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Laboratory study of geosynthetic cellular system (GCS) models under wave action in flume

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Abstract

The concept of Geosynthetic Cellular Systems (GCS) has recently emerged as a new method in construction of breakwaters and coastal protective structures. The method potentially has significant advantages compared to conventional systems from the standpoint of constructability, cost effectiveness, and environmental considerations. This paper presents the results of physical model testing on the hydraulic responses of GCS structures under wave action. A series of model tests were carried out in a wave flume on GCS models with different shapes and soil types, subjected to various wave characteristics. Horizontal wave forces acting on the models were measured at different elevations. The maximum horizontal force in each test was calculated and compared with conventional formula of predicting wave pressure on breakwaters. The results show that Goda's equation overestimates the hydrodynamic water pressure on these structures. This can be attributed to the influence of seeping water through the GCS models because of relative permeability of the GCS.

Keywords: *Geosynthetic cellular system (GCS), Breakwater physical modeling, Flume, Hydrodynamic wave pressure.*

1. Introduction

The background of CGS is the anchored geosynthetic system (AGS) stabilization method but with extensive modifications [1]. 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 nonintrusive, environmentally compatible alternative to hard armour, which in many instances is prohibited in environmentally sensitive areas, e.g. coastal sand dunes and beaches.

AGS can also protect slopes against both internal seepage that promotes piping, and mass instability. Surface load applications on homogenous, granular slopes tend to change the failure mode from a planar slide assumed by the infinite slope theory to a more rotational sliding [2].

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 geosynthetic; therefore, they have to be re-stretched after some time [3]. 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 Figure 1 [4].

An interesting variation of this idea would be to construct AGS-type breakwaters by replacing the rockfill with dredged material. Such a system could be economical in situations where the required rockfill materials are either unavailable or very costly and dredged materials are readily available. 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 usually built similarly to other structural frames. After the frame is built, geosynthetics are placed around the interior of the frame and connected to the frame using, e.g. plastic ties. The frame is then transferred to the desired offshore location and allowed to sink. After the frame is positioned on the sea bottom, it is backfilled with either on shore materials or dredged soils.

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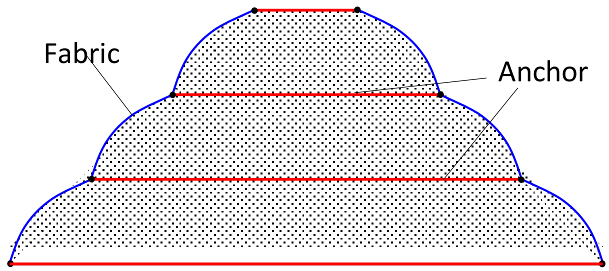


Fig. 1 Conceptual application of AGS to a levee or embankment with horizontal anchors [4]

Results of the analyses of CGS structures including internal stability considerations and the results of some laboratory experiments on small cylindrical models filled with water and sand are presented in elsewhere [4]. The behavior of GCS is compared with theoretical predictions, and a method called "Simple Method" is suggested to calculate deformation and internal forces in the GCS components.

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 [5], [6]. 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 [7].

To evaluate the ability of the GCS structures used as breakwaters, physical models of GCS breakwaters studied. The horizontal wave forces acting on the models were measured at different elevations at the front face of GCS models. Considering that the GCS breakwaters could not be categorized in either rigid type or flexible type breakwaters, in order to find the proper method for calculating the hydrodynamic wave pressure on structure for external stability check, the recorded hydrodynamic wave pressure on the GCS breakwater models are compared with conventional wave hydrodynamic pressure formula.

2. Experimental Setup

The experiments were carried out in the wave flume, at Water Research Institute Laboratory, WRI. The wave flume is 40 m long, 0.95 m wide and 1 m high with a piston type wave maker. To evaluate the response of GCS breakwater structures under wave attack, two geometries of square and trapezoidal sections have been studied (Figures 2). The square section model has height, width of 0.95 m and length equal to 1 m. The bottom and top length of trapezoidal section model was 1.5 and 0.5 m respectively and has height and width similar to the square section model.

(a) (b)

Fig. 2 GCS breakwater models Studied in wave channel, a. Square section, b. Trapezoidal section

The frame of models has been constructed using steel profiles, and the TS-40 of Tencate/Polyfelt nonwoven geosynthetic products has been used as proper fabric. The strength properties of fabric have been designed by Simple Method [4] and hydraulic properties requirements designed based on [8].

GCS breakwater models have been filled by two types of materials concluding Firouzkooh 161 Sand and Silt materials with grain size distribution illustrated in Figure 3. The method of Water Sedimentation has been used in order to fill the models to have uniformity in density in whole height of models [9].

