



Evolutionary Polynomial Regression-Based Models to Estimate Stability of Gravity Hunched Back Quay Walls

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ABSTRACT: This paper presents a new approach, based on Evolutionary Polynomial Regression (EPR), for predicting the safety factors of a quay wall against sliding, overturning and bearing capacity failure as functions of the soil's shear strength parameters, geometry of the wall and loading conditions. To this end, a database of around 80000 data sets was created based on a conceptual model, employing a MATLAB-aided program. Based on input and output values of this database and employing EPR, three different models for the estimation of safety factors were developed and their results were compared with the values in the database. Investigation into the performance of the developed models indicates that these models are capable of estimating the stability of quay walls with a precision of around 95%. Parametric analyses were performed on the models to identify parameters with a key role in the stability of quay walls. The parametric studies were indicative of the models' ability in capturing the effects of individual parameters on the wall safety factors, therefore proving the models helpful as tools in the preliminary design of gravity quay walls.

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1- Introduction

Gravity quay walls are one of the most common waterfront structures owing to being highly durable, simple to construct and having the capacity to reach deep seabed levels. One of the biggest limitations in the use of gravity quay walls rises from bearing capacity problems associated with their heavy weight. Unsatisfactory seismic performance is another issue which has in the past resulted in large displacements and damage to the backfill infrastructures. Modifying the back-face shape of a wall can be an effective way to enhance performance. Hunched Back Quay walls have been developed with the intention of resolving such issues [1].

A negative batter angle in a landward-leaning wall has the advantage of a smaller wedge in the wall backfill. As a consequence, a smaller lateral thrust is generated. However, the larger weight of the wall can result in bearing capacity issues, larger inertial forces during seismic events and expenses entailed for the extra material. Contrary to this, a seaward-leaning wall with a positive batter angle would lower costs, but lead to a larger failure wedge and a greater lateral thrust. A hunched back wall is essentially a combination of the aforementioned walls, developed to take advantage of their assets while toning down the flaws (Figure 1). The negative batter at deeper elevations leads to a reduction of the lateral thrust while the positive batter angle at shallower elevations

where the lateral pressures are small limits expenses and wall weight [2, 3].

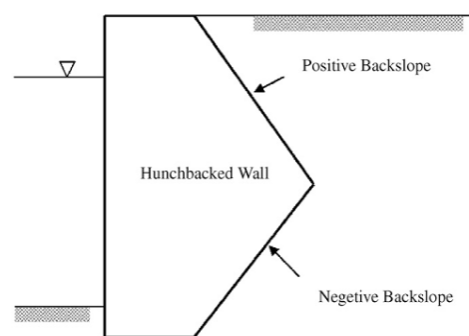


Figure 1. Definitions of negative and positive back-slopes

Sadrekarami [4] presented the results of shaking table tests on hunched back gravity quay walls made of concrete blocks to assess their static and seismic behavior. The results showed that the negative back-slope reduces the lateral earth pressures while the positive back-slope increases them making a very different pressure distribution pattern behind the wall in comparison to the typical triangular pattern which develops behind vertical back-face walls. In addition, the Mononobe-Okabe pseudo-static method was used to estimate the lateral earth pressures acting on the broken-back walls. Comparing

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the results with the model tests indicated that the M-O method was capable of predicting the magnitude and overall distribution of the lateral active pressure on the broken-back walls. Figure 2 shows the pressure distribution pattern on the walls obtained through model tests and the M-O method as presented by Sadrekarimi.

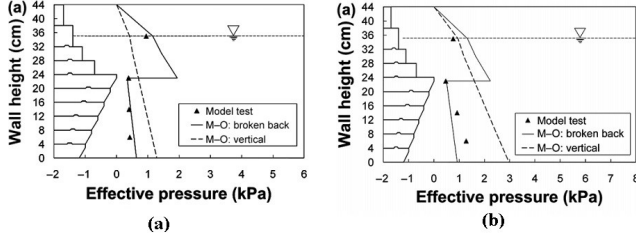


Figure 2. Effective lateral pressure distributions, measured from physical modeling tests and calculated by the Mononobe-Okabe method in (a) static and (b) dynamic conditions (after Sadrekarimi [4])

Sadrekarimi [3] investigated the seismic displacement of broken-back quay walls through reduced-scale shaking table model experiments. Complete quay wall collapse was not witnessed in any of the model experiments and damage was in the form of excessive displacement. Thus, the author proposed the use of performance-based design scenarios for such walls in seismic areas in a way that the design accounts for the likely displacement in addition to calculating the usual factors of safety. Sadrekarimi [5] employed pseudo-static limit equilibrium analyses to calculate the lateral earth pressures on broken-back retaining walls and evaluate their stability in sliding and overturning while a cost reduction analysis was conducted as well to comparing the external stability and the efficiency of broken-back walls with those of vertical-back walls. Results showed that a broken-back wall could be designed at a significantly reduced cost while maintaining sliding and overturning stability of a wall.

