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Modelling and Dimensioning of a Natural Riprap Protection Dike Against Coastal Erosion on the Djiffère and Joal Spit

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ABSTRACT

This work aims to study the design and modelling of a longitudinal riprap protection at the bottom of the beach in the areas most threatened by coastal erosion on the Djiffère and Joal spits. The choice of design and modelling techniques is based on the geotechnical and geological characteristics of the subgrade. The sizing allowed us to determine all the parameters necessary to quantify the riprap structure to be installed and the results show that it makes it possible to attenuate the energy of the waves but also to reduce their incident heights by 72%. The dike will help reduce the crossing and will act as a protection for the infrastructure located towards the mainland. The conditions of stability are assured, as well as an analysis of the impacts of the project on the sustainable development of the Joal and Djiffère coastline. Modelling with the Plaxis 3D software allowed us to model the dike with ease and to calculate the safety factor (determination of the balance, stability and displacements of the protective dike). It also allowed us to evaluate the sensitivity of the safety factor with respect to these parameters based on the finite element method using the Mohr Coulomb model. It appears that for a total stress of 1067 KPa, we observed a horizontal displacement of 10.19 cm for the rapid drawdown phase, which falls within the range of tolerable deformations for our dike, i.e. less than 20 cm. In this study, the rapid drawdown of the sea level in 5 days had no influence on the safety factor from ΣMsf = 1.861 to the value of the last phase ΣMsf = 1.972 satisfactory for the dike in terms of stability, i.e. less than 1.4. Thus, the rise in sea level has the effect of producing additional deformations.

Keywords: Protective dike, dimensioning, drawdown, geotechnical modelling, 3D Plaxis.

1. INTRODUCTION

The dynamics of coasts and coastal massifs is one of the main environmental problems facing coastal areas in the world and in West Africa [1]. Senegal, a coastal country located in the extreme part of West Africa, is also subject to this evolution [2], the factors of which can be intrinsic or external and strongly impacted by the activity of the local populations. These populations are closer to the marine area suitable for income-generating activities. However, with climate change, there has been an accentuation of the phenomenon of coastal erosion [3], the extent of which depends on both geographical (swell, wave) and geological factors. Thus, coastal erosion largely disturbs the environmental balance and leads to losses of surface area that can be deplorable. It is important today to fight against this phenomenon in order to preserve our coasts and the infrastructures built there, but also to protect tourist activity since most of the attractive sites and dedicated accommodation (hotels) are located there. This work will therefore consist of dimensioning a protective dike and visualizing its stability with a 3D plaxis model in the Joal and Djiffère sectors, two sites known for their tourist attraction, fishing and other coastal activities. First of all, the choice of the design method to determine the most appropriate physical dimensions for the structure or the element implemented from its technical aspect. In short, the sizing of the composition elements of the protective structure allows us to know the geometry of our structure. This protection system requires the use of rock as the materials of implementation [4]. As in all fields, the finite element method is often used in geotechnics to verify different behaviors such as stability and interaction with the environment, and to control the values of permissible displacements, and to help in the design of structures [5]. Indeed, the purpose of the modeling is to determine the safety factor (determination of the balance, stability and displacements of the protective dike) by the PLAXIS 3D numerical program. The numerical analysis also led to the design of a model (geotechnical properties and water level fluctuations). It is also a question of evaluating the sensitivity of the safety factor with respect to these parameters based on the finite element method using the model of Mohr Coulomb [6]. The objective of this article is first to determine the most appropriate physical dimensions for the riprap protection dike and these derive the geotechnical and geological characteristics of the study areas, then to evaluate the stability of the dike on the foundation soils by a numerical modeling using Plaxis 3D and finally to propose a coastal development policy.

2. General Context

The Senegalese coastline represents an area of strategic interest from a demographic, economic and environmental point of view. Natural environments, in a relatively preserved state of conservation, produce vital resources for the Senegalese population [2]. The Senegalese national economy is highly dependent on these coastal and marine resources, which constitute the main foreign exchange earnings, whether from fishing or tourism.