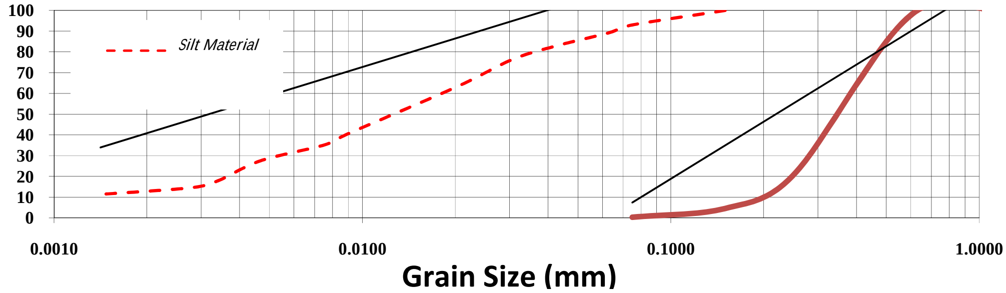


Fig. 3 Grain Size Distribution of Firouzkooh 161 Sand and Silt as filling materials

For the measurements of wave pressure on the models, four pressure gauges along the sea-side wall were deployed. Figure 4 shows their locations, and the pressure gauges were labeled respectively s HP-1 to HP-4 from toe to top along the face of model. In order to evaluate the wave characteristics, a capacitance wave gauge was set right on the location of the model.

Table 1 show four models in which tested under wave action in the experiments. In the Table ($H_{1/3}$) and (T) are the significant wave height and its related period respectively.

Table 1 Experimental Models and Wave Characterization

Test No.	Geometry	Filling Material	Wave Height (H_s , m)	Wave Period (T, Sec.)
GCS-1	Square section	Firouzkooh 161 Sand	0.308	1.83
GCS-2	Square section	Silt	0.315	1.83
GCS-3	Trapezoidal section	Firouzkooh 161 Sand	0.314	1.72
GCS-4	Trapezoidal section	Silt	0.305	1.98

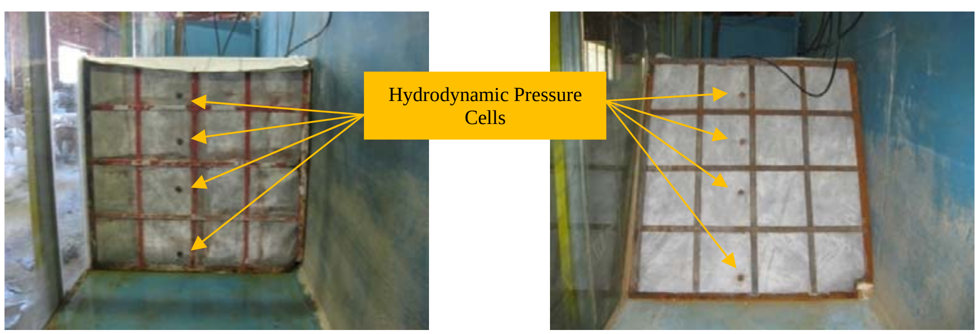
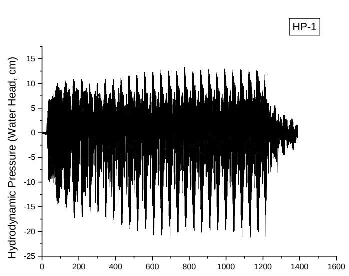
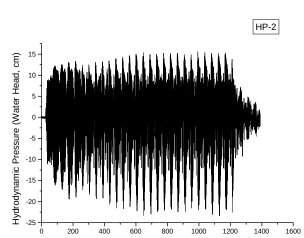


Fig. 4 Arrangement of hydrodynamic pressure gauges in front of square section and trapezoidal section GCS models

3. Experimental Results

Figure 5 shows the profiles of horizontal pressure on the model in GCS-1. Considering the water depth and wave height, the water elevation during the wave trough action might below the locations of gauges HP-3 and HP-4 and causes these gauges obtained incomplete pressure profiles and zero pressures as water level below their locations.





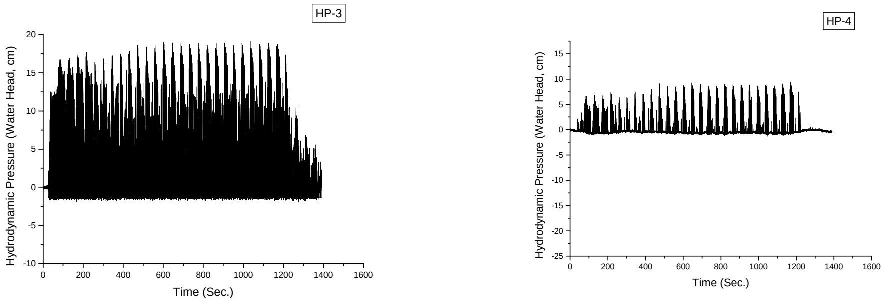


Fig. 5 The profiles of horizontal pressure on GCS-1 model

The resultant hydrodynamic force during test has been calculated in each data points, and the maximum load has been attained equal to 1.188 kN/m in time 260.405 s of test. The profile of hydrodynamic wave pressure related to corresponding time is illustrated in Figure 6.

