APPLICATION OF GEOSYSTEMS AS RETAINING STRUCTURES IN HARSH MARINE ENVIRONMENTS

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Abstract

A retaining structure in harsher marine environment is subjected to severe hydrodynamic forces due to waves, tides and currents. The random loading and variable subgrade conditions further add to the already existing complexity. Normal soil reinforced walls may not be effective under such conditions as saturated soil conditions may considerably reduce the pullout resistance of reinforced soil. In such conditions, geosystems may be more effective as retaining structures. Geotubes and geocontainers may be used to construct marine structure as standalone or as a core of other geosystems. The increase in hydrodynamic forces may be considered for unforeseen catastrophic situations like risk of tsunamis, hurricanes, typhoons etc. A model based on structural stability criterion has been developed for reliability based risk assessment of retaining wall subject to hazardous risk. Dynamic stability of geotubes against hydrodynamic loads has been investigated. The reliability risk assessment model was applied in order to evaluate the structural safety of the retaining structure by modeling random design and structural variables i.e. wave height, tidal ranges storm surge, wave setup and the structural system parameters with probability distributions.

Keywords: Geosystem; Geotube; Tsunami; Monte Carlo method; Reliability analysis

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1. Introduction

Several retaining systems exist that can be used to prevent and protect from harsh marine environment built up of severe hydrodynamic forces caused by waves, tides and currents. The various systems may be reinforced soil walls, gabions, permanent steel structures, earth levees, concrete barriers and flexible geosystems. Flexible geosystems like geotubes and geocontainers are techno-commercially superior due to their versatility, technically efficiency and optimum use of scarce resources (Pilarczyk, 2000). Conventional rigid structures are often being ruled out as they are unsuitable for deformable subgrade, dynamic and random loadings and being hard on scarce resources. Reinforced soil wall may not be effective, as frictional mobilization tend to fail under saturated soil conditions, while passive resistance based mechanism like anchored wall and diaphragm walls may have their own problems. It is difficult to place and compact the soil fill in reinforced soil walls used for constructing quays, sea walls and waterway structures and special precautions are required to guard against washout (Jones, 1996). For structures constructed underwater, fill has to be hydraulically placed and there may be problem in placing the reinforcement and compaction. Geosystems on other hand can be placed very easily even in deeper water. Another simple flexible system traditionally popular has been sand bagging. However, it is labour intensive, expensive and has no reusable components but is able to act as successful water barrier. It may not serve its intended purpose in case of emergency like tsunami, as it requires considerable manpower, construction time, and a readily available supply of bags, filling material, shovels, and transport vehicles (Biggar and Masala, 1998).

A source of both ease with regards to installation and efficiency with regards to reuse, geosynthetic tubes and geocontainers are an economical alternative to sandbagging and other flood protection devices (Biggar and Masala, 1998). These geosystems can be used with locally dredged material, or a slurry mixture composed of concrete, sand or mortar. Geotubes can be used as retaining structures like breakwaters, dikes etc. for controlling harsh marine environment or as partial retaining ones like river training works, spurs, groins, levees etc. Dikes can be constructed up to several meters high to provide flood protection. They can also be attached to the top of a floodwall to provide greater flood control (Perry and Myers, 1993). A 15 km dyke of sand filled geotube of average height of 6 m was constructed in Leybucht, Germany. Being environment friendly and having little environmental impact due to environmental hazards, they are very suitable for use in environmentally dangerous areas, such as wetlands (Leshchinsky and Leshchinsky, 1996a; Leshchinsky, 1992; Sprague and Fowler, 1994). They can be used as prevention against beach erosion (Plaut and Suherman, 1998), for protection of tunnels and under water pipelines and for the holding of contaminated materials and pollution (Plaut et al., 1998). The land application of geotubes may be a case of dikes, bunds for land reclamation, toe protection or gryones, or inside water applications may be offshore breakwaters, sills of perched beaches, dikes for artificial islands, or for interruption of gullies caused by tidal currents (Pilarczyk, 2000).
2. 2-D analysis of geotube

The analysis of geotube filled with slurry or fluid under pressure is analogous to equilibrium of the encapsulating flexible shell as shown in Fig. 1. The analytical output provides geometrical configuration of shell and circumferential tension around it (Liu, 1981; Kazimierowicz, 1994; Carroll, 1994). The basic assumptions for simplified analysis are plane strain conditions, negligible weight per unit length, hydrostatic stress conditions within tube and absence of shear stress between fluid and geosynthetic. The material of geotube being polyester or polypropylene is considered to be extensible material. Cantré (2002) further advanced the geotube analysis by simulating the behaviour of stacked geotubes under load and drainage conditions using computer program ABAQUS (Hibbit et al., 1999).