3. MATERIALS AND METHODS

3.1. Dimensioning of the embankment dike

The aim is to size a longitudinal protection structure in riprap at the bottom of the beach over a 1500 m linear line in the areas threatened by erosion on the Joal and Djiffère spits. The design of the dike requires several steps. First, a characterization of the site and collection of data in situ, and through certain previous studies [7]. These data are analyzed and processed to determine the characteristics of the swells in the breaking zone, at the coast for the dike (Table 1).

Table 1: Project wave parameters for the dike

Wave parameters	
Preferential Swell Direction (National Agency of Civil Aviation and Meteorology of Senegal)	N190
Significant height offshore Hs (m)	3,20
Average peak period of swells T (s)	14,10
Average period Tm (s)	12,30
Return period or Return frequency (years)	1/100
Water depth at the foot of the structure hpied (m)	7,56
Height exceeded by 2% of the waves Ru2% (m)	3,83
Crossing q (l/s/m)	24,00

Then, a choice followed by a classification of materials was made. For the construction of the riprap structures, we have the basalt of Diack. The latter is of volcanic origin and is very often used in public works. It is located 37km southeast of Thiès in the rural community of Ngoudiane. The geotechnical characteristics of Diack basalt are shown in Table 2.

Table 2: Geotechnical characteristics of Diack basalt (Senegal Materials Data Sheet)

Characteristics	
Bulk density	3
Test weight (KN/m3)	29.69
Crush resistance (kg/cm²)	1200
Breaking Load (kg)	120000

And in order, the choice of formulas and the calculation of the most appropriate physical dimensions for the structure or the element implemented in its technical aspect, which derive from the characteristics of the swell and the materials chosen. For the sizing of the carapace, which is the outer layer, made up of the largest and/or most durable materials, for the protection of the slopes against the swell [4] several authors have proposed methods for sizing this first protective layer. In our work Van Der Meer's formula is used.

$$\frac{Hs}{\Delta . Dn50} = Cpl * P^{0,18} * \left(\frac{Sd}{\sqrt{N}}\right)^{0,2} * (\xi_m)^{-0,5}$$

si $\xi_m < \xi_{cr}$ (Plug-in surge) **Equation 1**

$$\frac{Hs}{\Delta . Dn50} = Cs * P^{0,13} * \left(\frac{Sd}{\sqrt{N}}\right)^{0,2} * \sqrt{\cot\alpha}\, \xi_m^{-0,5}$$

si $\xi_m \geq \xi_{cr}$ (Swelling surge) **Equation 2**

- N = number of incident waves (-), which depends on the duration of the sea state
- Hs = significant wave height, H1/3 of the incident wave at the foot of the structure (m)
- ξm = breaking parameter calculated from the average period of the swell Tm(s),

$\xi m = \tan\alpha / \sqrt{(2\pi Hs)\,(gTm2)}$

α the slope angle

Dn50 nominal diameter exceeded by 50% of the blocks = (M50/ρs)1/3

Δ relative density degauged, ρr/ρw-1

P nominal permeability parameter of the structure (-); the value of this parameter must be between $0.1 \leq P \leq 0.6$

Cpl=6.2 with a standard deviation of σ = 0.4

Cs = 1 with a standard deviation of σ = 0.08

Sd = Damage Level

0.5 = no damage

2 to 3 acceptable damage

The transition between a plunging break and a swelling break is calculated from the slope of the embankment of the structure [4] (not the inclination of the seabed in front of the structure) and can be determined using the following equation using a critical value of the breaking parameter ξcr:

$$\xi_{cr} = \left(\frac{Cpl}{Cs} * p^{0,31} * \sqrt{\tan\alpha}\right)^{1/(P+0,5)}$$ **Equation 3**