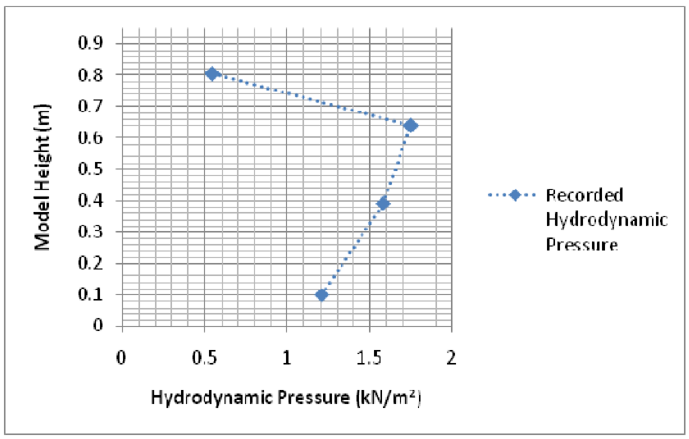
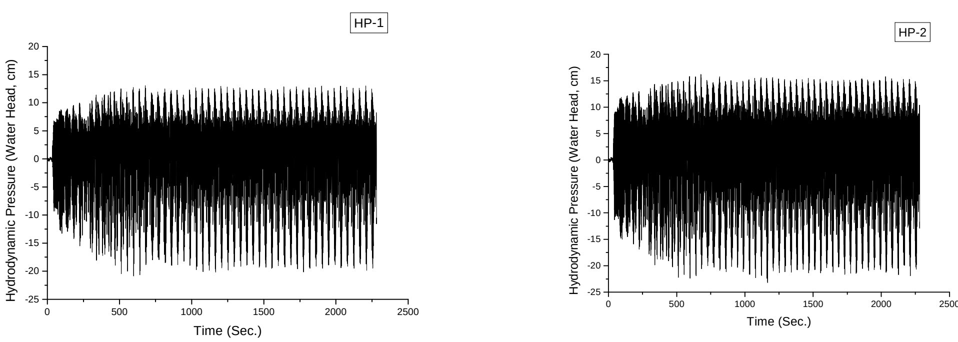


Fig. 6 Hydrodynamic wave pressure profile at the time of maximum resultant force in GCS-1

In Figure 7 the profiles of horizontal pressure on the GCS-2 model are illustrated. In this model, as like as GCS-1, the water elevation during the wave trough lowered below the locations of gauges HP-3 and HP-4 and causes these gauges obtained incomplete pressure profiles and zero pressures as water level below their locations. Figure 8 shows the hydrodynamic wave pressure profile at the time of maximum resultant force, which has been obtained equal to 1.198 kN/m at 679.88 s time.



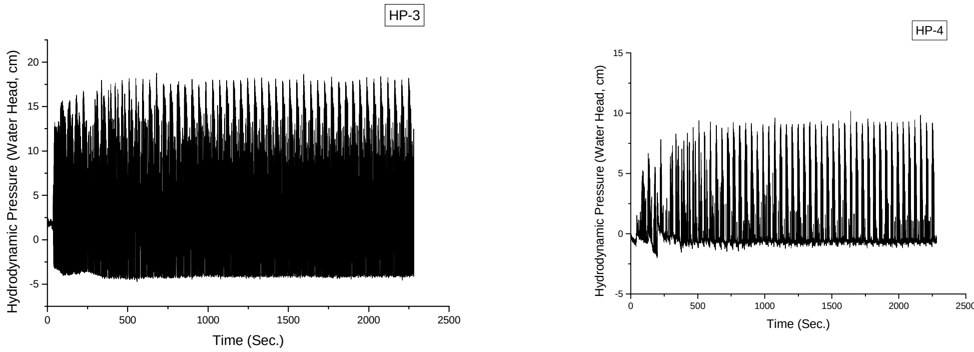


Fig. 7 The profiles of horizontal pressure on GCS-2 model

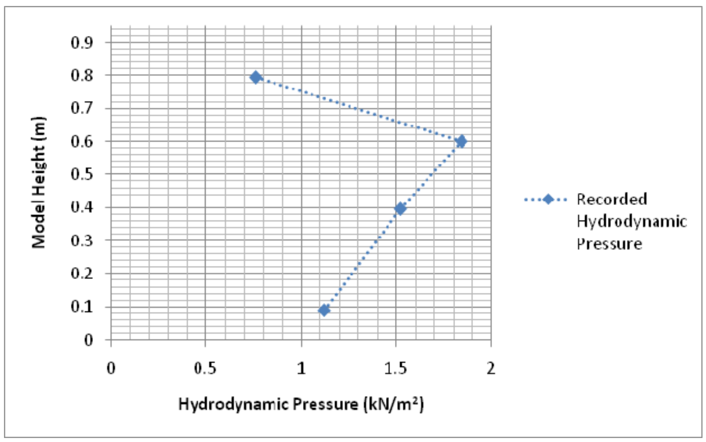


Fig. 8 Hydrodynamic wave pressure profile at the time of maximum resultant force in GCS-2

In Figures 9, 10 the profiles of horizontal pressure on the GCS-3 and GCS-4 model are illustrated. Figures 11, 12 show the hydrodynamic wave pressure profile at the time of maximum resultant force which has been obtained equal to 0.98 kN/m at 612.57 Sec. time and 0.99 kN/m at 5615.22 Sec. time in GCS-3 & 4 respectively.