The deformation of geotube could be written in form of nonlinear differential equation having no closed form solution and thus has to be solved numerically (Carroll, 1994).

\[ T \cdot y'' - (p_0 + \gamma \cdot x) \cdot [1 + (y')^2]^{\frac{3}{2}} = 0 \]  

(1)

where \( T \) being the geosynthetic tensile force in circumferential direction;

\( x \) and \( y \) being the horizontal and vertical coordinates;

\( y'' = \frac{d^2 y}{dx^2} \) being the second order differential;

\( y' = \frac{dy}{dx} \) being the first order differential;

\( p_0 \) being the pumping pressure of slurry into the tube;

\( \gamma \) being the unit weight of the slurry.

For obtaining explicit solutions, constraints will have to be imposed. Two physical constraints will replace two unknown parameters. One constraint is the geometrical boundary condition at top edge point (point O). The geosynthetic at this point should be horizontal for ensuring smooth transition from one half of the tube to other symmetrical part.

\[ \frac{1}{y'(0)} = 0 \]  

(2)

The second constraint can be imposed by specifying the flat base length \( b \):

\[ b = \frac{W}{p_0 + \gamma \cdot h} \]  

(3)
where \( W \) is the weight per unit length of slurry filling the entire section of the tube, and

\[
W = 2 \cdot \gamma \cdot \int_0^b y(x) dx
\]  

(4)

The circumference of the geotube, \( L \), can be specified as:

\[
L = b + \int_0^s ds
\]

(5)

where \( s \) represents the arc \( A_1O_2A_2 \) and \( ds \) being the differential arc length. \( ds = [1 + (y')^2]^{1/2} \) dx. After substitution in equations 3\textsuperscript{rd} and 4\textsuperscript{th}, the value of \( L \) can be expressed as:

\[
L = \frac{2 \cdot \gamma}{p_0 + \gamma \cdot h} \cdot \int_0^b y(x) dx + 2 \cdot \int_0^b \sqrt{1 + (y')^2} dx
\]

(6)

Thus relationship between \( T, h, p_0 \) and \( y(x) \) can be obtained for specified \( L \) value. Various computational methods have been specified (Liu, 1981; Kazimierowicz, 1994; Carroll 1994; Leschinsky and Leshchinsky, 1995,1996) as numerical solution involving trial and error procedure is tedious.

The axial tensile force per unit length, \( T_{\text{axial}} \), is required for designing of geosynthetic for geotubes:

\[
T_{\text{axial}} = \frac{2}{L} \cdot \int_0^b \left[ p_0 + (\gamma \cdot x) \right] y(x) dx
\]

(7)

The experimental work was conducted by Liu (1981) for verification of analytical results. The results obtained were compared with results from program GeoCoPS (Leshchinsky and Leschinsky, 1996) and were found to be in good numerical agreement. Silvester (1986,1990) presented analytical results in nondimensional chart and table for a particular circumference of tube in which values for height, width, cross-sectional area, contact width base and axial tension can be determined for input values of circumference and pressure head. Kazimierowicz (1994) determined that the final shape of geotube is governed by tube height, highest pressure, self-weight and tensile forces along circumference of geosynthetic.

Leshchinsky et al. (1996b) formulated equations for calculating the stresses in geosynthetic material. They observed that a large increase in pumping pressure would only produce a small increase in the tube’s storage capacity. Large geosynthetic tubes filled with sand tend to take on a more noncircular shape as they become larger. A parametric study was conducted for studying the variation in solution of geotube using Mathematica 5 (Wolfram 1996). The circumference of the geotube was chosen as \( L=10m \), the unit weight of slurry relative to water was taken as 1.3 and all factors of safety on geosynthetic strengths were set to 1.0. Fig. 2 shows the effects of the specified circumferential tensile force of the geosynthetic on the geometry of the geotube. A perfect circular shape with
diameter equal to $D = L/\pi = 3.181$ m, the required $T$ or $(p_0)$ must approach infinity. However, at $T$ as low as 16.22 kN/m the height is 2.0 m, i.e. $h$ is 62.87% of maximum theoretical height $D$. If $T$ is increased to value of 98.66 kN/m, the resulting height becomes 2.88 m or 90.53% of $D$. The variation of height has negligible influence on the cross-sectional area or capacity of geotube.

