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Parametric and comparative study of a flexible retaining wall

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ABSTRACT. *The purpose of this work focuses on numerical modeling and behavior analysis of a self-stabilizing retaining diaphragm wall, constructed in sand using conventional analytical calculation method based on subgrade reaction coefficient and by numerical method with finite elements method. For both two methods, we proceed to various simulations with the structure installed into the supported ground without surcharge. For the first method, the analysis has investigate the influence of main factors such as the wall rigidity, the different stages of excavation work, the Young's modulus, the cohesion and internal friction's angle of soil. The analysis of the FEM results is focused on wall deformation, bending moments and horizontal displacements. These results confronted with experimental and numerical results (GAUDIN, 2002), consequently show a very good coherence results.*

KEYWORDS

Centrifuge, diaphragm wall, finite elements, interface, numerical modeling, retaining wall, subgrade reaction.

1. Introduction

The design of retaining walls in day-to-day practice is currently based on different calculation methods. If classical methods considering specific modes of failure, the elastic line and the equivalent beam are still employed for certain types of walls, it is mainly the subgrade reaction and the numerical methods which are most frequently adopted. The subgrade reaction method or spring method is rather well mastered and uncertainties mainly rely in the choice of the coefficient of subgrade reaction (DELATTRE, 2001 and MONNET, 1994). The numerical methods have the advantage of taking into account more accurately the soil behavior, the soil-wall interface and also the ability to consider multiple hydraulic conditions and various options for modeling support conditions. However, the results obtained by these methods still require to be validated by engineer judgment or others experimental results (physical model in centrifuge for example) or measured in-situ. The main objectives of this study were to define the influence factors of the commonly used design methods and the resulting uncertainties encountered by the practitioners. This work focuses on numerical modeling and analysis the behavior of a free standing diaphragm wall, made of reinforced concrete, embedded in sand, by the subgrade reaction method using the K-REA software and by the numerical method based on finite elements with PLAXIS 2D-v8.5 software. For both two methods, different simulations have been performed with non-loaded supported soil. For the first method, we are interested in analyzing the influence of the main factors affecting soil movement and instability of the retaining wall. These factors mainly concern the wall rigidity, the construction sequence and mechanical parameters of the soil. One key step of this method is the difficulty in evaluating the coefficient of subgrade reaction K_h on a rational basis. Concerning the finite element method, the soil is homogeneous and dry, its behavior is described by an elastoplastic behavior and MOHR-COULOMB criteria with two different values of cohesion noted C and angle of internal friction ϕ , the diaphragm wall is modeled by "beam" elements. The simulations were performed with different mesh sizes and reduction factors of the soil-wall interface. For the two methods, the analyses are focuses on the wall deformation, the bending moments and horizontal displacements. These results obtained, are confronted with other experimental and numerical results (GAUDIN, 2002). This paper summarizes the performance of this parallel formulation and results obtained from the simulation of this diaphragm wall. The computational strategy employed in this study offers practical approach for performing finite element simulations in day to day practice.