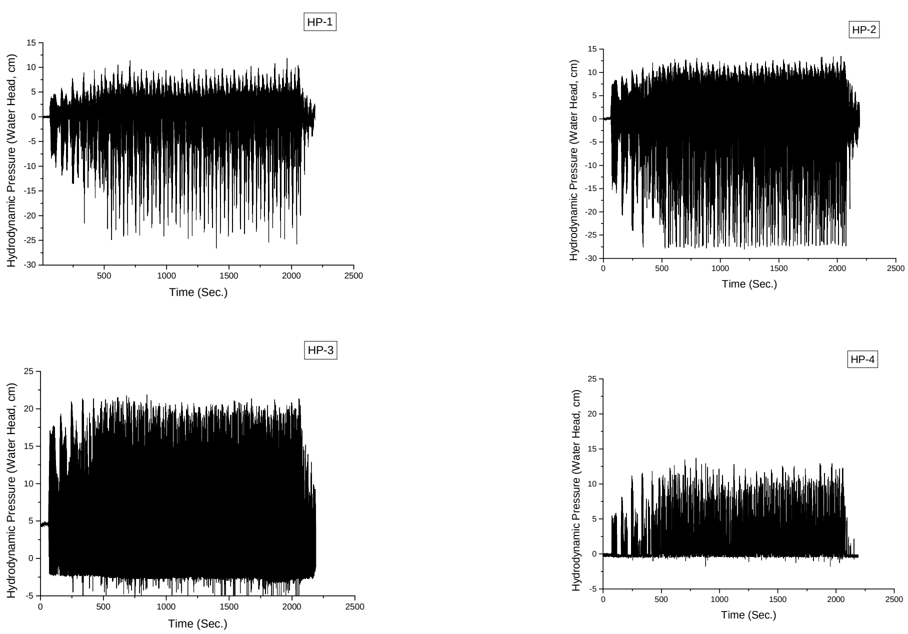


Fig. 9 The profiles of horizontal pressure on GCS-3 model

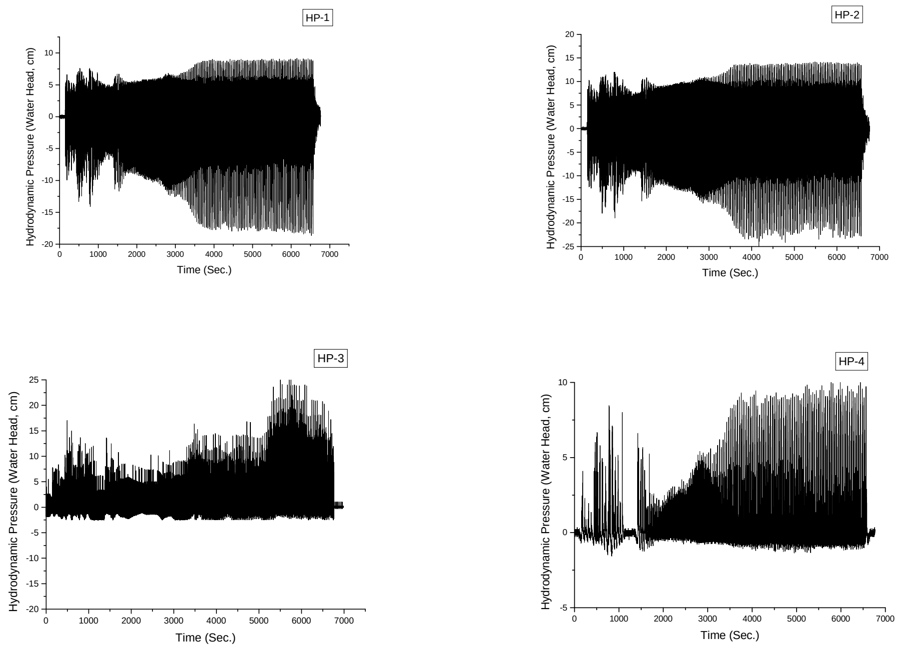


Fig. 10 The profiles of horizontal pressure on GCS-4 model

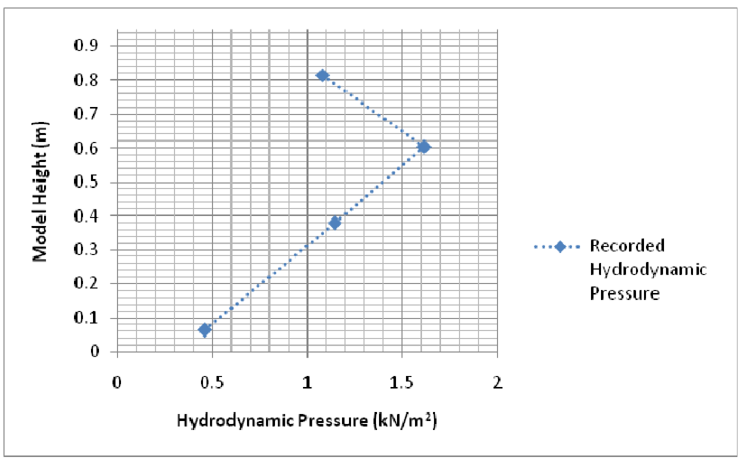


Fig. 11 Hydrodynamic wave pressure profile at the time of maximum resultant force in GCS-3

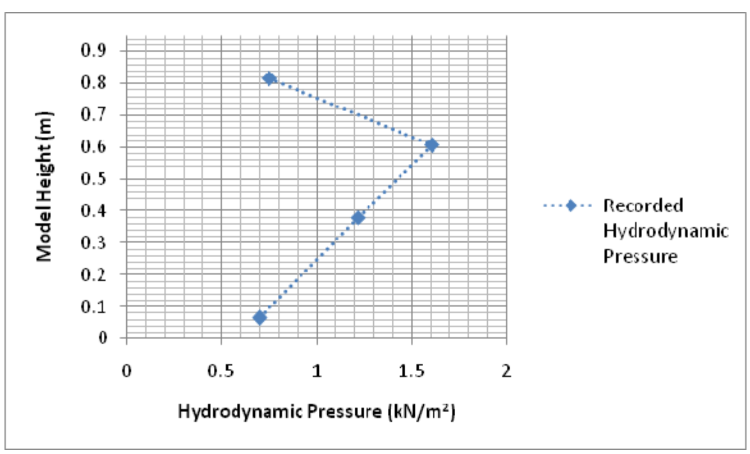


Fig. 12 Hydrodynamic wave pressure profile at the time of maximum resultant force in GCS-4

4. Comparison with Conventional Wave Pressure Formulas

Several wave force theories are promoted for the evaluation of the wave force acting on vertical and sloped breakwaters. For example, under the assumption of uniformly distributed loads with averaged wave pressure acting on vertical wall, Hiroi, in 1919 [10], proposed the first wave pressure formula. Sainflou in 1928 [11],[12], theoretically derived a simple form of standing wave force formula. In 1950, Minikin [13] formula was proposed from the studies of impact force tests. Based on the Ito (1971) [14] continuous loading and maximum wave height concepts, and the experimental/field data, Goda, in 1974[15], obtained