To calculate the average dimensions of the shell blocks, we use the Van Gent stability formula to check the values found from Van Der Meer. The number of waves N is given by the following formula:

$$N = \frac{\text{duration of a storm (h)} \times 3600 (s/h)}{Tm}$$

Equation 4

The acceptable level of damage depends on the slope of the structure and takes into account the settlement and sliding of the rockfill of the structure during its lifetime. We have a slope of 2/5 so Cotgα = 2.5, we choose the value Sd = 2 [4]. The nature of the breaking results from the comparison of the average (ξm) and critical (ξcr) breaking parameters.

$$\xi m = \frac{\tan\alpha}{\sqrt{\frac{2\pi Hs}{g \times Tm^2}}}$$ **Equation 5**

$\xi cr = \{(Cpl/Cs) \times P^{0,31} \times \sqrt{(\tan\alpha)}\}^{(1/P+0,5)}$

P the permeability coefficient of permeable structures Cpl = 8.4/Cs = 1.3.

If ξm > ξcr, then the breaking is swelling, hence the use of the formula that corresponds to this type of breaking in shallow water.

The stability number Ns is calculated by the following formula

$$N_s = \frac{Hs}{\Delta . Dn50} = 1,3 * P^{-0,12} * \left(\frac{Sd}{\sqrt{N}}\right)^{0,2} * \left(\frac{Hs}{H2\%}\right) * \sqrt{\cot g\alpha} * (\xi_m)^{p}$$ **Equation 6**

With H2%, wave height exceeded by 2% of the incident waves at the foot of the structure. Offshore H2% = 1.4Hs; in shallow water H2% < 1.4Hs due to the change in the distribution of the swell height and the energy spectrum of the swell. We then have 1.1 < H2% < 1.4 and we will choose H2% = 1.2Hs in our study. According to the equation Ns = Hs/ (Δ. Dn50) = 1.12 of the stability number Ns, we deduce the average size and then the average mass of 50% of the blocks. If the material used is basalt, then the average diameter of the blocks of the Dn50 shell is calculated by the following formula.

$D_{n50} = \frac{Hs}{\Delta . Ns}$ **Equation 7**

Avec $\Delta = (\frac{\rho s}{\rho w} - 1) = (\frac{2900}{1025} - 1) = 1{,}83$

ρs: density of basalt: 2,900 kg/m3: and ρw: density of seawater: 1,025 kg/m3

We can then determine the mass required for blocks of the M50 basalt shell corresponding to the average nominal diameter required, using the relationship: M50 = (Dn50)3 × ρs and According to the SPM (Shore Protection Manual) we have: Dimensions of the blocks of the underlay $\mathbf{M_{50sous-couche}}$ and $\mathbf{D_{50sous-couche}}$.

$M_{50sous\text{-}couche} = 1/10 \times M_{50carapace}$ **Equation 8**

$D_{n50sous-couche} = 1/2{,}2 \times D_{n50carapace}$ **Equation 9**

As a general rule, the minimum thickness of the natural riprap shell is at least equal to the thickness of two blocks. The empirical formula providing the thickness of the shell layer according to the Rock Manual is:

$e_{carapaces} = nk_t \times D_{n50}$ **Equation 10**

n: number of layers of the shell (=2), kt: layer thickness coefficient taking into account the density of laying of the layer [4]. The average mass of the shell blocks is between 3 and 6 tons. We choose the value of kt given by Bardon Hill and Tor Works in standard double layer.