1- 1- Loads acting on a hunched back gravity quay wall

The methodology proposed by “Technical Standards and Commentaries for Port and Harbor Facilities in Japan”, OCDI (2002), has been adopted to determine the loads acting on the hunched back gravity quay wall as follows: surcharge, deadweight of the wall, earth pressure in static and dynamic conditions and dynamic water pressure during an earthquake [6].

Okabe [7] and Mononobe and Matsuo [8] developed a pseudo-static method to integrate the seismic earth pressures on retaining structures into the soil thrust on the wall. Pseudo-static accelerations in the form of $a_h = k_h \cdot g$ in the horizontal and $a_v = k_v \cdot g$ in the vertical direction are applied to the active (or passive) wedge. Next, similar to the static Coulomb theory, force equilibrium of the failure wedge is used to obtain the pseudo-static soil thrust. Vertical accelerations are usually ignored. According to the M-O method, the lateral effective active pressure on the wall can be calculated using equation 1.

$$P_{AE} = K_{AE} \gamma_{sub} z \cos(\alpha - \delta - 90) \quad (1)$$

where K_{AE} is the pseudo-static seismic active earth pressure

coefficient obtained from equation 2, γ_{sub} is the submerged unit weight of the backfill soil, and z is the soil depth. α and $\sum(= \alpha - \delta - 90^\circ)$ are the angle of wall’s rear face and P_{AE} with the horizon, respectively, and δ is the interface friction angle between the wall and the backfill soil, taken as about 50% of the internal friction angle of the backfill soil, ϕ [1, 4, 9].

$$K_{AE} = \frac{\sin^2(\alpha - \theta + \varphi)}{\cos \theta \sin^2 \alpha \sin(\alpha - \theta - \delta) \left[1 + \sqrt{\frac{\sin(\varphi + \delta) \sin(\varphi - \beta - \theta)}{\sin(\alpha - \theta - \delta) \sin(\alpha + \beta)}} \right]^2} \quad (2)$$

where β is the backfill slope angle (assumed nil), and θ is the apparent seismic angle defined as:

$$\theta = \tan^{-1} \left[\frac{G_s}{G_s - 1} k_h \right] \quad (3)$$

where G_s is the specific gravity of backfill soil [7, 8]. Equation 4 is used to calculate the hydrodynamic force of the free water in the backfill.

$$P_{wd} = \frac{7}{12} k_h \gamma_w H_p^2 \quad (4)$$

where y_w is the water depth and H_p the total water depth. This forced is applied at a height of $0.4H_p$ above the base. The total hydrodynamic force is twice the value of equation 4 to include both the positive pore pressures in front and the negative pressures behind the wall [10].

1- 2- Stability analyses

Based on the Technical Standards and Commentaries of Port and Harbor Facilities in Japan, OCDI (2002), three failure modes are investigated: sliding, bearing capacity of the foundation and overturning. Equations 5 and 6 are used to obtain the safety factor against sliding and overturning respectively.

$$F_s = \frac{fW}{P} \quad (5)$$

$$F_s = \frac{Wt}{Ph} \quad (6)$$

where W and P are the resultant vertical and horizontal forces on the wall respectively, f is the coefficient of friction between the bottom of the wall and the foundation, t is the distance between the line of application of W and the front toe, h is the height of the application line of P on the wall, above the bottom and F_s is the safety factor. To evaluate the bearing capacity of the wall foundation the circular arc analysis based on the simplified Bishop method has been chosen [6].

2- Methodology and Modeling

2- 1- Evolutionary polynomial regression

Evolutionary polynomial regression (EPR) is a data-driven regression method, developed by Giustolisi and Savic (2006) based on evolutionary computing. In EPR the evolutionary procedure searches for the exponents of a polynomial function with a fixed maximum number of terms [11]. During one execution, it returns a number of expressions with increasing

numbers of terms up to a limit set by the user to allow the optimum number of terms to be selected [12]. In general, EPR is a two-stage method to construct models using polynomial structures. In the first stage, EPR searches for exponents of polynomial expressions by employing a genetic algorithm. In the second stage, numerical regression is used to compute the constant values of the previously selected terms by solving a least squares problem. To apply the EPR procedure, the evolutionary process starts from a constant mean of output values. By increasing the number of evolutions, it gradually picks up different participating parameters in order to form equations describing the relationship between the parameters of the system. The EPR procedure stops when the termination criterion (the maximum number of terms in the expression, the maximum number of generations, or a particular allowable error) is satisfied [13].