four equations for the design load on vertical walls and becomes the most popular equations in the recent coastal structure design. So the recorded data on square section models are compared with the Goda's equation. The related sketch is shown in Figure 13 and equations are as follows;

$$\eta^* = 0.75(1 + cos\beta)\lambda_1 H_{design} \qquad (1)$$

$$P_1 = 0.5(1 + cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_*\cos^2\beta)\rho_w g H_{design} \qquad (2)$$

$$\mathrm{P}_2 = \begin{cases} \left(1 - \dfrac{h_c}{\eta^*}\right) P_1 \text{ for } \eta^* > h_c \\ 0 \ \text{ for } \eta^* \le h_c \end{cases} \qquad (3)$$

$$P_3 = \alpha_3 P_1 \qquad (4)$$

$$\alpha_1 = 0.6 + 0.5\left[\frac{4\pi h_s/L}{\sinh\left(4\pi h_s/L\right)}\right]^2 \qquad (5)$$

$$\alpha_* = \alpha_2 = \min\frac{\mathrm{h_b} - d}{3h_b}\left(\frac{H_{Design}}{d}\right)^2 \ and\ \frac{2d}{H_{Design}} \qquad (6)$$

$$\alpha_3 = 1 + \frac{\mathrm{h_w} - \mathrm{h_c}}{\mathrm{h_s}}\left[1 - \frac{1}{\cosh\left(2\pi \mathrm{h_s}/L\right)}\right] \qquad (7)$$

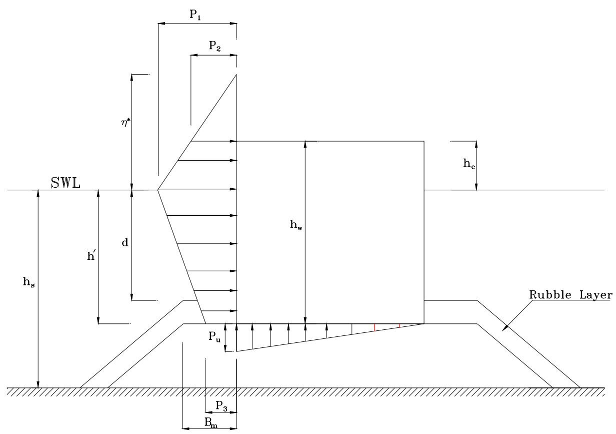


Fig. 13 Goda's equation and the related sketch [12]