The thickness of the underlay $\mathbf{e_{sous-couche}}$ is determined by the following formula

$e_{sous-couche} = nk_t \times D_{n50sous-couche}$ **Equation 11**

The materials are spread granularity and several blocks of different thickness (1 - 30 kg of sandstone, limestone or basalt). Considering a permeability of P = 0.4, Van Der Meer defines the calculation of the dimensions of the elements of the nucleus $\mathbf{Dn_{50noyau}}$ et $\mathbf{M_{50noyau}}$ as follows:

$D_{n50noyau} = \frac{D50sous-couche}{4}$ **Equation 12**

$M_{50noyau} = (D_{n50noyau})^{3} \times \rho r$ **Equation 13**

The toe stop is the lowest part of a coastal or river defence structure, which can support the protection of embankments and/or provide protection against scour [4]. The carapace sizing formula must then be used for these lower values. It can be a continuation of the carapace with blocks of reduced dimensions. An empirical formula presented in the book Ouvrages de protection contre la houle written by Daniel Caminade allows us to write:

$M_{50butée} = (\frac{ht}{1{,}27 * Hs}) * (\frac{M50carapace}{10})$ **Equation 14**

Using the formula of Pilarczyk (1998) presented in the Rock Manual, we can calculate ht (water height above the toe stop) as follows:

$h_t = (\frac{(\frac{Nsbutée}{Nod\ p0{,}15}) - 2}{6{,}2})^{1/2{,}7} * h_{pied}$ **Equation 15**

Nod = 2, corresponds to an accepted damage of a slight flattening of the stop for a ratio of 0.8, stability is perfectly ensured, and damage is reduced. We will opt for a stability of the stop Ns, stop = 6.5

$h_t = (\frac{(\frac{6{,}5}{2p0{,}15}) - 2}{6{,}2})^{1/2{,}7} * 4{,}13$ **Equation 16**

For better stability of the toe stop, the ht/hft ratio = 0.8.

The height of the stop is; humble = hpied − ht

The width of the toe stop is: toe width = 3 to 4 (Dn50 butée)

$3 \times 0{,}66 \leq largeur_{butée} \leq 4 \times 0{,}66$

At the foot of the structure, we can be confronted with a phenomenon of digging of the bottom. The slope of (2/5) will reduce these risks. Depending on the degree of scour (moderate or severe), several types of protection of the foot of the dike can be considered. The presence of sands and shell sands on the spires of Joal and Djiffère shows us that the option is to be considered for the protection of the foot. The basic principle of protecting the foot of a dike will be to dig a trench of at least 0.5 Dn50 depth beforehand which will constitute an extension of the shell. The width of the B ridge should be at least 3 to 4 times greater than the nominal median diameter of the riprap constituting the carapace.

$B_{min} = 3 \times D_{n50}$ $B_{max} = 4\times D_{n50}$ **Equation 17**

$3\times D_{n50} \leq Bmin \leq 4\times D_{n50}$

Pressure meter measurements carried out in the area revealed that the sand along the coast gives average limit pressures of 0.25 bar at the surface and becoming higher at depth, reaching values of around 1.8 bar. It can be said that the floor in place can support loads of up to 10T/m² on average.

$\boldsymbol{qu'}$ = 10 t/m^2; $\boldsymbol{\sigma O}$ = 9.8 t/m^2

Between the sand and the filter layer, a geotextile must be installed to prevent the sand from migrating towards the filter layer and the shell and thus causing the structure to subside. The geotextile must have characteristics adapted to its. The geotextile must consist of a filtration layer and a protective layer. The geotextile will be of the non-woven type, needle-punched with continuous polypropylene filaments [8]. The performance of crossing a slope dike is often improved by the use of a concrete crown wall. The protection must make it possible to stop the retreat of the shoreline in order to prevent houses from collapsing, but is not intended to limit the marine submersion of the upper and lower beaches during storms associated with high water levels. The structure will therefore remain passable by waves with a height that varies from 2.6 m for a return period of one year to 3.5 m for a return period of 100 years [9]. However, the crossings will be exceptional and the flows limited, because the level of the crest of the dike is set at 1.5 m above the highest water levels (including global warming).