The variation of effects of the pumping pressure on the geometry of the geotube is shown in Fig. 3. It was observed that there was significant increase in height $h$ at low pressures with small increase in $p_0$. After a critical value of 38 kPa, increase in height becomes insignificant, but requirement of geosynthetic strength increases exponentially. Thus, proper monitoring of inlet pressure is essential to avoid rupturing of geosynthetic. The effects of pumping pressure on both $T$ and $T_{\text{axial}}$ are shown in Fig. 4. This shows that it is economical to choose geosynthetic having an anisotropic strength as different forces ($T$ and $T_{\text{axial}}$) exist.

3. Design of geotube retaining wall considering dynamic loading

The conventional design of geotube wall considers only hydrostatic loading. However, in reality dynamic loading has to be considered. For this, wave characteristics, tidal range, storm surge, wave set-up and the structural system parameters (geotube and foundation part) are to be considered. Furthermore, additional risk due to tsunami, hurricane etc. has to be considered. The safety of geotube system was evaluated by modeling random load and resistance variables with probability distributions at their limit-state. The critical limit state considered was serviceability limit state for the safety evaluations, as the exceedance of the failure damage level may not result in complete collapse of the geotube system, but may not serve its intended function. The serviceability limit state can be defined in terms of tolerable deformations, settlements (total or differentials) and strains which can vary depending on ramification of failure.

The reliability analyses of geotube wall can be performed by using concurrent probabilistic methods which can result in development of some simple analytical tools. These simple design approaches can be employed where little or no test data is available due to resource constraint or rarity of phenomena or complexity of the problem. The reliability index concept is used in such situations where uncertainty in performance is related to uncertainty in underlying random variables influencing the performance. Given that probability of failure, $P_r(f)$ can be calculated for different performance modes and different significant wave heights, these values can be combined to provide the desired conditional probability of failure functions. Several modes of embankment performance can be developed as a function of significant wave height using a probabilistic capacity demand model. A conditional probability of failure function for each mode can be developed as a function of significant wave height by replicate analyses at different wave heights. Consequently, these individual conditional probabilities can be combined to develop a composite probabilistic function. The conditional probability of failure, $P_r(f)$ can be written as:

$$P_r(f) = P_r(f \uparrow H_s) = f(H_s, X_1, X_2, \ldots, X_n) \quad (8)$$
Symbol “↑” is read as “given” and the variable $H_s$ is the significant wave height. The random variables $X_1$ through $X_n$ represent relevant parameters such as soil strength, geotube weight, geosynthetic strength etc.

Various performance modes can be:

1. Geotechnical stability (Internal and Global)
2. Hydraulic stability (Stability against hydrostatic and hydrodynamic loads)
3. Structural stability (Safety against sliding, overturning, scouring)
4. Safety against thorough seepage
5. Stability against squeezing/Settlement
6. Geometry beyond the embankment toe

Only prominent modes are being discussed and rest can be similarly evaluated.

### 3.1. Geotechnical stability

As per limiting equilibrium theory for reinforced slope, the reason for slope instability of geotube embankment can be the disturbing moment, $M_D$ exceeding the resisting moment, $M_r$. The resisting moment is provided by the shear strength of soil and presence of geotextile in the slope. The mathematical model for calculating probability of failure of slope stability can be expressed as:

$$P_f = P(M_D > M_r) = \int_{M_r}^{\infty} f(M_D) dM_D$$

where $f(M_D)$ is the function of probability density distribution of disturbing moment. It is difficult to evaluate probability of failure directly from above expression and discretization helps in the solution.

$$P_f = \sum_{i=1}^{N} F_D(H_{iD}) \Delta F_0(H_{iD})$$

where $F_D(H_{iD}) = \int_{M_r}^{\infty} f\left(\frac{M_D}{H}\right) dM_D$ is the probability that disturbing moment, $M_D$ exceed restoring moment, $M_r$ at significant wave height. $\Delta F_0(H_{iD})$ is the probability of segment, $i$ in the frequency exceedance curve of significant wave height, $N$ being the number of segments of curve of significant wave height and also the frequency that needs to be calculated. In general, analytic solution of integration of $F_D(H_{iD})$ can not be obtained easily as $f(M_D/H)$ in $\int_{M_r}^{\infty} f\left(\frac{M_D}{H}\right) dM_D$ is a very complex function, but it can be calculated easily by Monte Carlo method. The main steps involved are as follows:

