Geosynthetic Cellular Systems (GCS) for Coastal Protection

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ABSTRACT
A new method for the design and construction of structures for coastal erosion protection applications has been developed. The method, called Geosynthetic Cellular Systems (GCS), appears to have significant advantages compared to conventional riprap systems from the standpoint of constructability, cost effectiveness, and environmental considerations. A simple method is presented for analyzing the geosynthetic in a GCS structure that behaves as a membrane. Internal stability considerations under vertical loads are considered and discussed. Some limited laboratory studies were also carried out and the results agree reasonably well with the analytical predictions.

RÉSUMÉ
Une nouvelle méthode pour le design et la construction d'ouvrages de protection contre l'érosion côtière a été développée. La méthode, nommée Systèmes Cellulaires Géosynthétiques (SCG), semble avoir de nombreux avantages par rapport aux ouvrages conventionnels d'enrochement du point de vue de la constructibilité, des coûts et des considérations environnementales. Une méthode simplifiée est présentée pour l'analyse du géosynthétique d'une structure SCG, géosynthétique qui se comporte comme une membrane. Les considérations de stabilité interne sous pression verticale sont examinées et discutées. Des études limitées en laboratoire sont aussi présentées, et leurs résultats confirment raisonnablement les prévisions analytiques.

1 INTRODUCTION

Sea level appears to be rising globally and this rise is expected to accelerate over the next 100 years (Douglas et al. 2001, USGCRP 2001). Current estimates for sea-level rise based on the last 150-years are in the range of 1 to 2.5 mm/yr with 1.8 to 2 mm/yr considered the "best estimate" (Gornitz 2000). An increase in sea level tends to increase coastal erosion by depleting the area of coastal land. For example, applications of Bruun's rule and other simplified prediction methods suggest that a 300 mm rise in sea level would erode the shorelines of New Jersey and Maryland 15-30 m, 30 to 60 m in South Carolina, twice that amount in California, and between 30 and 300 m in Florida (Bruun 1962, Kana et al. 1984, Kyper and Sorensen 1985, Wilcoxen 1986). Strategies for coping with coastal erosion and flood damage associated with a rising sea level include defending the shoreline by means of protective structures, beach restoration, and ultimately, retreat (NRC 1987, 1990, 1995).

Shoreline armouring is typically applied where substantial infrastructure and other assets are at risk. Hard structures include seawalls, groins, jetties, and breakwaters. Seawalls and bulkheads, a common form of shore protection in urban areas, intercept wave energy, and often increase erosion at their bases, which may eventually undermine them. Erosion can be reduced by placing rubble or rip-rap at the toe of the structure. Groins, often built in series, intercept littoral sand moved by longshore currents, but if improperly placed, they may increase beach erosion further downdrift. Similarly, jetties designed to stabilize inlets or to protect harbours, may lead to erosion. Offshore breakwaters (long rows of rip-rap dumped parallel to the shore to intercept waves) reduce wave energy before it reaches the beach. If they are submerged, they act as a coral reef and cause the waves to break before they reach the shore.

Because hard structures are expensive and may only increase erosion in other areas, a soft approach involving dune restoration and beach nourishment has emerged as the preferred means of shoreline protection (NRC 1995). Sand that has been dredged from offshore or other locations can be placed onto the upper part of the beach. Since erosional processes are continual, beach replenishment must be repeated frequently, often an average of every two to four years, and this is very costly. For example, the U.S. Army Corps of Engineers has spent a cumulative total of $2.4 billion in the U. S., including $884 million just within New Jersey, New York, and Pennsylvania on beach nourishment projects since the 1920s (Gornitz 2000). No single protective measure appears to work best in all situations. Therefore, both hard structures (sea-walls, dykes, groins and breakwaters) and soft approaches involving dune restoration and beach nourishment should be considered in an integrated coastal zone management approach.

This paper reports on a new method called Geosynthetic Cellular Systems (GCS) that may be considered as an alternative to hard coastal protective structures. GCS utilizes geosynthetics and granular soils (dredged materials) to create massive gravity structures that have the same functionality as conventional hard systems, but with some exceptional advantages. The proposed system uses dredged materials much more efficiently because the geosynthetic containment system retains them as long as the geosynthetic lasts. It should however be noted that several soft armor methods utilizing geosynthetics have already been proposed and used in practice. Geotextile wrap-around revetments are flexible structures that have been used since the 1980s as an economical solution for coastal erosion problems (Saathoff. and Kohlhase 1986, Recio-Molina and Yasuhara 2005). These are sand slopes where the sand is wrapped and encapsulated with geotextiles in layers in order to create a reinforced soil mass to act as a flexible revetment. An alternative method uses geotextiles as containment units in different forms like tubes, containers and bags in marine applications to prevent the erosion. Lawson (2008) has presented an extensive review of these systems and their use in a wide range of hydraulic, marine, and environmental applications.