2- 2- Database and conceptual model

In the preliminary stage of a quay wall design, the aim is to obtain the dimensions required to withstand the loading conditions while minimizing expenses. In this paper, the aim is to create EPR models for quick and simple preliminary designs of gravity quay walls which can then, be followed by a more elaborate analysis. A large database of hypothetical cases of hunched back gravity quay walls based on the conceptual model of Figure 3 was created. A series of parameters related to geometry, the shear parameters of the foundation and backfill soil and loading conditions are used to define this model.

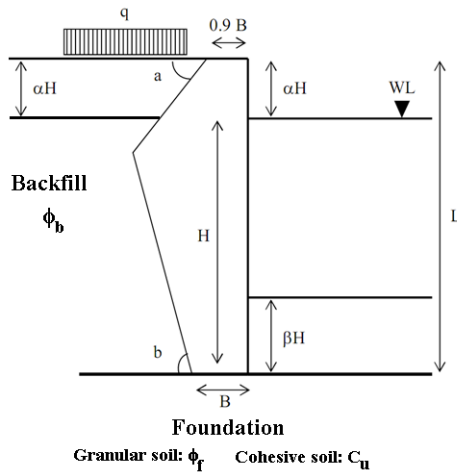


Figure 3. Employed conceptual model of a gravity hunched back quay wall

The large database used in the EPR-based models was created by the introduction of variables shown in Table 1. By changing these variables in the applicable range, more than 80000 hypothetical gravity quay walls were created. About 90% of these hypothetical cases were used in the development of the models while the remaining 10% were set aside for the validation of the models after their development. The methodology covered in section 1.2 was used to obtain the safety factors of each hypothetical wall against sliding, overturning and bearing capacity failure. The stability analyses were programmed in MATLAB R2012b (8.0.0.783) to facilitate the calculations. Variables shown in Table 1 were then entered as inputs and the obtained safety factors against

sliding, overturning and bearing capacity failure as outputs to develop the EPR models.

The geometry of the quay wall is defined using the following parameters: two parameters to determine the top and bottom wall rear-face batter angles (a and b respectively), the wall height (L) and the width of the wall at the top and bottom. The width at the bottom of the wall is shown with B while the width at the top has been assumed to be 90% of B. The parameter beta determines the embedment depth of the wall, while the water level, assumed to be equal in front and behind the wall is shown with H. The wall's freeboard is defined with the parameter alpha.

In the stability analysis of a gravity quay walls during a seismic event, four different scenarios can be defined in terms of pore pressure generation in the backfill. The first scenario would be the absence of water or a dry backfills (Case 1). In case of a submerged backfill three scenarios are considered: no pore water generation (Case 2); complete liquefaction of the backfill (Case 4) and the intermediate case between case 2 and 4 (Case 3). Here it is assumed that the generation of extra pore water pressure in the backfill during seismicity is unlikely due to the fact that in practice highly permeable compacted granular materials are used as the wall backfill to enhance the seismic performance of the wall and avoid the generation of excess pore water pressures. In the conceptual model, the backfill material are assumed as granular with no cohesion but two conditions are assumed for the soil beneath the foundation of the quay wall: drained condition where the soil is considered as granular and the internal friction angle, phi_f, is considered as the sole shear strength parameter and the undrained condition for cohesive soils in short-term conditions where undrained shear strength parameter, C_u, is used in the analysis. The internal friction angle of the backfill soil is shown with phi_b. Surcharge load behind the wall is defined by the parameter q. Seismic loading is defined by horizontal pseudo-static coefficient, k_h.

3- Performance analysis

3- 1- Fitting parameters analysis

Different statistical approaches have been used to evaluate the performance of the prediction models. These parameters are the coefficient of determination (COD), root mean square error (RMSE) and the coefficient of residual mass (CRM) defined by Equations 7, 8 and 9 respectively.