1. The minimum safety factors of slope instability caused by destabilizing forces (i.e. lateral earth pressures, forces due to hydrostatic and hydrodynamic effects) are determined by any of these methods: two part wedge method or slice method for circular slip analysis or conjugate stress analysis or log spiral analysis. The
serviceability limits basically are governed by post-construction movements of embankment. They may occur due to foundation settlements, internal compression of fill, internal creep strain of reinforcement, uniform or differential settlements resulting from subsidence, creep strain of soils with high fines content. They should be restricted to certain values based on ramification of failure.

2. The generation of pseudo random numbers is done for producing random sampling of geotechnical and geosynthetic parameters like $c, \phi, \gamma$ etc. is done.

3. The disturbing moment, $M_D$ and restoring moment, $M_r$ is calculated for any particular surface or slip circle using any standard slope stability software.

4. The number of cases when $M_D > M_r$ is counted and recorded as $m$.

5. The steps from 2nd to 4th are repeated $n$ times till convergence has been obtained.

6. According to Bernoulli’s theorem and characteristics of normal distributed random variable, $F_D(H_D) = \frac{m}{n}$ is obtained.

3.2. Hydraulic stability

For the reliability based risk assessment, the second order reliability index ($\beta$ II) method and the Conditional Expectation Monte Carlo (CEMC) simulation were utilized for analyzing the safety levels of geotube retaining wall against hydraulic loadings. By adopting these simulations, the uncertainties associated with most of the design variables were incorporated throughout the life of structure (120 years). The life of geotubes can be enhanced by using multilayered system. The safety of geotube retaining wall was evaluated by modeling random loading and resisting strength variables with probability distributions at their limit-state. The primary variable vector $z$ in the normalized space defined the random variable. The failure surface was approximated by a rotational parabolic surface, which is similar to real situation as shown in Fig. 5. Though, ideal shape is shown to be trapezoidal for assumptions of various geometrical dimensions of wall, the real shape will be parabolic due to curved profile of partially filled geotubes. The parabolic limit state in standard normal space may be represented by the structural performance function of $g(z)$ (Balas and Ergin, 2000):

$$g(z) = a_0 + \sum_{i=1}^{n} b_i z_i + \sum_{i=1}^{n} c_i z_i^2$$

where $a_0$, $b_i$ and $c_i$ are the regression coefficients of the second order polynomials; $z_i$ are the standardized normal random variables and $n$ is the number of random variables.

The conditional expectation Monte Carlo (CEMC) simulation was employed to analyze the soil-structure interaction and performance of the geotubes system under specified loading conditions. The probability of failure, $P_{rf}$ was obtained as function of control random variable vector of $z_i = (z_{i1}, z_{i2}, ..., z_{ik})$ as follows (Ergin and Balas, 2000):
\[ P_{ij} = E[z_{ij} | z_{ij} \neq z_i, z_{j} = z_{j1}, z_{j2}, \ldots, z_{j_k}] \]  \hspace{1cm} (12)

where \( E[\cdot] \) is the mean conditional expectation and \( P_{ij}(z_i) \) is evaluated for \( z_{i1}, z_{i2}, \ldots, z_{ik} \), by satisfying the conditional term for the last control variable as follows:

\[ P_{ij}(z_{ik}) = \Pr \{ z_{ik} < g_{ik}(z_j : j = 1, 2, \ldots, n; n \neq i) \} \]  \hspace{1cm} (13)

where \( k \) is the number of control random variables in the simulation.

The control random variable for the structural performance of geotubes system against hydraulic loading was the significant design wave height following the Gumbel extreme value distribution. For a selected design wave of certain height, storm surge, tidal level and tsunami waves were randomly generated (on average 50,000 times) from the probability distributions for estimating the hydraulic design load of the geotubes system. The reliability of loading condition was investigated (on average 50,000 times) by the \( \beta_{II} \) method at the limit state conditions. Thus, for each randomly generated loading condition, the composite damage probability represented both the occurrence probability of loading conditions and exceedance probability of the limit state (i.e. damage level of the geotube structure).