2 BACKGROUND AND DESCRIPTION OF GCS

The background of CGS is the anchored geosynthetic system (AGS) stabilization method (Ghiassian et al. 1997), but with extensive modifications. In the AGS method, a geosynthetic is draped over the face of a slope and tensioned through steel rods or nails that are driven into the underlying soil mass. The developed tension and curvature of the geosynthetic combine to compress the soil and increase the confining or normal stresses on potential failure surfaces. AGS can provide a non-intrusive, environmentally compatible alternative to hard armouring, which in many instances is prohibited in environmentally sensitive areas such as coastal sand dunes and beaches. AGS can also protect slopes against both internal seepage that promotes piping and mass instability, as well as external wave forces that may cause serious scour and erosion. Performance of AGS depends on the developed tension in the geosynthetic and on the pullout resistance of the anchors, especially over the lifetime of the structure. Experience has shown that tensile forces in the anchors decrease due to creep and stress relaxation in the soil as well as in the geotextile; therefore, they have to be re-stretched after some time (Vitton 1991). In applications such as levees where the AGS method is applied to both sides of the levee, the required tensile force in the anchors is achieved by using one set of anchors connecting the two sides. The pullout resistance of the anchors is not a factor in this case because the two sides interact through anchors that span across the slope, as shown conceptually in Fig. 1.

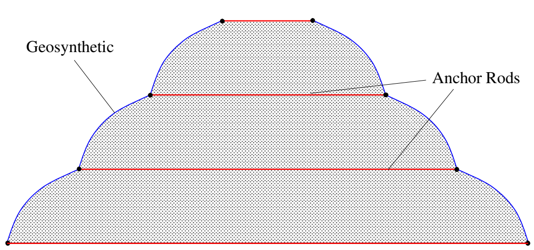


Figure 1. Conceptual application of AGS to a levee or embankment with horizontal anchors

An interesting variation of this idea would be to construct AGS-type breakwaters by replacing the rockfill with dredged granular soil. Such a system could be economical in situations where the required hard armor materials are either unavailable or very costly and dredged materials are readily available. Also by using an appropriate geosynthetic, vegetation can be established on the surface so that the structure becomes even more attractive.

GCS systems are somewhat similar to traditional cellular cofferdams, although essentially any geometric configuration can be designed and constructed. GCS is composed of three main components: soil, geosynthetic, and a frame. The geosynthetic acts as a shell to transfer lateral soil loads to the frame elements, as well as a filter to keep granular particles inside the GCS while allowing water to drain.

The frame is built similarly to other structural frames but its members (beams and columns) must be made of corrosion resistant materials. For example, PVC pipes with light weight and versatile connections make transportation to the site and construction of the frame easy and feasible. This is especially advantageous and economical for temporary applications, in which the GCS structure could easily be unzipped, repositioned, and refilled. The vertical spacing of elements can be small at the bottom and increase with height (similar to the variable spacing of reinforcing sheets sometimes used in a reinforced soil slope or wall). After the frame is built, geosynthetics are placed around the exterior of the frame and connected to the frame using, e.g., plastic ties. The frame is then transferred to the desired off-shore location and allowed to sink. After the frame is positioned on the sea bottom, it is backfilled with sand and gravel. Depending on local conditions, either on shore materials or dredged soils can be used for backfill. If compaction is necessary, heavy duty concrete “stinger” vibrators may be used to compact the soil during filling. Because of lateral earth pressure, the geosynthetic sheets undergo tension and transfer loads to the frame elements, which act as anchors. Figure 2 shows a schematic of GCS with vertical and horizontal frame elements.

In the following sections, the analyses of CGS structures including internal stability considerations are presented. Then the results of some laboratory experiments on small cylindrical models filled with water and sand are presented and their behavior is compared with theoretical predictions. Details of the analytical derivations and experimental work are presented elsewhere (Ghiassian and Holtz 2005).

3 THEORETICAL CONSIDERATIONS

Analysis of GCS requires the determination of forces (stresses) in the three components of the system, i.e., geosynthetic, soil, and frame elements. The most important part however is the analysis of the geosynthetic, because it behaves as a membrane. Membranes can only carry internal tensile forces tangent to their deformed shapes. Examples include certain biological tissues, inflatable systems, soap bubbles, and shells subjected to high pressures that result in very large strains. Membranes have negligible bending stiffness and cannot support compressive stresses without wrinkling. The basic assumption implied in the analysis of membranes is that the deformed shape of the membrane adapts itself to the external loads, a feature that demands a non-linear theory. In this theory, the deflection of any point in the body is not necessarily proportional to the magnitude of applied load, and the state of stress depends markedly on the final shape of the membrane. This behavior is more pronounced for flat membranes that are loaded out of their planes. In these conditions, the membrane undergoes finite curved deformations until an equilibrium shape is reached in which the sectional (tensile) loads have components normal to the plane of the undeformed membrane. Theoretical solutions lead to nonlinear displacement equations that can be evaluated using approximate methods (Otto 1979).