In the above equations; L is wave length, H_{Design} is the maximum wave height in front of the breakwater, h_b is the water depth at a distance of $5H_s$ seaward of the breakwater, H_s is the significant wave height, β is angle of incident wave, λ_1, λ_2, λ_3 are modification factors depending on structure type. Other parameters are shown in the sketch. The evaluation of this formula with rigid caisson type breakwaters is described by Chiu et al. (2007) [16] and it could be concluded that Goda's wave force theories overestimate the wave forces acting on caisson in large wave condition which could be defined as (H/T) ratio greater than 13 (cm/sec.) in this reference. This phenomenon attributed to the random property of waves and of the interactions among waves, sandy seabed, rubber mound foundation and breakwater. The conservative result of Goda's theory is also addressed in CEM (2006) [12].

For sloped breakwaters, Tanimoto and Kimura (1985) [17] performed model tests and demonstrated that the Goda formula can be applied by projection of the Goda wave pressure calculated for a vertical wall with the same height as illustrated in Figure 14.

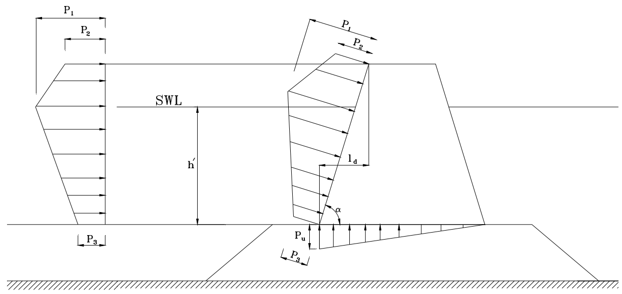


Fig. 14 Proposed method by Tanimoto and Kimura (1985) [17] for using the Goda's formula for sloped structures [12]

Comparison of this formula with experimental results by Vicinanza et al. (2006) [18] shown that the predicted values by formula are about 10% greater than the measured ones.

Using the mentioned formulas, the collected wave pressure data in GCS-1, 2 models are compared with calculated wave pressure distribution based on Goda's equation in Figures 15, 16. By calculating the resultant force of the illustrated wave pressure distributions the difference between recorded wave pressure and calculated pressure by Goda equation are equal to 48.9% and 48.1% for GCS-1 and GCS-2 respectively. In comparison with Chiu et al. (2007) [16] these differences are greater.

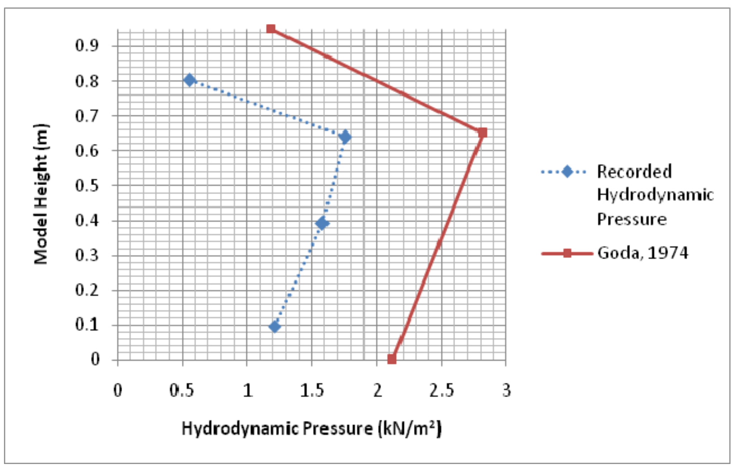


Fig. 15 Comparisons of theoretical wave forces (Goda, 1974 Eq. [15]) with experimental Data (dashed line) in GCS-1

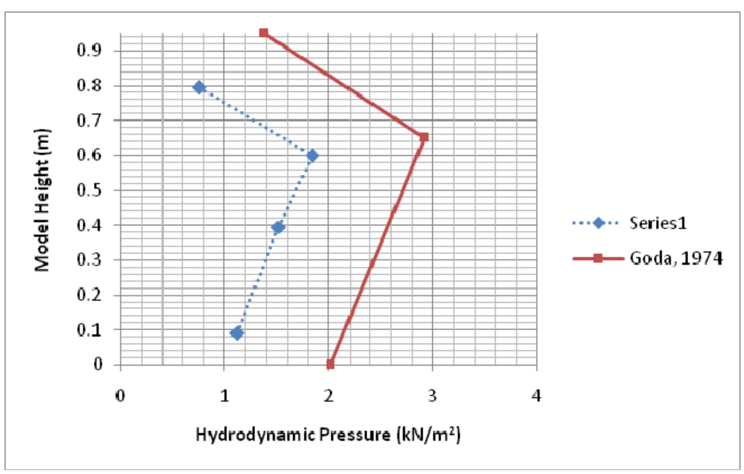


Fig. 16 Comparisons of theoretical wave forces (Goda, 1974 Eq. [15]) with experimental Data (dashed line) in GCS-2