### 3.3 Analysis of risk for geotube wall

The structural safety of proposed geotubes wall was analyzed by modeling random loading and design variables i.e. wave height, storm surge, tidal variation, wave set-up and the structural system parameters like geosynthetic tensile strength, subgrade characteristic, dimension of geotubes system, fill characteristic etc. as shown in Table 1. The distribution parameters of basic variables for reliability-based risk assessment were adopted as shown in Table 2.

The height of the geotube wall was kept as 10 meters. Individual geotubes of 3.2 meter diameter were connected with straps (geosynthetic friction ties) to form a slope of 1:3. The significant design wave height and period were taken as \( H_s = 10 \) meter and \( T_s = 5 \) sec obtained from extreme value computations. Maximum tsunami height was taken same as the design height i.e. 10 meters. The maximum horizontal extension of the inundation produced by the tsunami will follow the trend shown in Fig. 6 (adopted from Farreras, 2000).

Various attempts have been done towards quantification of tsunami in terms of intensity scale, occurrence probabilities and magnitudes (Sieberg, 1927; Ambraseys, 1962; Imamura, 1942,1949; IIADA, 1956, 1970, Abe, 1979, Shuto and Matsutomi, 1995). A sample statistical analysis, as proposed by Ergin and Balas (2002) in terms of probability of occurrences of the mean values of the ranges was adopted and further extrapolated to include higher tsunami heights (Fig. 7). A certain confidence limit (represented by dotted lines in Fig. 7) indicating the tsunami height ranges was attributed to regression line (represented by solid line in Fig. 7), presenting the statistical characteristic of the tsunami mean values. In simulation process, the generated tsunami heights were converted to the construction depth of geotube wall by presuming near-shore attenuation, the fact practically
observed in recent tsunami. For this transformation, Cnoidal theory was utilized. A simple approximation to exact Cnoidal theory developed by Iwagaki et al. (1982) was adopted:

\[
\frac{H}{H_0} = \left(1 + \frac{z_w d}{\sinh w_d}\right) \tanh w_d + 0.0015 \left(\frac{d}{L_0}\right)^{-2.8} \left(\frac{H_0}{L_0}\right)^{1.2}
\]

(14)

where \(H_0\) and \(L_0\) are the offshore wave height and wave length respectively; \(w\) is the wave number; \(H\) is the local wave height; \(d\) is the local depth of geotubes wall.

The approximate Cnoidal theory was used to find local wave heights in front of the geotube wall. The tsunami height ranges based on structural risk assessment were adopted as shown in Table 3. Based on the occurrence of tsunamis elsewhere (local data being absent) the return period of tsunami heights were estimated as shown in Table 4. It is worth noting that reliability of tsunami model depends on the number and accuracy of data, time period and statistical technique being used.

The generation of input data like offshore wave height and the offshore tsunami height were generated by the CEMC simulation from the extreme value probability distribution of FTT-I and the probability distribution given in Fig. 7 (adopted from Ergin and Balas, 2002). The stability of geotube wall was investigated by the \(\beta_{11}\) method. The structural performance function \(g(z_i)\) for geotube wall against hydraulic loading was simulated under design conditions using probability distributions for two cases:

Case I: Tsunami risk not considered

Case II: Tsunami risk being considered

The scatter range of the randomly generated values was between \(g_{\text{min}} = -55.52\) m and \(g_{\text{max}} = 5.62\) m for case I; \(g_{\text{min}} = -56.87\) m and \(g_{\text{max}} = 6.47\) m for case II. This shows the influence of uncertainties prevalent in the design parameters of the limit state functions having the simulated mean values of \(\mu_g = 1.085\) m (Case I) and \(\mu_g = 1.097\) m (Case II) (Table 5). For comparison, the failure probabilities for both cases were plotted in Fig. 8. Effect of geosynthetic failure due to degradation was not considered, as it will remain same for both cases. It was observed:

- The failure probability and the influence of tsunami risk on the failure mechanism increase with the project life.
- The failure probability for case II (inclusion of tsunami risk) is always higher than case I (tsunami risk not included)

The life of geotube wall may be limited by life of geosynthetic, which may be due to various factors (Davis and Landin, 1997, 1998):

- Geosynthetic resistance to puncture and abrasion
- Geosynthetic degradation in the environment (UV degradation, chemical exposure etc.)
- Lack of precise hydraulic, hydrodynamic and geotechnical design guidance.