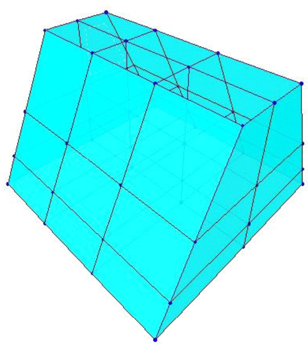


Figure 2. Schematic representation of a GCS structure

Two sources of nonlinearity are often considered: geometric and material. In the former, the strain-displacement is nonlinear, while and in the latter the stress-strain relations are nonlinear. Geosynthetics are generally expected to exhibit both geometric and material nonlinearities in GCS applications.

The computational effort in using the above theory, however, appears to be laborious especially for cases of both geometric and material nonlinearities. Because the actual displacement-deformation relationships are nonlinear, the displacement equations are also nonlinear. Therefore, no exact solution of the differential equations for the displacement can generally be obtained. Equivalent integral equations (a variational problem) could be evaluated according to some approximate methods like Ritz' method (Otto 1979). Computer codes that can analyze nonlinearities are obviously more viable and appropriate for analyzing these systems.

In this study, the three GCS components have been analyzed as in the following sections.

3.1 Soil

Since the geosynthetic behaves as a membrane, it can reasonably be assumed that sufficient lateral deformation occurs in the geosynthetic so that active earth pressure conditions prevail. Using the Coulomb's active earth pressure equation for horizontal surface and zero interface friction (the interface friction angle in GCS structures is expected to be small and can conservatively be neglected, or μ=0), the following expression for the active coefficient can be obtained in terms of soil friction angle and wall (geosynthetic) inclination.

$$K_a = \frac{\sin^2(\beta-\phi)}{\sin\beta(\sin\beta+\sin\phi)^2} \qquad [1]$$

where ϕ is soil friction angle and β is the inclination of the soil "wall" with the horizontal surface.

It should be noted that the amount of lateral soil pressure in a GCS structure with inclined sides is smaller than that for vertical sides. Besides, the external stability of the structure increases as the sides become more inclined from the vertical. Pertaining to the wave action, the inclined sides are also preferred to the vertical sides because any wave run-up becomes smaller. Thus, if the water depth around the structure is not an important design consideration, it might be advisable to make the system with inclined sides.

As a conservative approach, a plane strain condition is assumed in the analysis. The active soil pressure is applied normal to the sides at any depth based on Eq. 1. If hydrostatic pressures are present inside and outside, only lateral active pressures based on effective overburden pressures are considered. Otherwise, if the soil inside GCS and above the water level is expected to become saturated due to wave action or other causes, the water pressure above the water level also should be added to the active soil pressures.

3.2 Geosynthetic

The theoretical solutions based on large deformation membrane theory for geosynthetics, subjected to non-uniform loading conditions are not feasible without a computer code developed for this purpose. The code may analyze the whole system, i.e., three components of geosynthetic, soil, and elements, or it is only used for analyzing the geosynthetic. For the former, an appropriate constitutive model should be selected for the soil, and for the latter, the soil pressure can be calculated based on the active condition (Eq. 1) and be applied directly on the geosynthetic. A simple approximate approach called "Simple Method" is also proposed which provides good insight into the deformational behavior of the geosynthetic. It can also predict conservative values for the tensile force and deflection of the geosynthetic to be used for preliminary design purposes.

3.2.1 Flat Geosynthetic

As a flat geosynthetic bends to a curvilinear shape under normal stress, the induced hoop force for an infinitesimal element of the geosynthetic can be estimated from the following expression.

$$T = \sigma R \qquad [2]$$

where T is the tensile force in the geosynthetic per unit width, σ is the normal pressure on the element, and R is the radius of curvature of the deformed element. It can be seen that the tensile force is influenced by the deformation of the geosynthetic i.e., the radius of curvature R.

Consider an initially flat geosynthetic at the end of loading after it has deformed to a curvilinear shape. A unit width strip of the geosynthetic with a span length of S is assumed to be fixed at both ends and deforms to a cylindrical shape under the applied uniform normal stress of σ with a maximum deflection of δ and in a plane strain condition. The assumption of cylindrical deformation was made by Timoshenko and Woinowsky-Krieger (1987) for an approximate solution for the bending of long rectangular plates. Moreover, experimental observations of deformed membranes confirm that this assumption is realistic and rational (Otto 1979).

Equilibrium and deformation equations were written based on the following assumptions (for details, see Ghiassian and Holtz 2005).

- Negligible influence of boundary conditions at the supports on the state of tensile stresses in the geosynthetic,
- Frictionless interface between soil and geosynthetic,
- Normality of the applied pressure on the deformed shape of geosynthetic.

Figure 3 shows the relationship between applied normal pressure (σ) and deformation ratio (δ/S) for different values of S for a specific example geotextile (Mirafi Geolon-HP570). To illustrate this simple method, an example of the above geosynthetic with dimension of 2m by 2m is solved and the results are compared to those obtained from the large membrane deformation theory as presented in Table 1.