$$COD = 1 - \frac{\sum_{i=1}^n (M_i - P_i)^2}{\sum_{i=1}^n (M_i - \bar{M})^2} \tag{7}$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (P_i - M_i)^2}{n}} \times 100 \tag{8}$$

$$CRM = 1 - \frac{\sum_{i=1}^n (P_i)}{\sum_{i=1}^n (M_i)} \tag{9}$$

Table 1. Descriptive statistics of the variables used in the EPR models

| Parameter | Cu (kPa) | φ_f (degree) | φ_b (degree) | q (kN/m) | k_h | a (degree) | b (degree) | α | β | H (m) | L/B |
|--------------------|-------------|-------------------------|-------------------------|-------------|-------|---------------|---------------|----------|---------|----------|------|
| Minimum | 200 | 25 | 35 | 0 | 0 | 60 | 45 | 0.1 | 0 | 5 | 2 |
| Maximum | 800 | 35 | 45 | 55 | 0.2 | 89.9 | 75 | 0.2 | 0.2 | 10 | 4 |
| Mean | 466.67 | 30 | 40 | 23.33 | 0.1 | 74.97 | 60 | 0.15 | 0.1 | 7.5 | 3 |
| Standard Deviation | 249.45 | 4.08 | 4.08 | 23.21 | 0.082 | 12.21 | 12.25 | 0.05 | 0.08 | 2.04 | 0.82 |

where M_i is the actual value and P_i the predicted value, \bar{M} the mean of the actual data and n is the number of data. The RMSE is the variance of the residual error, and should be minimized when the outputs fit a set of data. In the case of a perfect fitting the RMSE is zero. The lower the RMSE, the higher the accuracy of the model predictions. The coefficient of residual mass, CRM, represents the difference between the actual data and the predicted values. The optimum value of CRM is zero. Positive values of CRM indicate underestimation and vice-versa.

3- 2- Parametric analysis

For further verification of the models and to analyze the individual contribution of each input parameter on the outputs, parametric analyses have been performed. The method of parametric analysis is based on changing one predictor variable at a time while the other predictor variables are kept constant at the average values of their entire data sets. Parametric analysis investigates the response of the predicted values from the EPR models to a set of input data generated over the training ranges of the minimum and maximum data. These variables are presented to the prediction model and the output is calculated. This procedure is repeated using another variable until the model response is tested for all input variables.

4- Results and Discussions

4- 1- EPR models for safety factors of quay wall in drained conditions

The following ten input variables were used to model the safety factors of quay walls against three modes of failure in drained condition: φ_f , φ_b , q, k_h , L, L/B, B, a, b and β . The following models are obtained for safety factors against sliding, overturning and bearing capacity failure respectively:

$$FS_{1(Sliding)} = -5.075 k_h \left(\frac{L}{B}\right)^{0.5} - 0.016 q - 0.0054 a b^{0.5} + 0.012 \phi_b B^{0.5} \beta^2 + 0.02 \phi_f^{0.5} \phi_b + 1.804 \quad (10)$$

$$FS_{2(Overturning)} = -2.57 k_h - 0.007 q - 0.0003 b^2 \left(\frac{L}{B}\right)^{0.5} - 0.0006 a^2 + 1.057 \times 10^{-6} \phi_b^{0.5} a^2 b + 4.79 \quad (11)$$

$$FS_{3(Bearing)} = 6.098 \beta - 0.0012 b^2 k_h - 3.93 \times 10^{-6} b^2 q - 0.237 a^{0.5} L^{0.5} + 0.019 \phi_f \phi_b^{0.5} B^{0.5} + 1.02 \quad (12)$$

Next the EPR models are validated using both the data used in the development of the models and the data set aside for validation. Figure 4 shows the predicted values of safety factors using the models versus the actual database values obtained from stability analysis for. Comparison of the results indicates that the developed equations are capable of predicting safety factors within a 95% confidence interval. Table 2 presents the performance of the EPR prediction models.

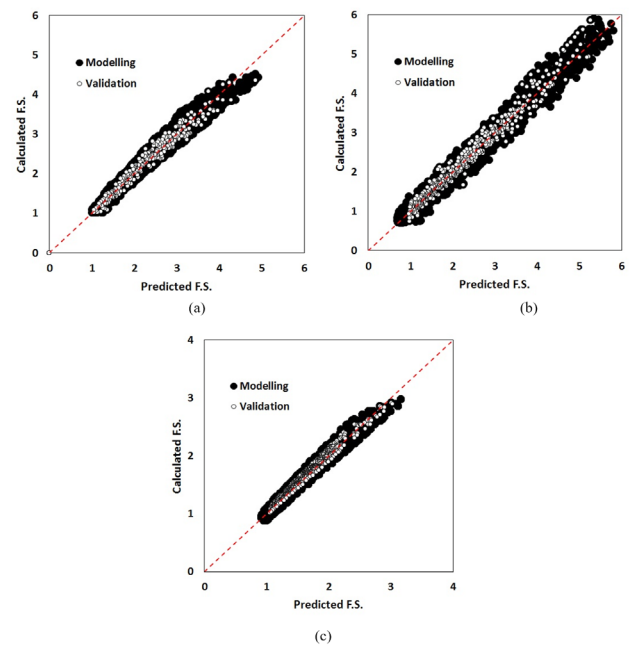


Figure 4. Predicted safety factors against values obtained from deterministic analyses for: a) sliding b) bearing capacity failure c) overturning; (drained conditions)