The data regarding the fabric degradation rate due to natural UV (Ultraviolet) light is very scanty. Though laboratory tests by exposing geosynthetics to intense UV degradation have been conducted and the results suggest fair resistance to UV degradation. However, extrapolation of results to actual field condition is difficult. However, the effect of UV can be considerably reduced or eliminated by submerging or covering of geotube by sediments, rock pieces or marine growths. The geotubes may be damaged by puncturing or abrasion. Even a very high strength geosynthetic (tensile strength higher than 1000 kN/m) could be punctured by a sharp object like knife. Various factors like vandalizing by public, ice or debris forced against the geotubes by waves or currents, damage during shipping, handling and erection, may cause puncturing or abrading the material. Various methods have been proposed for estimating stability of geotube structures. Yet they are not fairly accurate. It was suggested that U. S. Army Corps of Engineers (SPM, 1984) that methods proposed by Goda (1985) for predicting loads on vertically faced structure could be used. The resisting forces (bed friction and weight) can be estimated and a force balance would indicate whether structure is likely to move against wave and current loading. Additional force due to tsunami can also be added. The frictional analysis suggested at US Workshop on geotextile tube applications (Davis and Landin, 1997, 1998) were 18 degree for fabric on sand. For higher loading situations, geotubes can be connected with strips and anchored to the ground. Waterways Experiment Station (WES) advocates use of discrete element model for simulation of deformation of lube in two dimension cross section in loading. A graphical technique was advocated by Sprague (1995) for estimating the strength of fabric. He also presented a technique for selecting the spacing of inlet parts along the crest of the tube.

A geotube under severe hydrodynamic conditions is subjected to drag, inertia and lift forces, which can reduce tube stability drastically. Thus, higher restoring forces are required to maintain stability. Hydrodynamic loading should also be accurately determined. The various hydrodynamic loadings may be due to following reasons:

- Non breaking waves on a fully submerged tube
- Non breaking waves on a non-submerged tube
- Breaking waves on a non-submerged tube
- Current forces
- Forces due to tidal variation
- Tsunami forces

The internal stability of geotube and overall stability of geotube systems have to be considered. Various model studies conducted (Delft Hydraulics/Nicolon, 1994) showed that degree of fill of the sand tubes is an important parameter for stability. Though, it may be possible to fill geotubes to 95% of theoretical capacity, but it becomes less stable due to
circular shape and may require high tensile strength. From field observations, the average height is kept 2/3 of the theoretical diameter.

A schematic layout of failure mechanisms of stacked geotubes is shown in Fig. 9 (ACZ, 1990). The internal stack stability and hydraulic stability may be maintained by strapping individual geotubes and anchoring the geosystem with subgrade. The design of geotube wall is an interactive process, in which stability of system and functional requirements (desired function of reduction of incoming waves/proper transmission coefficient) are to be satisfied. Based on that, width and height of the geotube system is decided. The low wave transmission (requisite wave reduction) is an important requirement of geotube wall. Not much work has been done on transmission characteristics of geotubes. The results based on Danish Hydraulics (DHI, 1970) are shown in Fig. 10. Instead of using steeper slope geotube system, gentle slope helps in reducing wave transmission. Two parallel geosystems with some distance are more effective in reducing wave transmission. The creation of longitudinal reef system composed of geotube system placed perpendicular to the coast is also an effective system (Goda, 1995, 1996).

The shape of individual geotube depends on the static head of fill material (sand, slurry or liquid concrete), laying method and behaviour of fill material inside the geotube. The geosynthetic is conceptualized as impervious flexible shell and fill material as liquid of certain density. There are already design graphs developed by Silvester (1990) and numerical design by Dov and Ora Leshcinsky (1995), which show a good agreement with experimental data by Liu (1981). The height of final geotube wall can be calculated by using these methods.

In case of geotubes placed with their longitudinal axes parallel to the geotube wall, the critical wave height is nearly equal to theoretical dimension of geotube. In case of longer wave periods, hydrostatic pressure acting at front of top tube will be the governing factor. The contact point will be the bottommost point of topmost geotube. The disturbing moment, \( M_d \) (hydrostatic loading), around contact point, against the top layer due to wave pressure will be (Pilarczyk, 2000):

\[
M_d = \frac{3}{4} (1 + k)d \frac{1}{2} H^2 P_w g
\]

where \( k \) is the reflection coefficient, \( d \) is the height of geotube wall. \( H \) is the wave height, \( P_w \) is the hydrostatic pressure of fluid, \( g \) is acceleration due to gravity.