It is seen that both predicted values of maximum deformation and tensile force from the simple method are higher than the theoretical predictions. The differences however are about 25%, so that the method can conservatively be used in practice. One advantage of the simple method is that the geosynthetic material nonlinearity can easily be incorporated in the analysis by incremental loading, whereas in the theoretical solution, this condition will only add more complexity to the solution. Considering large safety factors that are generally used in the design of geosynthetics (Koerner 2005), the amount of conservatism in the simple method seems insignificant.

It can further be shown that the deformation of the flat geosynthetic can actually be presented as a function of D=Et/S, called the total stiffness of the membrane if 1/S is considered as the geometric stiffness of the geosynthetic. Smaller values for S will obviously increase the total stiffness of the geosynthetic resulting in smaller values for δ as can be seen from plots in Fig. 4. These plots can more be generalized in terms of a dimensionless coefficient N called a Stability Number and defined as N=D/σ. It is seen in Fig. 4 that only one plot represents the relationship between parameters N and δ/S for all types of geosynthetics and applied normal stresses.

From Fig. 4, it appears that some bending deformation of the flat geosynthetic (e.g., δ/S≈0.1) is inevitable even at very large stability numbers. On the other hand, at small values of N (say less than 10), the change in stability number has much more influence on deformation. This means that the deformation of the geosynthetic is more responsive to the variation of the stability number at lower values of N. Thus, it can be concluded that neither very stiff nor very soft geosynthetics (with corresponding large and small stability numbers respectively) would be appropriate for GCS applications. With appropriate type of geosynthetic, the bending ratio can be limited in a reasonable range (say 0.1<δ/S<0.2).

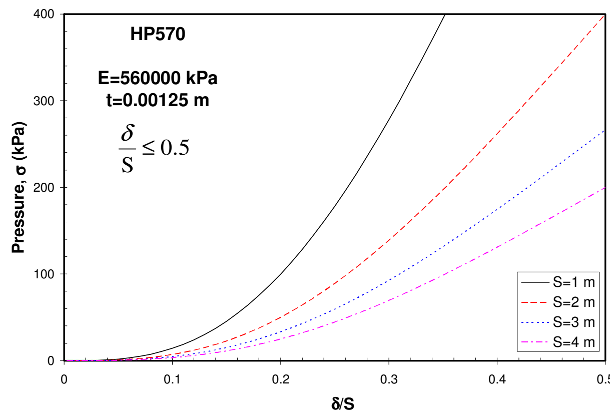


Figure 3. Deformation ratio versus normal stress in the Simple Method for Mirafi Geolon-HP570 geosynthetic

Table 1: Comparison between the results of the simple solution and the large membrane deformation theory for an example geotextile of 2mx2m dimension

Geotextile: Geolon HP570 (Mirafi) E (Young's modulus) = 560000 kPa t (thickness) = 0.00125 m S = 2 m ν (Poisson's ratio) = 0.3 and σ = 115 kPa (values selected such to use the plots in Otto, 1979)
Deformation Theoretical Solutions (Otto, 1979) $\chi = \frac{\sigma S(1-\nu^2)}{Et} = \text{Stability Coefficient} \propto \frac{1}{\text{Stability Number in Simple Solution}}$ $= \frac{115\times 2\times(1-0.3^2)}{560000\times 0.00125} = 0.3 \Rightarrow \frac{\delta_{max}}{S} = 0.225 \Rightarrow \delta_{max} = 0.225\times 2 = 0.45\ \text{m}$ Simple Solution $\sigma = 115\ \text{kPa} \Rightarrow \text{Figure 3} \Rightarrow \frac{\delta}{S} = 0.278 \Rightarrow \delta_{max} = 2\times 0.278 = 0.56\ \text{m}$
Tensile Force $\text{Theoretical Solution (Otto, 1979)} \Rightarrow \frac{T_{max}(1-\nu^2)}{Et} = 0.145 \Rightarrow T_{max} = 111.5\ \text{kN/m}$ $\text{Simple Solution} \Rightarrow R = \frac{\delta^2 + \frac{S^2}{4}}{2\delta}$ (see Ghiassian and Holtz, 2005)

$$\Rightarrow R = \frac{0.56^2 + \frac{2^2}{4}}{2 \times 0.56} = 1.17 \Rightarrow T_{max} = \sigma \times R = 115 \times 1.17 = 134.6 \text{ kN/m}$$

3.2.2 Curvilinear Containers

Due to their very small bending stiffness, geosynthetics are expected to carry surface pressures through in-plane tensile deformation. In this regard, according to Eq. 2, the curvature of the geosynthetic has an important effect on the magnitude of the induced tensile force. Therefore, it might seem beneficial to make containers with initial curvilinear rather than flat surfaces in order to lessen the influence of geometric nonlinearity of the geosynthetics. The internal horizontal frame elements might not be required in this case but vertical and horizontal side elements would be necessary to carry the weight of the empty containers before the placement of fill materials. From the construction and functionality standpoints, cylindrical containers would be the most appropriate configuration for GCS applications (called CGCS) as this provides a uniform horizontal hoop stress across the cylinder at each elevation.