The values of COD, RMSE and CRM obtained for both the model and the validation data illustrate the high accuracy of the models. The model for the prediction of safety factor against overturning seems to have the least scatter among others, however its accuracy is at the same level of other models with a COD of 95%.

Parametric studies have been performed for further verifications of the developed models. Figure 5 presents the results of the parametric study on the EPR models in drained condition.

Table 2. Performance of the EPR model for safety factors (drained conditions)

| Output | COD | | RMSE | | CRM | |
|------------------------------------|-------|------------|-------|------------|-------|------------|
| | Model | Validation | Model | Validation | Model | Validation |
| FS ₁ (sliding) | 96% | 96% | 0.15 | 0.15 | 0 | 0 |
| FS ₂ (overturning) | 95% | 97% | 0.09 | 0.06 | 0 | -0.011 |
| FS ₃ (bearing capacity) | 97% | 97% | 0.18 | 0.17 | 0 | 0 |

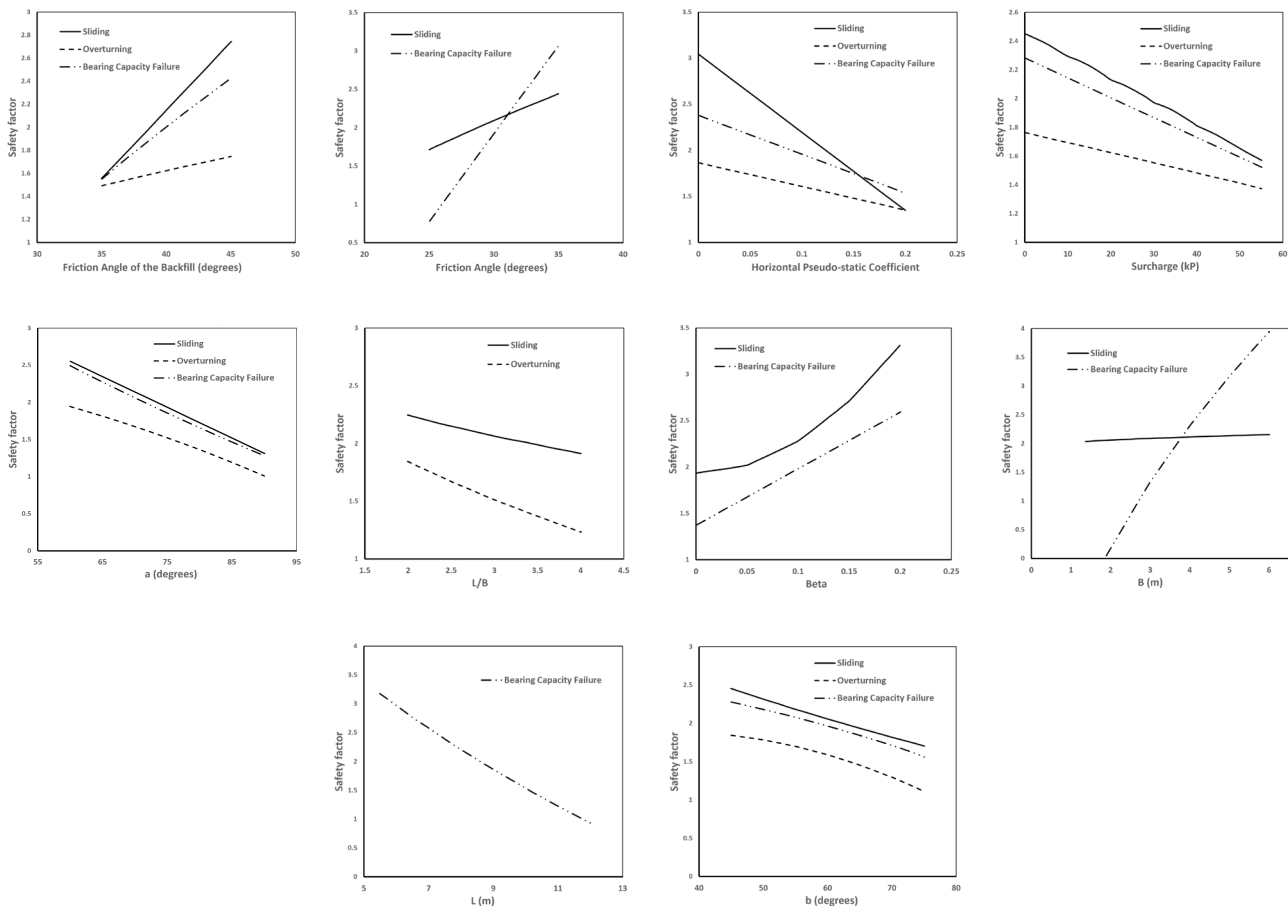


Figure 5. Parametric study results on the safety factor prediction models

Increasing internal friction angles of backfill and foundation, the embedment depth and the wall width increases the safety factor against sliding, although the safety factor against sliding seems not that sensitive to the wall width. Increasing surcharge, horizontal pseudo-static coefficient, (L/B) and back slope angles leads to the reduction of safety factor against sliding, however it seems the (L/B) has a less pronounced effect in the reduction of the safety factor of the quay wall against sliding.

According to the sensitivity analysis illustrated in Figure 5, increasing the internal friction angle leads to an increase in safety factor against overturning. Increasing the rest

of variables such as surcharge, horizontal pseudo-static coefficient, back slope angle, “a” and “b” and (L/B), the safety factor against overturning decreases. It seems this factor of safety is more sensitive to the back-slope angle since changing this parameter from its minimum value to its maximum causes an almost 50% decrease in the factor of safety. From Figure 5 it can be seen that increasing the friction angles, the width and the embedment depth of the quay wall drastically increases the safety factor of the quay wall against bearing capacity failure while other variables decrease it.