2. Historical overview

Physical modeling using centrifuge is a complementary way of study and research in addition to more theoretical approaches and tests on full-scale structures whose objectives are to study the behavior of geotechnical structures or dimensioning exceptional structures. During the year 1999, the British Geotechnical Society has ranked centrifuge modeling in the fifth place between the most important developments in geotechnical fields (CHARLES, 2014). In this period, experimental works undertaken by LYNDON and PEARSON, 1985 have studied the effect of the pressures on the structure during failure under rotational and translational kinematics when GARNIER et al., 1987 focused on the influence of wall's roughness on the structure behavior. BOLTON et al., 1988 have investigated the deformation and failure mechanisms of a rigid retaining structure during excavation at short and long term. During the same year, ZHU and YI, 1988 used physical modeling tests to simulate real retaining structures. Since the year 1994, the influence role of the structural elements on retaining wall behavior, was taken into account and we remind here especially the work of POWERIE et al., 1994 that aimed at modeling the process of excavation installation and bracing of a diaphragm wall in clay. In the same context, SCHURMANN and JESBERGER, 1994 have performed centrifuged tests to study the pressure profiles developed on a sheet piling driven into dry sand during excavation. In the same period, a qualitative step forward was given to this technique and mainly relates to the first excavator device operating during flight for studying centrifuged excavations models developed by KIMURA et al.; 1994. This tool was used in other studies including those conducted by TAKEMURA et al., 1999. During the year 1998, it became possible to study the three dimensional behavior of structure as done by LOH et al., 1998 to observe the behavior of two freestanding retaining walls, stuck in a reconsolidated kaolin clay. In practice, the tests conducted by TOYOSAWA et al., 1998 aimed at studying the possible failure mechanisms mobilizing the ruin of an anchored sheet piled model. In 1999, it was made a feasibility study, to establish a reduce thickness and an optimum instrumentation of wall to measure the satisfactory bending moments. Thanks to improvements given to centrifuges including the use of teleoperator during flight, GAUDIN (2002) conducted experimental tests to study the behavior of a flexible and free standing retaining wall. These works currently remain the reference for further researcher on the behavior of this category of geotechnical structures. More recently, several researchers including MATSUO et al. 2002, NAKAMURA, 2006, AL ATIK and SITAR, 2008, MORIKAWA et al., 2011, GABRIEL and SITAR 2013, DASHTI et al., 2013, have pursued studying behavior of retaining walls but focused on seismic aspect using shaking table embedded in the centrifuge.

In order to point out of the use evolution of the centrifuge in the geotechnical fields, we have attempted to update the histogram showing the number of papers dealing with centrifuge experiments by categories of structure prepared by CORTÉ and GARNIER, 1986. To do this, we used a range of recently published articles that focus particularly on physical modeling centrifuge. This histogram updated and shown in figure1 is not exhaustive and has no indicative value. It results that tests on shallow foundations have kept their first position while those on retaining walls hold the same importance with much progress. Both, they represent the same greater percentage which is worth approximately 16% from this range of articles. However, the deep foundations and slopes lose their rankings in first and second places. Down, they account for 13% and 14%. In the similar row, the buried pipe, tunnels and cavities represent separately the same percentage of 12%. With less progress, the trenches, reinforced soils, dams and embankments account for 4% and 6% of studied articles.

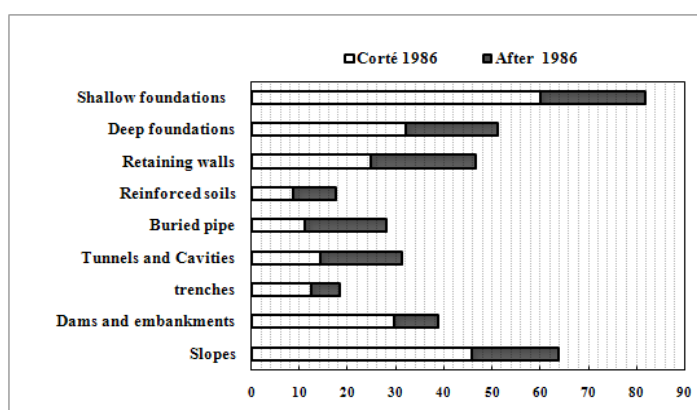


Figure 1. Indicative distribution by type of works of publications dealing experiments in centrifuge (After Corté, 1986)

3. Model used for the benchmark

Due to the great step forward achieved by GAUDIN, 2002 in IFSTTAR (Institut Français des Sciences et Technologies des Transport de l'Aménagement et des Réseaux) during experimental work on a reduced-scale model retaining wall, embedded into dry sand, to investigate its general behavior as well as interactions with a strip foundation, we have chosen this tests as reference for our numerical study.

3.1. Model description

The model wall reduced to 1/50th scale, was constituted of AU3G aluminum, 2mm thick and 24 cm high, which thus represents a prototype wall 12 m in height (of which the first 10 m have been penetrated into the ground) with a product of inertia EI equal to 6.54 MN.m² (corresponding approximately to an Arbed PU6 type profile). The model has been placed in a rectangular container 1200 × 800 × 360 mm in dimensions. 22 pairs of strain gauge sensors instrumented the central part of the wall. Measuring gauges gave directly the bending moment at the considered depth. Other instrumentation attached to the model allowed the determination, during the excavation, horizontal displacements of the wall, as well as settlement of supported earth. We note here that the centrifuge tests have been described in detail in [37].