Recorded wave pressure data from GCS-3, 4 are compared with calculated wave pressure distribution based on Tanimoto and Kimura's equation in Figures 17, 18. The difference between recorded wave pressure and formula are equal to 53.2% and 48.7% for GCS-3 and GCS-4 respectively which is larger than those reported by Vicinanza et al. (2006) [18].

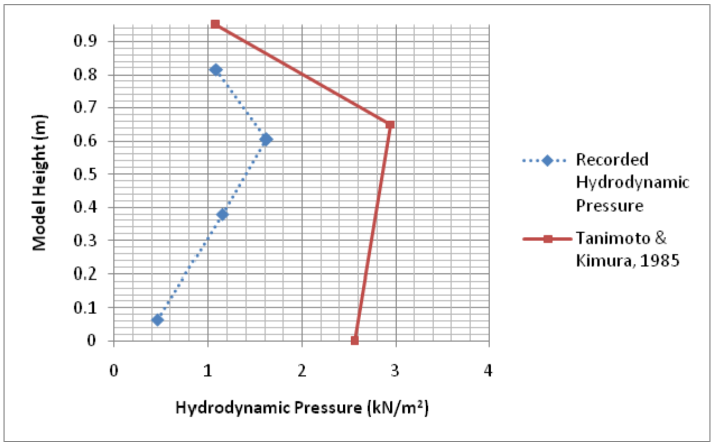


Fig. 17 Comparisons of theoretical wave forces (Tanimoto & Kimura, 1985 [17]) with experimental Data (dashed line) in GCS-3

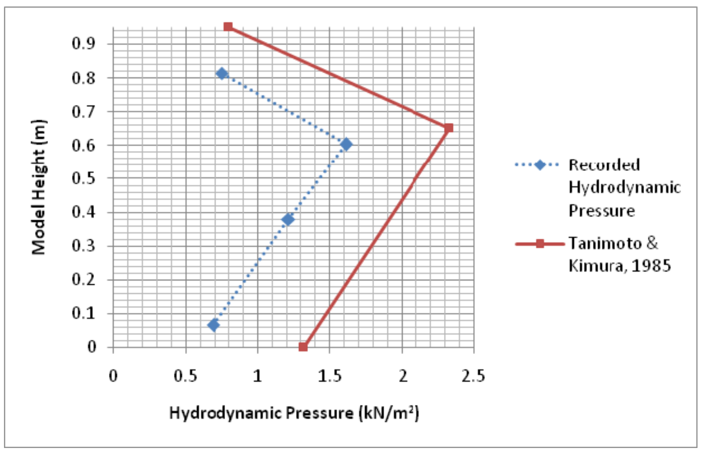


Fig. 18 Comparisons of theoretical wave forces (Tanimoto & Kimura, 1985 [17]) with experimental Data (dashed line) in GCS-4

In cubic models the filling material characteristics have no significant effect on the evaluated difference between recorded and calculated wave pressures. It could be related to differences between wave heights in these two models. Model GCS-2 studied under greater wave height and this leads to smaller difference between the recorded and predicted wave pressure in comparison with GCS-1 which has been filled by sand and as described below, because of larger permeability of sand it is predicted to have greater difference between recorded and predicted wave pressures.

In Trapezoidal section model filled by Silt (GCS-4), the difference between recorded and predicted wave pressures is smaller than model with sandy filling material. This could be because of larger permeability of sand as filling material and consequently more dissipation of wave's energy in to the model. Vice versa GCS1 &2, in model GCS-3 the wave characteristics are sever but the resultant wave pressure is smaller than GCS-4.

It is appeared, that the Goda and Tanimoto and Kimura's equations overestimate the hydrodynamic wave pressure on the GCS models. It should be noted that Goda's equation is introduced based on experimental modeling of impermeable vertical walls on rubble mound base, whereas the GCS models have some degree of permeability because of the composite nature of the GCS structure, especially the permeability of fabric and filling material. So this permeability can lead to seeping water through models and dissipate the wave pressure. Furthermore the GCS models have been studied on rigid flume bottom and in comparison with Goda's formula models, as reported by Chiu et al. (2006), the interactions among waves, sandy seabed in Goda models could be one of the reasons of the illustrated differences in Figures 15-17 between recorded hydrodynamic wave pressures and predicted ones.