3.2. Modelling of the natural riprap dike on Plaxis 3D

In order to better visualize the behavior of our dike and its interaction with the soil in place, a numerical simulation is carried out using the Plaxis 3D software. The geotechnical reconnaissance of the site has shown a stratigraphy of the soil consisting of: More or less clayey dune sands, they cover almost the entire region with an average thickness of 10 m. These sands rest on a lateritic armour, which are ferruginous gravels coated in a clay-sandy matrix, 1 to 2 m thick [10]: In the modelling, the procedure used is as follows: The creation of the physical model of the dimensions of the dike with the defined boundary conditions. The dike to be considered is 9.0 m high. The width of the top and the width of the base of the dike are 4.5 m and 37.60 m respectively. The normal water level behind the dike is 2.5 m high from the subgrade. We are considering a situation where the water level drops by 4 m. The definition of the characteristics of the initial soil, the materials for the layers and the chosen law of behaviour corresponding to the Mohr-Coulomb law, which is adapted to the available data [11]. These geometric, physical and geotechnical characteristics are summarized in Table 3.

Table 3: The characteristics of the materials chosen for the dike model

Parameters	Symbols	Sea sand	Kernel	Layers
Dry density kN/m^3	γd	18,19	18	30
Density weight hum kN/m^3	γh	21,00	19	0,1
Vertical permeability m/s	Ky	1,30	$3{,}6.10^{-3}$	0,25
Horizontal permeability m/s	Kx	1,10	$3{,}4.10^{-3}$	0,25
Young's modulus kN/m^2	E	$3{,}07.10^3$	$3{,}25.10^3$	6.10^4
Poisson's Ratio	ν	0,28	0,3	0,15
Cohesion kN/m^2	C	1,01	0,1	20
Friction Angle °	Φ	34,6	35,3	40
Angle of Expansion °	Ψ	4,50	1,0	10

The generation of the mesh, we set the mesh fineness (Global Coarseness) to "fine". The reference model is made by 40-knot elements. The number of elements is 800 elements and the number of nodes is 7600 nodes. The definition of the initial conditions, for the calculation of the initial stresses, the initial stresses are generated by taking the default values of K0, the value of K0 is taken automatically according to Jacky's formula, the weight of the ground is kept at 1, which corresponds to a total application of gravity [12]. The hydraulic conditions, the initial sea level is more than 15 m deep, for the calculations in the initial conditions it was taken at the base of the model, i.e. at the level of the sand at -30m (Figure 6).

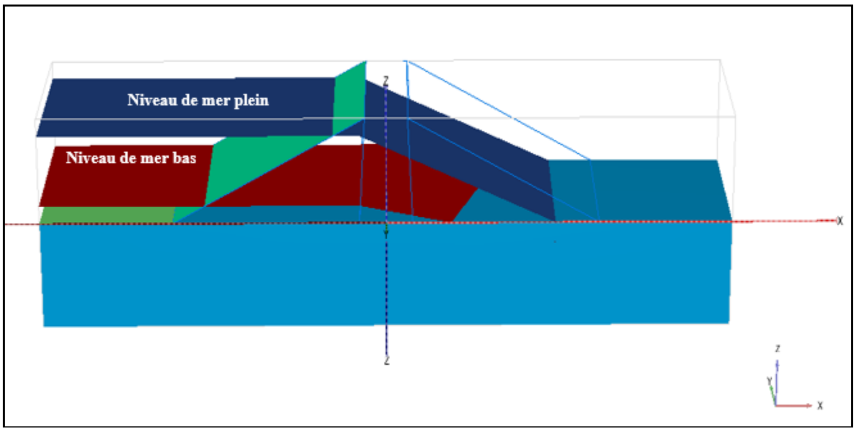


Figure 1: sea water level in contact with the dike

The calculation phases, the first calculation is made with the data mentioned above according to the following phases:

Phase 0 Initial Phase

Phase 1 Plastic phase with very low sea water level

Phase 2 Solid dike plastic phase at 15 m level

Phase 3 Safety factor for Phase 2

The preparation of the calculation phases is illustrated in Figure 8.