4- 2- EPR models for safety factors of quay wall in undrained conditions

The following 10 input parameters were used for modeling the safety factors of quay walls against three modes of failure in undrained conditions: C_u , ϕ_p , q , k_h , L , L/B , B , a , b and β . The following models are obtained for predicting safety factors against sliding, overturning and bearing capacity failure respectively:

$$FS_{1(\text{Sliding})} = 59.37 \beta^2 - 2.058 \times 10^{-5} B a b \left(\frac{L}{B}\right)^2 - 0.46 C_u^{0.5} k_h \left(\frac{L}{B}\right)^{0.5} - 0.0018 C_u^{0.5} q + 0.0011 C_u^{0.5} \phi_b - 0.91 \quad (13)$$

$$FS_{2(\text{Overturning})} = -1.6 k_h \left(\frac{L}{B}\right)^{0.5} - 0.0071 q - 0.0002 b^2 \left(\frac{L}{B}\right)^{0.5} - 0.0005 a^2 + 9.86 \times 10^{-7} \phi_b^{0.5} a^2 b + 4.79 \quad (14)$$

$$FS_{3(\text{Bearing capacity})} = 0.007 C_u - 2.84 \times 10^{-8} C_u b^2 q - 2.65 \times 10^{-5} C_u a L^{0.5} \left(\frac{L}{B}\right) + 0.946 C_u a^2 b^2 k_h \left(\frac{L}{B}\right)^{0.5} + 0.003 C_u \phi_b^{0.5} + 0.0015 \quad (15)$$

Figure 6 shows the predicted values of safety factors in undrained conditions versus the actual database values used in the development of the model and the values set aside for validation. Comparison of the results illustrates the high accuracy of the developed models for both the model and the validation data. Table 3 presents the performance of the EPR prediction models in undrained conditions.

It is obvious from the values of COD, RMSE and CRM that the proposed equations are highly accurate. In the undrained condition, the model for predicting the safety factor against overturning has the least scatter.

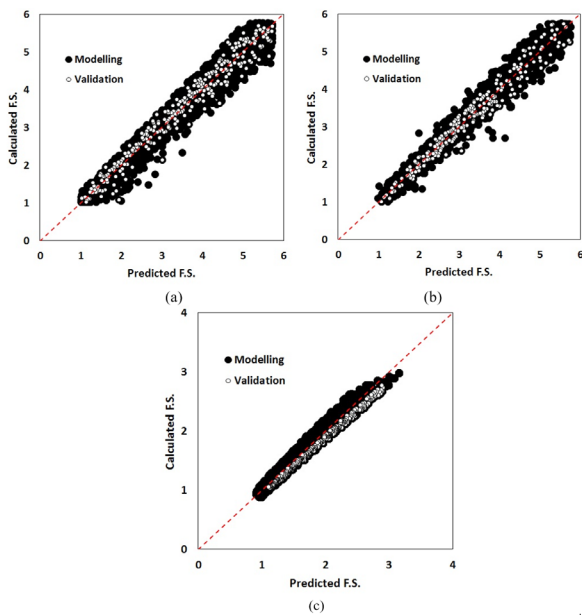


Figure 6. Predicted safety factors against values obtained from deterministic analyses for: a) sliding b) bearing capacity failure c) overturning; (undrained conditions)