So the attained difference between recorded data and predicted pressures by conventional formulas might be caused by the two followings;

- Considering the permeability of GCS models in comparison with rigid type caissons, the larger amount of differences was because of seeping water through the GCS model fabric and filling materials, and therefore reducing the hydrodynamic pressure because of this seepage.
- Interactions among waves, sandy seabed, and rubber mound foundation and breakwater has been reported in [16] for studies on rigid type caissons but difference between recorded and calculated wave pressures in these references was smaller than those obtained for GCS Models and as described above this is related to GCS permeability.

5. Conclusion

GCS models, as new idea for coastal structures has been evaluated under wave action in flume. Horizontal wave forces acting on the GCS models were measured at different elevations. The maximum resultant horizontal force in each test was evaluated and wave pressure diagram compared with theoretical predictions. The results show that Goda's equation overestimates the hydrodynamic water pressure on these structures. Such differences and conservative prediction wave pressures by Goda's formula has been reported by others [12], [16], and [18]. But the amount of this difference is larger in GCS models in comparison with rigid caissons. This can be attributed to the influence of permeability of GCS models and seeping water through the GCS models, and it can be summarized as one advantaged of this structure. The difference for models filled by Sand was greater than those filled by Silt as filling material, because of larger permeability of sand. The conventional hydrodynamic wave pressure formulas could be used in the evaluation of wave pressure in the design of GCS structures, conservatively. But the amount of design factor of safeties might be selected as the lower bound of the proposed values in references.

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References

[1] Ghiassian H, Gray D. H, Hryciw R. D. Stabilization of coastal slopes by anchored geosynthetic systems, ASCE Journal of Geotechnical and Geoenvironmental Engineering 1997, Vol. 123, pp. 736-743.

[2] Ghiassian H, Jalili M, Kasemi D. Model study of sandy slopes under uniform seepage and reinforced with anchored geosynthetics, Journal of Geosynthetics International, 2009, Vol. 16, pp. 452 –467.

[3] Vitton S. J. Load transfer mechanisms in anchored geosynthetic systems, Thesis submitted in partial fulfillment for the degree of Doctor of Philosophy, The University of Michigan, Ann Arbor, MI, 1991.

[4] Ghiassian H, Holtz R. D. Geosynthetic Cellular Systems (GCS) for Coastal Protection, 62^{th} Canadian Geotechnical Conference & 10^{th} Joint CGS-IAH-CNC Groundwater Conference, Geohalifax, September 2009, Canada.

[5] Saathoff F, Kohlhase S. Research at the Franzius-Institut on geotextile filters in hydraulic engineering, Proceedings of the 5th Congress, Asian and Pacific Regional Division, ADP/IAHR, Seoul, 1986, pp. 9–10.

[6] Recio-Molina J, Yasuhara K. Stability of modified geotextile wrap-around revetments (GWR) for coastal protection, Geosynthetics International, 2005, Vol. 12, pp. 260–268.

[7] Lawson C.R. Geotextile containment for hydraulic and environmental engineering, Geosynthetics International, 2008, Vol. 15, No. 6, pp. 384-427.

[8] FHWA-HI-95-038, Geosynthetics design and construction guidelines, National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C, 1998.

[9] Ishihara K. Soil Behavior in Earthquake Geotechnics, Oxford Engineering Science Series 46, Oxford Science Publications, 1996.

[10] Hiroi I. On a method of estimating the force of waves, Memoirs of Engg. Faculty, Imperial Univ. Tokyo, 1919, Vol. X, No.1, pp.19.

[11] Sainflou M. Treatise on Vertical Breakwaters, Annals des Ponts et Chaussee, Paris, France (Translated by W.J. Yardoff, U.S. Army Corps of Engineers), 1928.

[12] CEM, Coastal Engineering Manual, EM 1110-2-1100, U.S. Army Corps of Engineers, 2006.

[13] Minkin R.R. Wind, Waves and Mritime Structures, Charles Griffin and C., Ltd., 1950.

[14] Ito Y. Stability of mixed-type breakwater- a method of probable sliding distance, Coastal Engineering in Japan, 1971, Vol. 14, JSCE, pp. 53-61.

[15] Goda Y. A New Method of Wave Pressure Calculation for the Design of Composite Breakwater. Proccec of 14^{th} International Conference Coastal Engineering, 1974, Vol. 12, No.3. Copenhagen, Denmark.

[16] Chiu Y. F, Lin J. G, Chang S. C, Lin Y. J, Chen C. H. An Experimental Study of Wave Forces on Vertical Breakwater, Journal of Marine Science and Technology, 2007, Vol. 15, N0. 3, pp. 158-170.

[17] Tanimoto K, Kimura K. A Hydraulic Experimental Study on Trapezoidal Caisson Breakwaters, Technical Note No. 528, Port and Harbour Research Institute, Yokosuka, Japan (in Japanese), 1985.

[18] Vicinanza D, Kofoed J. P, Frigaard P. Wave Pressures on Seawave Slot-Cone Generator, Proceedings of the 30th Italian Conference on Hydraulic Engineering: IDRA, Rome, 2006.