The restoring moment, around contact point, owing to the weight of top tube, is equal to

\[
M_r = A (P_c - P_w) g \frac{1}{2} B
\]

where \( P_c \) is the geocontainer pressure, \( A \) and \( B \) are the constants.

For maintaining static equilibrium,
\[ H < \frac{8}{3} \frac{A}{1} B \frac{\Delta}{h_l^2(1+r)} \]  

(17)

where \( \Delta = \frac{(P_c - P_w)}{P_w} \)  

(18)

Assuming \( A = C_1 h_l B \) and \( h_l = C_2 B \), where \( C_1 \) and \( C_2 \) being the constants.

The equation can be modified as

\[ \frac{H}{\Delta B} = \frac{C_1}{\frac{3}{4}(1+k)C_2} \]  

(19)

Using values from the report (DELT HYDRAULICS, 1973),

\[ R = \text{Reflection coefficient} = 0.45 \]  
\[ C_1 = 0.78 \]  
\[ C_2 = 0.65 \]

The stability will be \( \frac{H}{\Delta B} = 1.1 \)

Similarly, the equilibrium expression for a tube placed in direction of wave propagation will be:

\[ \text{Wave loading: } \frac{3}{4}(1+r)HAP \]  

(20)

\[ \text{Friction force: } f\Delta l(P_c - P_w)g \]  

(21)

where \( f \) is the coefficient of friction between the tubes and \( l \) is the length of geotube.

\[ \frac{H}{\Delta l} = \frac{f}{\frac{3}{4}(1+r)} = \frac{f}{1.1} \]  

(22)

Available list results show value of \( \frac{H}{\Delta l} \) to be nearly 1.0.

Similarly, resistance of structure to horizontal displacement can be derived

\[ fG \geq F \]  

(23)

where \( f \) is the interlocking (contact) coefficient, \( F \) is the wave pressure force and \( G \) is the submerged weight of structure. The most important sliding force in offshore conditions is
caused due to hydrodynamic forces. The most critical is the wave pressure force especially in tsunami type situations can cause external instability in the structure. The same force caused by random wave loading can be unpredictable and the same can be determined with the local sea wave data available for 50 to 100 years or by using reliability analysis in case of lack of proper data.

4. Conclusions:

The possibility of replacing conventional retaining wall systems with geosystems made of geotubes was explored. The dynamical aspects of geotube systems against current and wave attack were also investigated. The hydrodynamic loads were randomly generated by simulation and inclusion of additional loading due to unforeseen events like tsunamis was also considered. In the reliability based risk assessment, second order reliability index method was used, in which the failure envelope was approximated by rotational parabolic surface. The effect of tsunami was also considered in the structural assessment risk model. The probability of failure mode was predicted by the parabolic limit state surface having identical profile at the design point with additional risk consideration of tsunamis. The reliability approach was given precedence over deterministic approach due to random nature of hydrodynamic loading and consequent random behavior of structural performance and the same could be determined at conceptual stage itself. The suggested reliability approach can handle the uncertainties and inherent in hazardous situation, storm surge and wave data. The reliability method has advantages compared to the deterministic approach, since the random behavior of structural performance can be estimated at the planning stage itself.

It was observed that the impact of hazard risk on the failure mechanism becomes more critical with the increase in design height of the retaining structure. The serviceability state was found to be more critical than stability state, as the exceedance of the failure damage level may not result in complete breakdown of structure, but may not serve its intended design functions. The new reliability approach can be used more effectively within few minutes for design of retaining structure. It is advisable to include hazard risk in reliability-based model, since it enhances the failure risk as risk parameter. This is more critically significant in greater hazard prone-coasted areas where magnitude of tsunami or other hazard and its occurrence probability in on higher side.

The approach being numerical can be easily quantified with high speed computers and can give fair estimate of geosynthetic properties and configuration of geotube and stack formation. Inclusion of tsunami risk has become important as tsunami may occur even at places thousands of kilometers away from the place of its origin. The assessment of structural risk incorporating uncertainties and randomness of hydrodynamic loading may help in devising early risk mitigation scheme for important coastal projects, such as naval ports, nuclear power installation, and human settlements.