Figure 7 present the results of the parametric study on the EPR models in the undrained condition. As can be seen in Figure 7, increasing the internal friction angle of the backfill, the undrained shear strength of the soil beneath the foundation and the width and the embedment depth of the wall leads to an increase in the safety factor against sliding. Increasing variables such as the surcharge, the horizontal pseudo-static coefficient, the back-slope angle of quay walls and (L/B) decreases the safety factor against sliding.

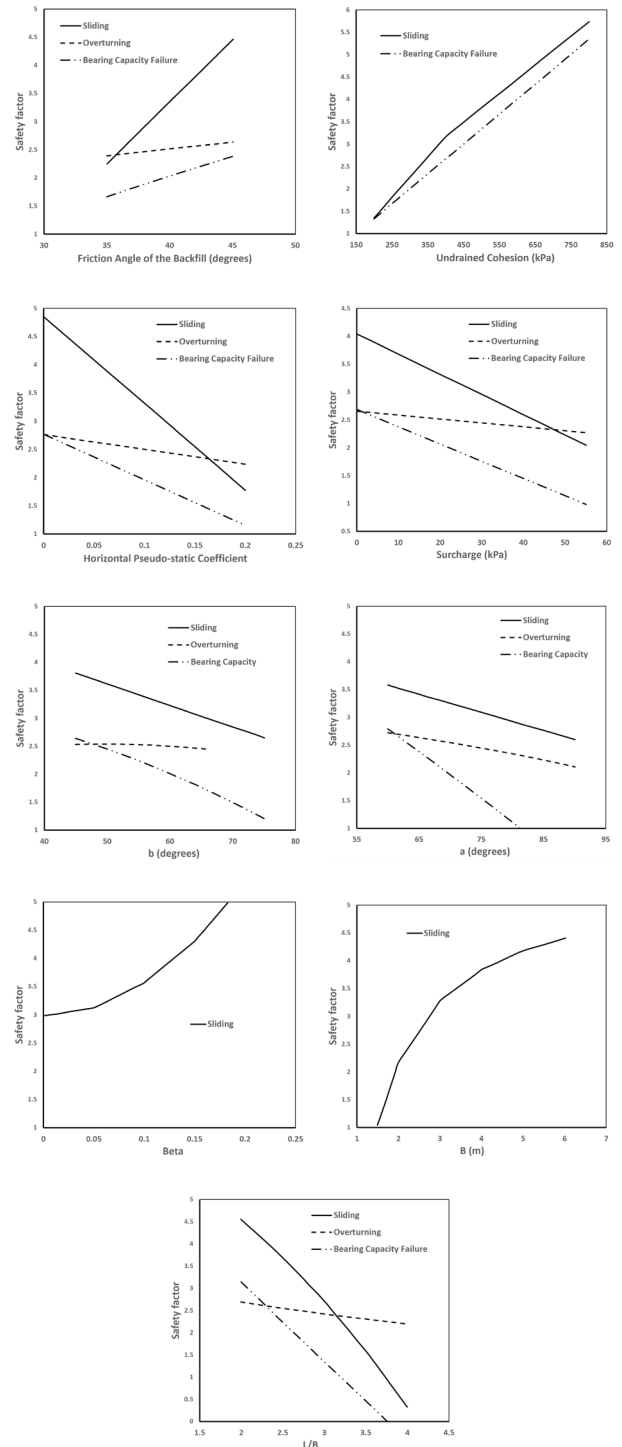


Figure 7. Parametric study results on the safety factor prediction models

Table 3. Performance of the EPR model for safety factors (undrained conditions)

| Output | COD | | RMSE | | CRM | |
|------------------------------------|-------|------------|-------|------------|-------|------------|
| | Model | Validation | Model | Validation | Model | Validation |
| FS ₁ (sliding) | 96% | 96% | 0.24 | 0.25 | 0 | 0 |
| FS ₂ (overturning) | 95% | 93% | 0.09 | 0.12 | 0 | 0.06 |
| FS ₃ (bearing capacity) | 95% | 96% | 0.24 | 0.22 | 0 | 0 |

The safety factor against overturning increases with increasing internal friction angle of the backfill however increasing variables such as surcharge, horizontal pseudo-static coefficient, quay wall back slope angles and (H/B) leads to a decrease in this safety factor.