3.2. Mechanical parameters of the model

The material used for this study is the Fontainebleau sand. This mono granular sand is commonly used in centrifuge or calibration chamber tests in France. It is fine and clean siliceous sand (GAUDIN, 2002), its shear strength parameters are derived from drained and undrained triaxial (compression and extension) shear tests having the following characteristics: γ (KN/m³) = 16; c (kPa) = 2.60; ϕ (degree) = 39.40. For the diaphragm Wall, an elastic law has been adopted, characterized by an elastic modulus $E = 22.350$ GPa, having an equivalent thickness of 0.152m and height of 10m.

4. Subgrade reaction coefficient method

In the first phase of this study, numerical simulations of this free-standing diaphragm wall were performed using the subgrade reaction method. These simulations were made using K-REA software for modeling and analyzing the behavior. The calculation is based on the determination of the active/passive earth pressure coefficients. The literature suggests three main methods to determine these coefficients; namely COULOMB, RANKINE methods, CAQUOT-ABSI and KERISEL tables. The subgrade reaction coefficient calculation can be made by the three methods of BALAY, SCHMITT formulas and the CHADEISSON abacuses. To find a better combination between these so-called calculating methods and subgrade coefficient determination methods, a preliminary comparative study was conducted. This confrontation with the computation results of numerical calculations, allows choosing of binomial methods, one for active and passive earth pressure coefficients calculating and another one for the subgrade reaction calculation. Once, the best combination between these methods is determined, we have proceed with simulations to study the behavior of the wall which gave the results shown hereafter.

4.1. Results and interpretations

The results obtained from the different simulations as shown in the table 1, figures 2, 3 and 4 below, suggest the following main conclusion:

It appears that the combination of the two separate methods of RANKINE and KERISEL tables with CHADEISSON abacuses, gave results very close to those obtained in centrifuge experiments for the test labeled A01 (GAUDIN, 2002). The maximum bending moment estimated at 121kN.m/ml, is also coherent with the experimental result with a slight difference for the maximum horizontal displacement estimated of 37 and 37.10 cm close to the 37.85cm value obtained during experiments. Also, the BALAY formula with RANKINE method gives maximum bending moment

estimated of 121kN.m/ml and in the same way maximum horizontal displacement was estimated of a 37.9cm very close to 37.85m. The SCHMITT formula strongly underestimated the results for the three methods, due to higher reaction coefficient. However the others methods give a close subgrade reaction K_h . Hence, this coefficient from SCHMITT formula is greater than those from CHADEISSON and BALAY methods. That is to say for example $400821 < 52238 < 58411$ for $C=2.6\text{kPa}$.

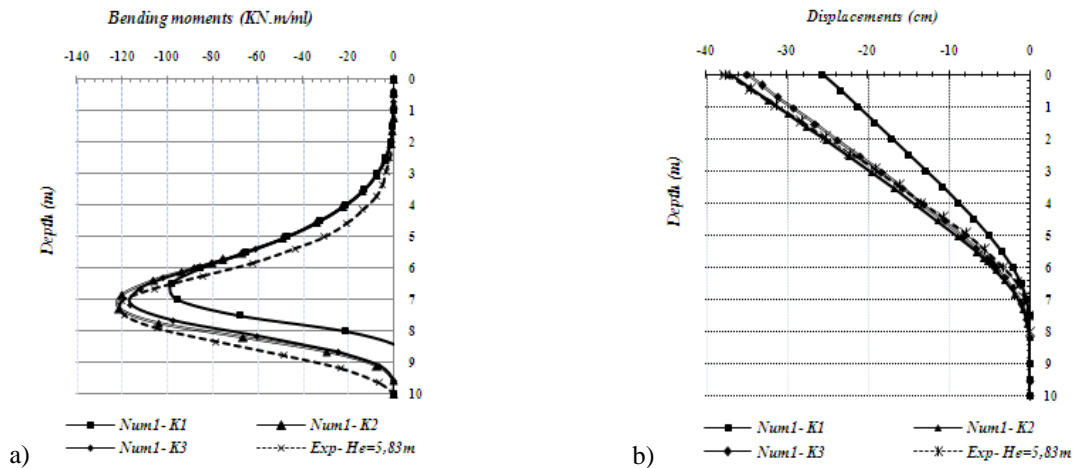


Figure 2. (a) Bending moments and (b) horizontal displacements profiles obtained with k_h derived by the 3 studied methods (K1 BALAY, K2 SCHMITT and K3 CHADEISSON) – with KERISEL and ABSI's tables.