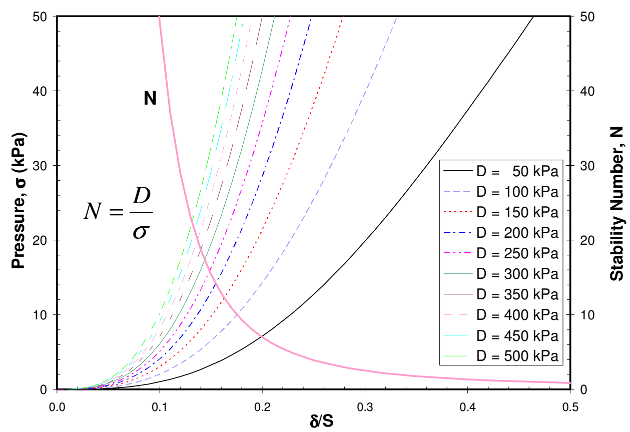


Figure 4. Variation of normal stress and stability number N versus deformation ratio for a flat membrane with various total stiffness (D) using the Simple Method

Based on the same argument given for the analysis of flat geosynthetics in GCS structures, the simple method can similarly be applied to estimate the maximum hoop force and lateral deformation of cylindrical geosynthetics. Using the same two equations of equilibrium and deformation and neglecting the boundary effects (for details, see Ghiassian and Holtz 2005), expressions can be written for a cylindrical geosynthetic with modulus E, thickness t, and initial radius R_o which undergoes uniform lateral deformation δ under internal pressure σ. Figure 5 shows the results presented in terms of the deformation ratio δ/R_o and stability number N.

A comparison between Figs. 4 and 5 reveals an important fact that flat membranes can stand surface pressures with much more controllable and reasonable deflections than cylindrical membranes, even at very small values of D (i.e., very soft geosynthetics and/or large dimensions). Cylindrical membranes, on the other hand, appear to be less deformable at larger values of D than flat geosynthetics, but this trend changes rapidly with the decrease of D such that much larger deformation occurs in the geosynthetic with smaller D. In other words, CGCS structures appear to be more rigid at larger D values and become increasingly prone to instability at smaller values of D.

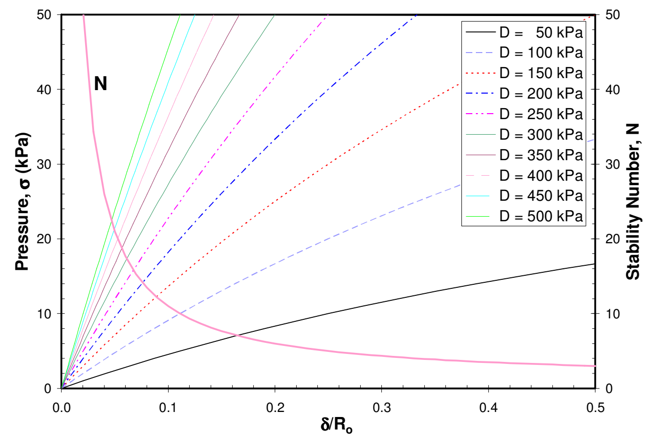


Figure 5. Variation of normal stress and stability number N versus deformation ratio for a cylindrical membrane with various total stiffness (D) using the Simple Method

The same trend is seen for the effect of pressure. At small pressures, the rate of deformation ratio change is smaller than that at high pressures. The combined effects of pressure and geosynthetic stiffness for both flat and cylindrical membranes can be shown when the stability number N is plotted versus deformation ratio as shown in Fig. 6. The predicted deformation ratio for flat membranes at N=50 is about 0.1 whereas this value for cylindrical membranes is about 0.02. Conversely, at small N, e.g., N=2 these ratios become approximately 0.3 and 0.5, respectively. Therefore, it can be concluded that flat membranes behave more flexibly at higher N values, but the opposite is true for cylindrical membranes--they are more flexible at lower N values. The transition value in the figure for the deformation ratio appears to be about 0.22 corresponding to stability number of about 5.5.

The results of the small deformation elastic theory (SAP analysis) were compared to the simple method for the following CGCS example. A cylindrical container has height of 6 m, and diameter of 3 m. The internal earth pressure coefficient for a friction angle of 30° is estimated from Eq. 1 to be 0.33. The geosynthetic is assumed to be a Geolon HS1715 (Mirafi) with the following properties: E=1230000 kPa, t=0.002 m, ν=0.45.

The results show that maximum lateral deflection occurs near the bottom and has a magnitude of 1.9 cm. The maximum hoop force (F11) is 34.9 kN/m and maximum vertical force (F_{22}) is 8.8 kN/m, both occurring very near the bottom of the structure. If the Poisson's ratio (ν) of the geosynthetic is changed to 0.3, these results are 1.9 cm, 33.4 kN/m, and 5.2 kN/m, respectively. Therefore, the effect of higher Poisson's ratios will be a slight increase of tensile forces. For geosynthetics, values between 0.45 to 0.49 for ν are reasonable, as these materials will undergo negligible volume changes when strained.