As expected increasing the undrained shear strength of the soil underneath the foundation as well as the internal friction angle of the backfill increases the safety factor against bearing capacity failure. Increasing surcharge, horizontal pseudo-static coefficient, back slope angles of the quay wall and (L/B) causes a decrease in foundation bearing capacity safety factor.

5- Conclusions

This paper presents models developed using evolutionary polynomial regression, EPR, for predicting the safety factors of quay walls against sliding, overturning and bearing capacity failure. In the EPR approach, there is no need to preprocess, normalize or scale the data. An intriguing feature of EPR is its ability to present more than one model for a complex phenomenon. The best models are chosen based on their performance on a set of data. Hence, the geotechnical properties of the soil and the geometric parameters of the wall as well as loading conditions have been employed to estimate the safety factors of a quay wall in the drained and undrained conditions. By introducing these variables in their common range, around 80000 hypothetical cases were created and their stability estimated using a MATLAB aided program based on OCIDI (2000) methodology of quay wall stability analysis. Results of these analyses were collected as a data bank with the variables effective on the soil-quay wall stability as inputs and three values of calculated safety factors against sliding, overturning and bearing capacity failure as outputs. Around 90% of the values in the data bank were used to develop the models while the rest were used to validate the models afterwards. The results of the developed EPR models have been compared with the values in the data bank. Comparison of the results demonstrates that the developed models provide highly accurate predictions. The performance of these prediction models was examined by three statistical approaches. The results show the precision of the model predictions are around 95%. The results of sensitivity analyses also show that the geometry of the wall has a paramount effect on the stability of quay walls. Embedment depth could increase the stability of quay walls greatly mainly when the walls are subjected to seismic loading. Results from parametric studies indicated that the developed models are able to capture the effects of individual parameters on the quay wall safety factors and therefore the models can be employed in a preliminary design of the quay

wall geometry and then assess the stability of this preliminary design with more sophisticated approaches.

Nomenclature

| | |
|------------------------------|--|
| a | Wall Top Batter Angle |
| a _v | Vertical Pseudo-Static Acceleration, m/s ² |
| a _h | Horizontal Pseudo-Static Acceleration, m/s ² |
| b | Wall Bottom Batter Angle |
| B | Wall Width at the Bottom, m |
| C _u | Undrained Shear Strength of the Soil Beneath the Foundation, kN/m ² |
| COD | Coefficient of Determination |
| CRM | Coefficient of Residual Mass |
| f | Coefficient of Friction Between the Bottom of the Wall and the Foundation |
| F _s | Factor of safety |
| g | Gravitational Acceleration, m/s ² |
| G _s | Specific Gravity of Backfill Soil |
| h | Height of the Application Line of the Resultant Horizontal Forces Acting on the Wall Above the Bottom of the Wall, m |
| H | Water Level, m |
| H _p | Total Depth of the Water, m |
| K _{AE} ^p | Pseudo-Static Seismic Active Earth Pressure Coefficient |
| k _h | Horizontal Pseudo-Static Seismic Coefficient |
| k _v | Horizontal Pseudo-Static Seismic Coefficient |
| L | Wall Height, m |
| P | Resultant Horizontal Force Acting on the Wall, kN/m |
| p _{AE} | Lateral Effective Active Pressure on the Wall, kN/m ² |
| P _{AE} | Lateral Active Force on the Wall, kN/m |
| P _{wd} | Hydrodynamic Force of the Free Water in the Backfill on the Wall, kN/m |
| q | Surcharge Load Behind the Wall |
| RMSE | Root Mean Square Error |
| t | Distance Between the Line of Application of the Resultant Vertical Forces on the Wall and the Front Toe, m |
| W | Resultant Vertical Force Acting on the Wall, kN/m |
| y _w | Depth of Water, m |
| z | Soil Depth, m |

Greek symbols

| | |
|----------------|---|
| α | Ratio of the Wall Freeboard to the Wall Height |
| β | Ratio of the Embedment Depth of the Wall to the Wall Height |
| γ_{sub} | Submerged Unit Weight of the Backfill Soil, kN/m ³ |
| δ | Interface Friction Angle Between the Wall and the Backfill |
| θ | Apparent Seismic Angle |
| Σ | Angle of Lateral Active Force on the Wall with the |

Horizon

φ_b Internal Friction Angle of the Backfill Soil
 φ_f Friction Angle of the Soil Beneath the Foundation

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