4. RESULTS AND DISCUSSIONS

4.1. Design results

Dimensioning makes it possible to obtain the most appropriate physical dimensions for the structure or the element implemented from its technical aspect. These elements of the composition of the protective structure are summarized in Table 7 and they allow us to know the geometry of our structure.

Table 4: Summary of the dam design parameters.

Slope of the structure tan α = 2/5		**Mean Iribarren coefficient ξm = 2.46**	
Acceptable Damage Level Sd = 2		Stability number Ns = 1.12	
Width of the crest 4.68m ≤ Bc ≤ 6.24 m		Number of waves N = 1532	
Revenge crest Rc = 4 m		Width 1.98 m ≤ width ≤ 2.64 m	
Caractéristiques	D_{n50} (m)	M_{50} (kg)	e_{couche} (m)
Carapace	1,56	11010	2,80
Sous-couche/filtre	0,71	1101	1,31
Noyau	0,18	16,90	Variable
Butée de pied	0,66	1937	1,72

The cross-section of the dike is shown in Figure 9 at a scale of 1/10m

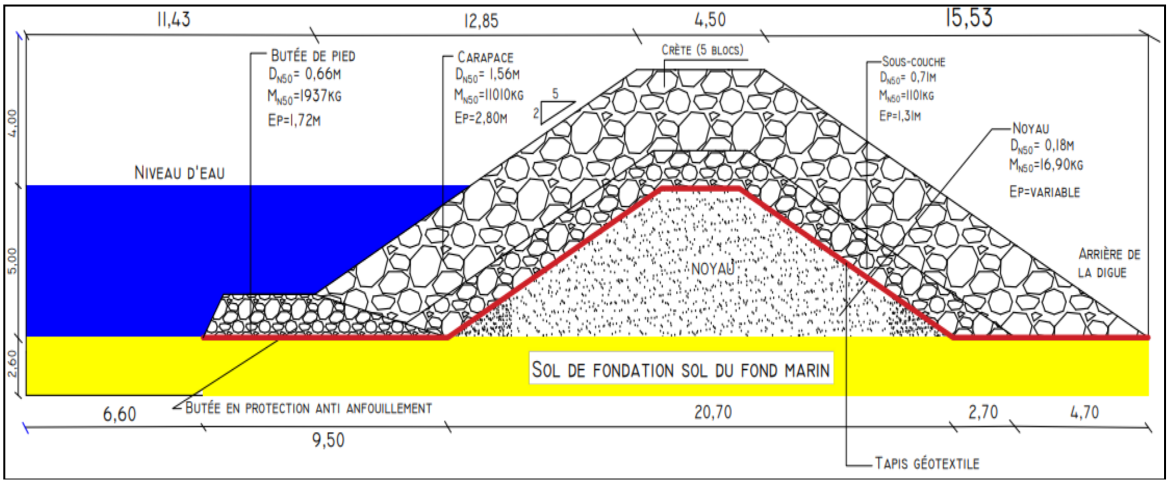
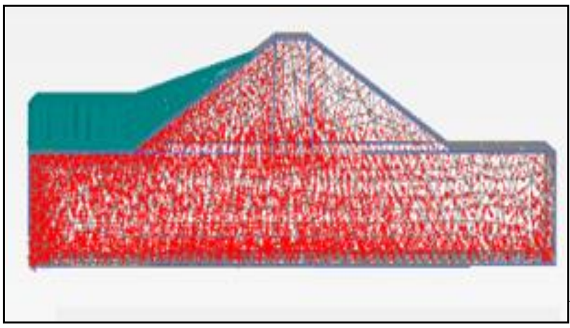
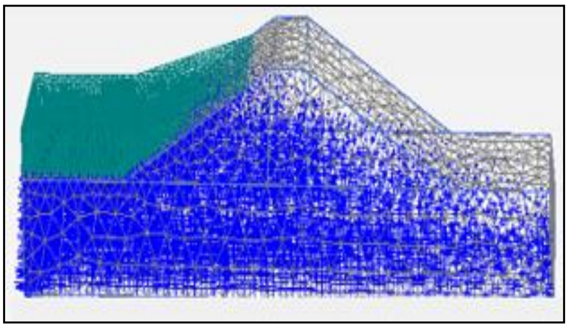