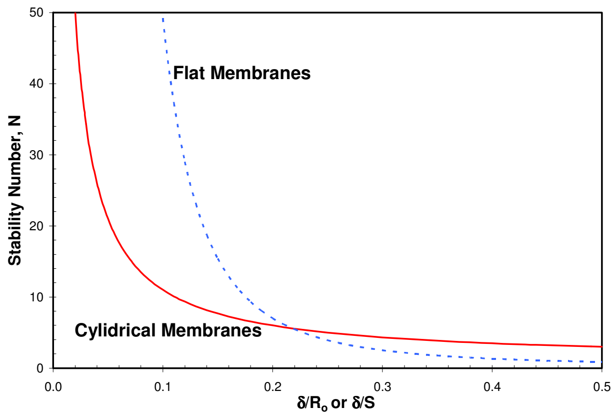


Figure 6. Stability number versus deformation ratio for cylindrical and flat membranes using the Simple Method

The simple method for this same CGCS gave a lateral deflection at the bottom of 1.85 cm and the maximum hoop force of 30.4 kN/m corresponding to the internal pressure of 20 kPa as shown in Table 2.

Table 2. Comparison between the results of the simple solution and the large deformation theory for a cylindrical geotextile with height of 6m, and diameter of 3m

Geotextile: Geolon HS1715 (Mirafi) E (Young's modulus) = 1230000 kPa t (thickness) = 0.002 m σ = 20 kPa R_o = 1.5 m
Deformation Simple Solution $\delta = \frac{\sigma R_o^2}{Et - \sigma R_o}$ (see Ghiassian and Holtz, 2005) $= \frac{20 \times 1.5^2}{1230000 \times 0.002 - 20 \times 1.5} = 0.0185$ m = 1.85 cm
Tensile Force Simple Solution $\Rightarrow T = \sigma R = 20 \times (1.5 + 0.0185) = 30.4$ kN/m $N = \frac{1230000 \times 0.002}{1.5 \times 20} = 82$

The above analysis shows that the results of lateral deformation and maximum tensile force determined by elastic theory are approximately 3% and 15%, respectively, higher than the simple method's results. The trend in fact is similar to that for flat membranes but much less pronounced. Recall that the elastic theory results are not generally acceptable for flat geosynthetics because the membrane behaviour of the geosynthetics is not appropriately considered. The above example shows that the sensitivity of predicted deformation and forces in the geosynthetic to the method of analysis has greatly diminished due to the initial curvature of the cylindrical geosynthetic. Another word, for flat membranes, the sensitivity is so high that the elastic solution gives very erroneous results. However, for initially curvilinear surfaces, the influence of membrane behaviour on the results is significantly decreased so that the results from two methods are in quite close agreement. As demonstrated in the above example, for strong geosynthetics with high stability numbers, the elastic solution may give higher values for deformation and forces than those from the simple method. However, for softer materials, the elastic theory gives smaller values than those predicted by the simple method (This was also observed in experimental studies presented in Section 5).

A conservative recommendation for situations where the rigorous or computer analysis of large membrane deformation is not feasible, it is suggested that two approaches of the small deformation elastic theory and the simple method be used for analyzing CGCS structures. Then, the larger values for deformation and corresponding tensile forces obtained from these two methods are selected for design. In the simple method, the maximum lateral stress corresponding to the bottom of the cell is used in the analysis.

3.3 Frame

It is assumed that elastic theory is applicable to the analysis of frame elements in GCS. Therefore, any computer program based on elastic theory and small deformations can be used. The soil loads are applied directly to the geosynthetics that are carried by the frame elements. As explained before, if geosynthetics behave as membranes, they cannot be analyzed by the elastic theory. However, the magnitudes of the loads supported by the frame elements through the geosynthetics are not influenced by the geosynthetic deformation. Therefore, the program can accurately analyze the frame elements for the design of different sections.

4 EXPERIMENTAL STUDIES

Some limited and preliminary experiments were performed to examine the validity of the above analysis and accuracy of the simple method in prediction of membrane behavior of geosynthetics.

4.1 Compression Tests

Soil specimens with 100 mm (4 in.) diameter and 225 mm (9 in.) height were prepared of Ottawa fine sand. The average diameter of the sand (D_{50}) was 0.26 mm, and the coefficient of uniformity of sand (C_u) was 1.56. All of the sand was finer than a No. 20 sieve. Dry sand was poured into the triaxial mold through a funnel with the spout near the soil surface in order to achieve a moderately loose state, a state considered to model likely field conditions.

Dry density of specimens varied between 1.59 and 1.60 g/cm^3, corresponding to a relative density D_r about 43%.