There is still lot of uncertainties in the existing design methods due to complexities and uncertainties involved in various loading conditions. Thus, further refinements in design techniques, use of reliability methods and practical experience under different loading conditions is required for satisfactory prediction of response of geosystems. However, present approach regarding prediction of loads, stability computations at planning stage will
be of much assistance for preparing preliminary design of geosystems for unpredictable hazardous loading situations.

5. References


Sprague, C. J., and Fowler, J. (1994), "Dredged material-filled geotextile containers:


**Nomenclature**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A,B</td>
<td>Constants in restoring moment equation</td>
</tr>
<tr>
<td>a₀</td>
<td>Regression coefficient of second order</td>
</tr>
<tr>
<td>b</td>
<td>Flat base length of Geotube</td>
</tr>
<tr>
<td>bᵢ</td>
<td>Regression coefficient of second order</td>
</tr>
<tr>
<td>β</td>
<td>Reliability index</td>
</tr>
<tr>
<td>βᵢᵢ</td>
<td>Reliability index of second order</td>
</tr>
<tr>
<td>c</td>
<td>Cohesion</td>
</tr>
<tr>
<td>cᵢ</td>
<td>Regression coefficient of second order</td>
</tr>
<tr>
<td>C₁,C₂</td>
<td>Empirical Constants</td>
</tr>
<tr>
<td>d</td>
<td>Depth of geotube wall</td>
</tr>
<tr>
<td>dₛ</td>
<td>Differential arc length</td>
</tr>
<tr>
<td>dₜ</td>
<td>Water depth</td>
</tr>
<tr>
<td>Dₙ</td>
<td>Nominal dia of geotube</td>
</tr>
<tr>
<td>Δᵢ</td>
<td>Relative density</td>
</tr>
<tr>
<td>E</td>
<td>Mean conditional expectation</td>
</tr>
<tr>
<td>f</td>
<td>Coefficient of friction between geotubes</td>
</tr>
<tr>
<td>F</td>
<td>Wave pressure force</td>
</tr>
<tr>
<td>F(Mₜ)</td>
<td>Probability density distribution function</td>
</tr>
<tr>
<td>g</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>g(z)</td>
<td>Structural performance function</td>
</tr>
<tr>
<td>G</td>
<td>Submerged weight of structure</td>
</tr>
<tr>
<td>γ</td>
<td>Unit weight of fluid</td>
</tr>
<tr>
<td>h</td>
<td>Height of geotube</td>
</tr>
<tr>
<td>H</td>
<td>Local wave height</td>
</tr>
<tr>
<td>H₀</td>
<td>Offshore wave height</td>
</tr>
<tr>
<td>Hᵢ</td>
<td>Incident wave height</td>
</tr>
<tr>
<td>Hₛ</td>
<td>Significant wave height</td>
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<tr>
<td>Hₜ</td>
<td>Transmitted wave height</td>
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<tr>
<td>Hₚ</td>
<td>Tsunami wave height</td>
</tr>
<tr>
<td>k</td>
<td>Number of control random variables in simulation</td>
</tr>
<tr>
<td>l</td>
<td>Geotube length</td>
</tr>
<tr>
<td>L</td>
<td>Circumference of Geotube</td>
</tr>
<tr>
<td>L₀</td>
<td>Offshore wave length</td>
</tr>
</tbody>
</table>
m  Number of failure trial cases
M_D  Disturbing moment
M_o  Overturning moment (hydrostatic)
M_r  Resisting moment
\mu_g  Simulated mean values of standard performance function
n  Total number of trials, number of random variables
N  Number of segments of curve significant wave height, frequency
p_0  Pumping pressure of slurry filling in geotube
P_c  Geotube pressure
P_r(f)  Probability of failure for any mode
P_w  Hydrostatic pressure of fluid
\phi  Angle of internal friction
R  Radius of curvature
R_T  Tidal range
T  Geosynthetic Tensile force
T_{axial}  Axial tensile force per unit length
T_s  Time period of wave
w  Wave number
W  Weight per unit length of filled Geotube
W_G  Width of geotube
x  Horizontal Coordinate
X_i  Random variable
y  Vertical Coordinate
z_i  Control random variable