Figure 2: Summary diagram of the dike cross-section

This part allowed us to determine all the parameters necessary to quantify the riprap structures to be put in place. As the conditions of stability were ensured, an analysis of the project's impacts on the sustainable development of the Joal and Djiffère coastline was carried out in order to propose the correct position of the dike to avoid blockage of the coastal drift and sedimentary transport [13]. The dike will reduce the energy of the swells but also reduce their incident heights by 72%. These actions will result in the deposition of sand and the fattening of the beach. The dike will help reduce the crossing and will act as a protection for the infrastructure located towards the mainland.

4.2. Results of the simulation in Plaxis 3D

The results of the phases of calculating the flow of sea water in terms of pore pressure distribution are presented in the series of figures 10. Four different situations were considered. The total stress obtained by simulation in a solid dike is 1061 KN/m2 being higher than the load provided by the structure (Figure 10a), it is clear that our dike induces permissible deformations in this soil. The pore pressures for the initial phase (sea water level) are obtained with a value of 546.2 kN/m2 (Figure 10b). For Phase 01, the numerical calculations show a distribution of the pore pressure after rapid drawdown, a vertical settlement of 10.9 cm was obtained (Figure 10c); which falls within the range of deformations tolerable for our dike, i.e. less than 20 cm. For Phase 02, the numerical calculations show a distribution of pore pressure after slow drawdown, a vertical settlement of 10.21 cm was obtained (Figure 10d); This also falls within the range of deformations tolerable for our dike, i.e. less than 20 cm. For Phase 03, the numerical calculations show a distribution of pore pressure after for a low level of the dike, a vertical settlement of 10.09 cm was obtained; which still falls within the range of tolerable deformations for our dike, i.e. less than 20 cm.

a

b


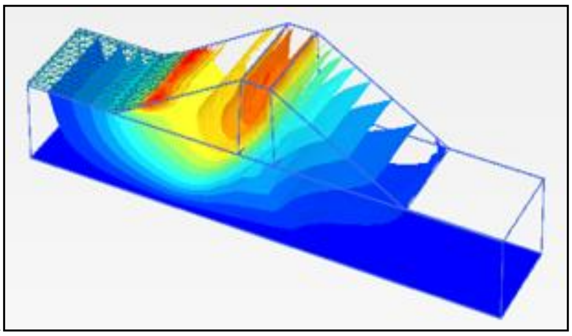
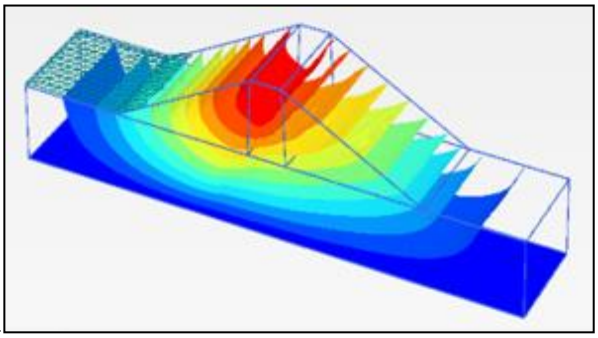
c  d 

Figure 3: Distribution of pore pressure for the different phases (a. total stresses solid dike 1061 kN/m², b. pore pressure for a high water level, c. pore pressure after rapid drawdown, d. pore pressure after slower drawdown)

Pore pressure for the different phases	Minimum value	Maximum value
Total stress solid dike (scaled up 5,00.10^{-3} times)	m² kN/2306.10	m²1061 kN/
Pore pressure for a high water level (scaled up 0,020 times)	m² kN/0,000	m²,2 kN/546
Pore pressure after rapid drawdown (times 2 days)	10,19m	24,92m
Pore pressure after slower drawdown	10,21m	30,00m