Three triaxial compression tests were performed under small confining pressures provided with vacuum at 100, 165 and 210 mm Hg vacuum, corresponding to 33, 55, and 70 kPa, respectively. Corresponding friction angles based on the peak principal stress difference were 35.3, 35.6, 40.5 degrees. The failure mode in all tests was specimen bulging with a ductile stress-strain relationship that is usually obtained for loose sand. These results indicated that shear strength of specimens were apparently more influenced by the change of density resulting from the confining pressure rather than the effects of dilatancy, because larger friction angles have resulted from higher confining pressures. This means that the influence of confining pressure on the density of a specimen has to be considered to determine accurate friction angles. Because of difficulty in evaluating this effect for the analysis of CGCS specimens, which were tested at zero cell pressure, it was decided to evaluate the friction angle for each test from the angle of repose. By pulling up the membrane around the soil specimen and allowing sand to pour out and create a mound on the laboratory bench, the angle of repose could be measured. Several determinations gave values of an average of 31.5^o, smaller than the values obtained from the compression tests. These results confirm that the change of density due to the confining pressure had an important influence on the shear strength of loose sand specimens.

4.2 Tensile Tests of Geosynthetic

The mechanical properties of geosynthetics are required for both the elastic analysis and the simple method. The "geosynthetic" used in CGCS laboratory experiments was a triaxial rubber membrane that can excellently model the membrane behavior of geosynthetics in CGCS specimens. Three stress-controlled tensile tests were performed on specimens 100 mm diameter, 0.2 mm thickness, and 350 mm in length. Maximum strain was about 23%,.and the stress-strain relationship was linear although nonlinear behavior could be observed by straining the membrane to much higher values. The modulus of the membrane was determined as 1980 kPa.

4.3 Model CGCS Specimens

4.3.1 Tests with Water

In examining the behavior of rubber to model a geosynthetic, some experiments were first conducted using water as the fill material. Thus unlike soil that is based on some assumptions for the lateral earth pressure, the magnitude of the internal pressure on the membrane could exactly be determined with water. In addition, because water has no shear strength, the pressure is applied normal to the geosynthetic at all points and at any state of deformation. This condition agrees well with the assumption made in the simple method. The deformation pattern of the membrane was measured from photographs taken during and after each test. Based on these results, better interpretation of the theoretical predictions were possible. In addition, these results were used in analyzing the data of stability tests on sand as presented in the following section.

Figure 7 presents the photographs of the stability tests with water during different stages of the test. The comparison between theoretical (elastic theory and the simple method) predictions and measured values of lateral deformation are given in Fig. 8. The height of specimen was 31.4 cm and both ends were fixed. The top cap was released at the end of the test to observe its effects. It appeared to result in a slight increase in lateral deformation (case c).

As Fig. 8 shows, the pattern of deformation is quite different from that predicted by the two methods, which is anticipated. Obviously, the simple method is a crude method and is not expected to entirely capture the deformation pattern because the influence of boundaries is not incorporated in the method. Therefore, maximum deformation occurs at the bottom where maximum pressure exists. Elastic theory cannot predict well either because it is based on small deformation theory. However, it gives a better prediction for the pattern as well as the magnitudes of deformation at different elevations for case (a) where the water elevation is 21.3 cm corresponding to the stability number of 3.9.

(a) (b) (c)

Figure 7. Stability test of a circular container made of rubber membrane filled with water, a: fixed cap and water elevation 21.3 cm, b: fixed cap and water elevation 31.4 cm, c: free cap and water elevation 29 cm

As the water elevation increases to 31.4 cm, the stability number decreases to N=2.6. Consequently, the membrane deforms further from those predicted by the two methods (Figs. 8-b, 8-c). The elastic theory predictions for the deformation appear much smaller than the actual values compared to those of the simple method. It seems that if the deformation of the "geosynthetic" at the bottom is determined from the simple method, a good and conservative approximation may be obtained. One important point to be mentioned is that the membrane in this test has undergone to about 60% lateral strain, which is much higher than the range of strain, applied in the tensile tests. If the material nonlinearity at higher strains was incorporated in the simple method (with incremental loading), the results would be expected to improve. Nevertheless, for design purposes as mentioned before, it is recommended that first, appropriate geosynthetics should be selected such that stability number does not become small, say less than 5. Then, both methods of elastic theory and simple approach can be examined and conservative values of deformation and tensile force are determined and used for design.

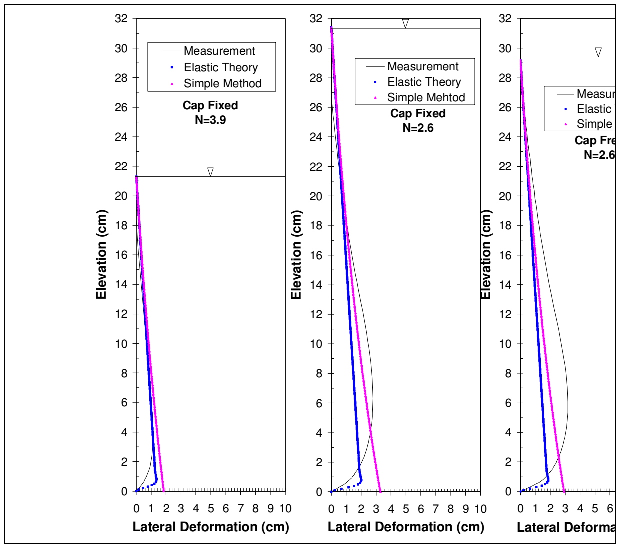


Figure 8. Comparison between theoretical predictions and measured values of lateral deformation in stability tests of a circular rubber membrane container filled with water