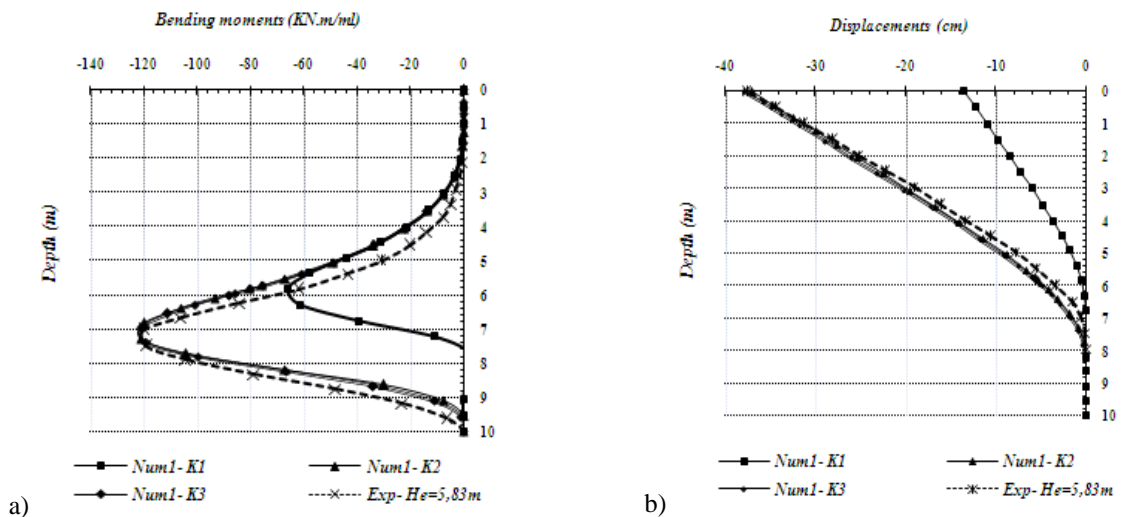


Figure 3. (a) Bending moments and (b) horizontal displacements profiles obtained with k_h derived by the 3 studied methods (K1 BALAY, K2 SCHMITT and K3 CHADEISSON) - With RANKINE's method

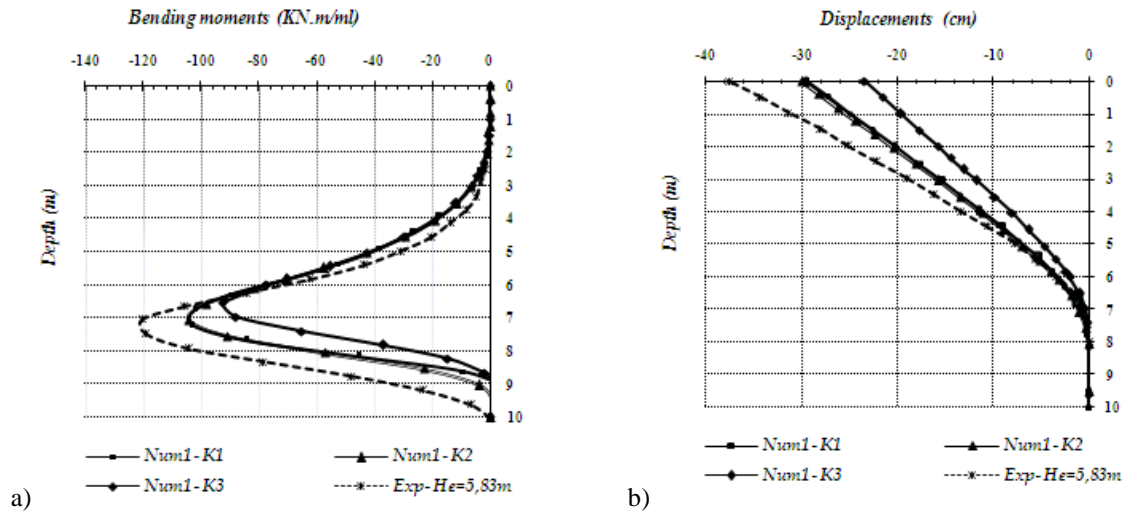


Figure4. (a) Bending moments and (b) horizontal displacements profiles obtained with k_h derived by the 3 studied methods (BALAY, SCHMITT and CHADEISSON) – With COULOMB’s method.