In this modelling, attention is focused on the variation of the safety factor of the dike for the different situations. Thus, the evolution of ΣMsf is plotted as a function of the displacement of the crest point of the dike:
It can be seen that ΣMsf = 1.861 and according to Figure 15

>1.4	**satisfactory for the dike in terms of stability**

In this part of our work, we were able to establish the model corresponding to the dike based on the Plaxis 3D V20 program for the stability study. In this study, the rapid drawdown of sea level in 5 days did not influence the safety factor from ΣMsf = 1.861 to the value of the last phase ΣMsf = 1.972 Thus, the rise in sea level has the effect of producing additional deformations. Thereafter, the rate of deformation generally decreases over time, with the exception of variations associated with periodic changes in sea level in certain periods.

CONCLUSION

For a very long time, coastal areas have been of particular interest to humans, as places of settlement, trade, passage, or resource generators. Today, we are increasingly witnessing the disappearance of this particular space, threatened by natural and anthropogenic processes. The need to protect coasts from erosion has become a major concern. It is in this context that this article is part of the modeling and sizing of a protective dike in natural riprap. The objective was to determine all the parameters necessary to quantify the riprap structures to be installed, to evaluate the stability of the dike on the foundation soils by means of a numerical model.

Declaration by Authors

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REFERENCES

1. Djagouna ERIC M. Valere., (2016). Cartographie De La Dynamique Du Trait De Côte A Grand-Lahou: Utilisation De L'outil « Digital Shoreline Analysis, System (Dsas) ».
2. Faye I., (2010). Dynamique du trait de côte sur les littoraux sableux de la Mauritanie à la Guinée-Bissau (Afrique de l'ouest): approche régionale et locale par photo-interprétation, traitement d'images et analyses de cartes anciennes. Thèse de l'université de Bretagne occidentale.
3. Niang-Diop I. (1995). L'érosion côtière sur la Petite-Côte du Sénégal à partir de l'exemple de Rufisque. Passé – Présent –

Futur. Thèse Université, Angers, tome 1, 318 pp, 112 fig., 47 tab.

4. CIRIA, CUR, CETMEF, (2009), "Guide Enrochement. L'utilisation des enrochements pour les ouvrages hydrauliques. Version française du Rock Manual, P09-01, CETMEF, Compiègne").
5. Kleine, A. (2007). Modélisation numérique du comportement des ouvrages souterrains par une approche viscoplastique. Lorraine, Françe.
6. Jenck, L. T. (2002, mars 4). Modélisation physique en géotechnique, 1re partie. Revue française de géotechnique.
7. Yves Birame Diadhiou, Abdoulaye Ndour, Isabelle Niang et Awa Niang-Fall, (2016). Étude comparative de l'évolution du trait de côte sur deux flèches sableuses de la Petite Côte (Sénégal): cas de Joal et de Djiffère.
8. Luc Thorel, O. J. (2021). Modélisation physique en géotechnique, 1ère partie. Rev. Fr. Geotech., 2.
9. SALL M. (2006), "Crue et élévation du niveau marin à Saint-Louis du Sénégal : impacts potentiels et mesures d'adaptation ". [Thèse de doctorat], 332 pages.
10. SARR, R. (1998) - Les ostracodes paléocènes du horst de Diass (Sénégal): biostratigraphie, systématique, paleoen vironnement. Rev. MicropaLeont., Paris, 41 (2): 151-174.
11. Mammeri, B. (2017). Les lois de comportement.
12. BITAR, A. A. (2007, Juin 4). Modélisation des écoulements en milieu poreux hétérogènes 2D / 3D, avec couplages surface / souterrain et densitaires. TOULOUSE, France.
13. Abdelali, S. M. (2011, 02 21). Effet des paramètres géotechniques sur la modélisation numérique des ouvrages souterrains. Algerie.

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