beads have fused and inter-particle (inter pre-

Fig. 17. Magnified (X30) image of fused pre-puffs and void labyrinths at a geofoam surface

puff) void spaces are visible. For the sensel dimensions of the tactile sensors that were used, about 8 sensels cover the field of view (about X30 magnification) in Fig. 17. Some sensels would therefore cover fully or partially void space while others would come in contact with pre-puff elements. However, at a cut surface, as in the background of Fig. 15, most pre-puff units become cut. The void spaces become exposed to atmospheric pressure and on occasion intact pre-puffs remain undamaged at or very near to the cut surface. The few sensels that came in full contact with undamaged pre-puffs were likely positions at which maximum pressures developed. However, even at a cut pre-puff, some gas cells were open while others remained intact. The image in Fig. 16 covers a portion of a cut pre-puff from a sample that was previously loaded to 10% strain. Most of the cells were open and the collapsed walls formed hexagonal configurations. Closed cells are evident at the south east corner of Fig. 16. In mating cut geofoam to geofoam as opposed to geofoam to metal surfaces, the maximum tactile pressures at geofoam interfaces would be lower and less susceptible to creep. Within the interior of the geofoam mass, pressure in the discontinuous voids can increase in response to loading and deformation of the fused pre-puff particles. The initially spherical pre-puff particles tend to acquire polyhedral shapes with loading and deformation. As the network of fused pre-puffs contains the trapped air in the labyrinths, pressure increases within the voids contribute to stiffening and more uniform internal pressure distribution. Along the outer perimeter surface, voids between pre-puffs remain exposed to atmospheric pressure. As a result, the portions of geofoam in the proximity of the outer boundary surface would tend to manifest more compressible behavior. When spherical pre-puffs and discontinuous void spaces come in contact with loading platen, higher pressures begin to develop at the sphere contacts like for individual grains in a granular matrix. However, whereas granular particles can rearrange, the fused pre-puffs remain relatively immobile. Load shedding would tend to be restricted and the intact pre-puffs and cells within sustain pressure. The maximum stresses registered by the sensels reflect the combinations of relaxation, as the wall of closed cells expand, changes in gas volume occur and interaction with adjoining cells continue. Void spaces and open cells occur across a cut face and their influences on creep deformation behavior

become exaggerated with decreasing sample height. Modulus values determined by testing small samples represent significant underestimates of the behavior of full sized blocks.

CONCLUDING REMARKS

With 30 years of field experience and research, some changes in determination and application of geofoam design parameters appear to be in order. Density is a good index property for design parameters and classification of geofoam. Both strength and modulus of geofoam increase with density. Field and laboratory results indicate modulus values determined by testing small size laboratory samples can be increased. Thus for the common 20 kg/m³ density geofoam and the generic design value most often used, a modulus of up to 10 rather 5 MPa is more consistent with actual behavior of large blocks. The mix of closed cells and voids over a geofoam surface and associated variability of contact pressure intensities evident from tactile sensor observations also provide insight into friction coefficients for geofoam to geofoam sliding. The interface friction between geofoam surfaces is higher than values for geomembranes because of the surface conditions. Creep concerns previously identified on the basis of laboratory tests on small samples have not been evident to the same extent in field observations. Even where excessive deformations under sustained pressures occurred in laboratory tests, the creep states remained in primary stage and did not lead to rupture or failure. Field observations indicate seating and gap closure movements mostly occur during construction. Post construction total and differential settlements of geofoam fills have generally remained tolerable. By increasing the design modulus and with better understanding of creep behavior in geofoams, lower density and less expensive grades of geofoam can be justified for some applications. Alternatively, more intense static and transient design loads can be supported with higher density geofoams. The increase in modulus improves the correspondence of design estimates and performance observations for dead load or surcharge loading. For transient response, such as from traffic loading on rigid or flexible pavement structures, the combined action of the load distribution slab and the geofoam fill should be considered. Additional effort has been directed to examine alternative approaches of representing concrete slab and geofoam composite subgrades for pavement design.

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