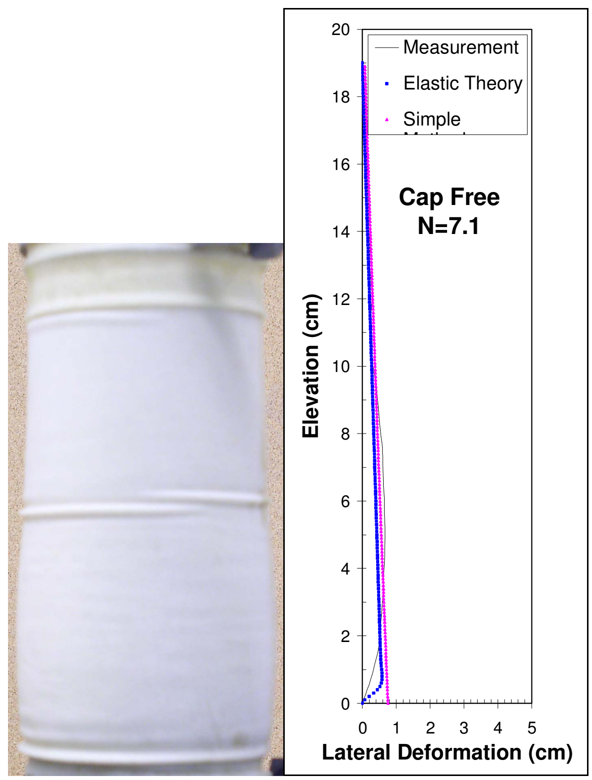
4.3.2 Tests with Sand

In stability tests on sand, sand specimens were prepared using the same procedure as used in the triaxial compression tests. In order to minimize the effect of the confining pressure on the density, only small vacuum (16 kPa) was applied, just enough to hold the specimen in place before running the stability test. Two tests were performed, and in each test, the specimen with 225 mm height, which was initially at a stable condition, failed after the vacuum was released. The failed specimens in fact reached equilibrium under the gravity load with the deformed membrane according to the large deformation membrane theory.

In this process, some settlement occurred in the sand column and the specimen height reduced to 18.9 cm. In addition, because of negligible rigidity of the rubber membrane, some wrinkles appeared in the specimens. Figure 9-a shows the specimen after reaching the equilibrium state between soil pressure and membrane deformation. This is likely to be the same behavior as in actual CGCS structures.

A comparison between theoretical predictions and actual measurements for the lateral deformation of the sand column is given in Fig. 9-b. The stability number was 7.1, larger than the values for water specimens because the lateral pressure has decreased from the hydrostatic to active earth pressure condition.

The results show that both methods predict well the maximum lateral deformation of 7.8 mm and 8.1 mm, respectively, for the elastic theory and simple method in comparison to the actual value of 6.7 mm. It should be noticed that the weight of cap has been considered in the simple method but not in the elastic solution; therefore, some small deformation was predicted at the cap elevation in the simple method. The deformation pattern near the bottom however is not captured by either of methods for the reasons explained before, although the elastic theory results agree better. Again, these results show that for situations where stability numbers are high (say larger than 5), both methods appear to be applicable which will result in a conservative design. For these conditions, if the system has structural elements, the small deformation elastic analysis can be used for designing all elements of the system including the geosynthetic. The experimental results also confirmed that the assumption of the active conditions for calculation of the earth pressure was correct.



(a) (b)

Figure 9. (a) Stability test of a rubber membrane circular container filled with sand, (b) comparison between theoretical predictions and measurements of deformation

5 CONCLUDING REMARKS

A new idea for the design and construction of cellular soil containers using geosynthetics and granular fill has been proposed and developed. The method, called

Geosynthetic Cellular Systems (GCS), uses a geosynthetic container system of different configurations that is backfilled with granular soil. GCS appears to have interesting advantages over conventional cellular cofferdams and similar structures in terms of constructability, cost effectiveness, and environmental considerations. In particular, the system can be used for coastal protective structures such as breakwaters, jetties, etc. where the riprap is replaced by granular soil. In addition, bioengineering techniques can be incorporated into the system to improve the appearance of the structure. CGS may be a viable alternative to sand-filled tubular structures sometime used to mitigate coastal erosion. Such structures can be either temporary or permanent, depending on the need.

Theoretical analyses of CGS were presented and discussed. Because the membrane behaviour of geosynthetics, particularly under high surface pressures, cannot be accurately analyzed by small deformation elastic theory, a simple method has been proposed that results in a more reasonable and conservative analysis of the system. Some limited laboratory experiments on small cylindrical specimens were also conducted and the results were compared to those predicted by the simple method as well as by elastic theory. More research is still ongoing on prototype models subjected to gravity and wave loads, utilizing the membrane theory to model the system realistically and to validate the simple method and its limitations.

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