Table1. Results obtained by combining methods.

	Exp	BALAY formula			SCHMITT formula			Abacuses CHADEISSON		
		Coulomb	Rankine	Kerisel & Absi Tables	Coulomb	Rankine	Kerisel & Absi Tables	Coulomb	Rankine	Kerisel & Absi Tables
Height referential excavation (m)	5.73 5.83 5.95									
Height limit excavation (m)	6.39	6.55	6.55	6.64	5.83	4.90	5.49	5.83	6.39	6.39
Maximum bending moments (kN.m/ml)	120.90	-92.50	-121	-116	-103	-66.10	-98.20	-104	-121	-121
Maximum Horizontal displacements (cm)	37.85	-23.40	-37.90	-34.90	-29.50	-13.60	-25.60	-29.90	-37.10	37

4. 2. Validation on experimental results

For the second stage of the computations, the first combination using the CHADEISSON’s abacuses and the three methods is chosen in order to validate the experimental results. Various simulations are designed and performed to verify the convergence of calculations during the stages excavation to reach experimental heights 5.73m and 5.83m corresponding to the respectively noted experimental tests A01 and A02 (GAUDIN,2002). We present here only the results obtained by combining of the RANKINE method with the CHADEISSON’s abacuses as it is illustrated according to figures 5 and 6 below.

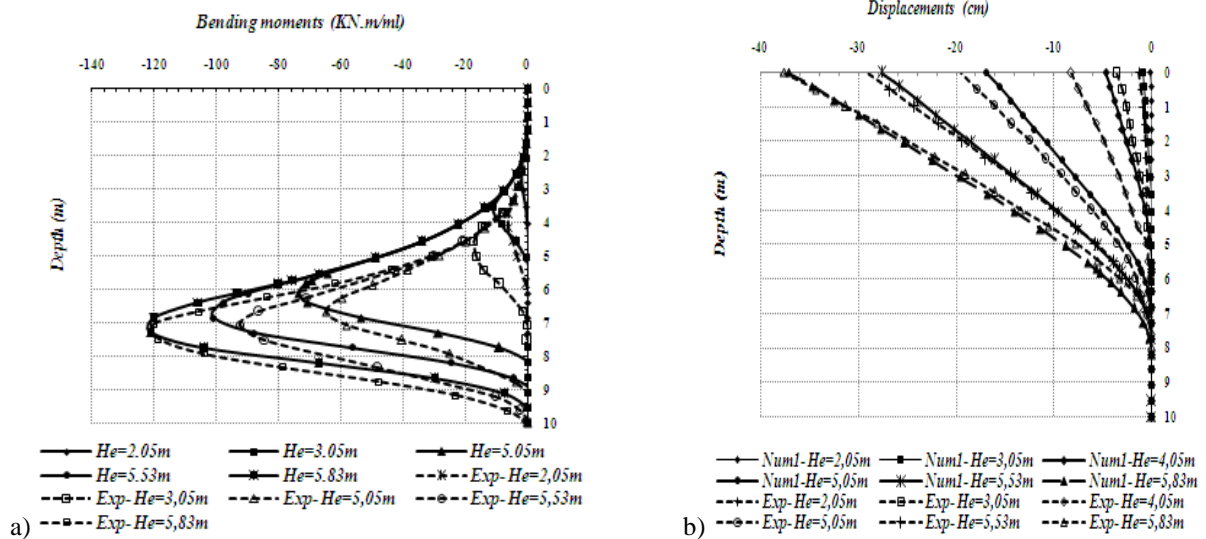


Figure5. (a) Bending moments and (b) horizontal displacements profiles obtained with k_h derived by the combined methods (RANKINE and CHADEISSON)

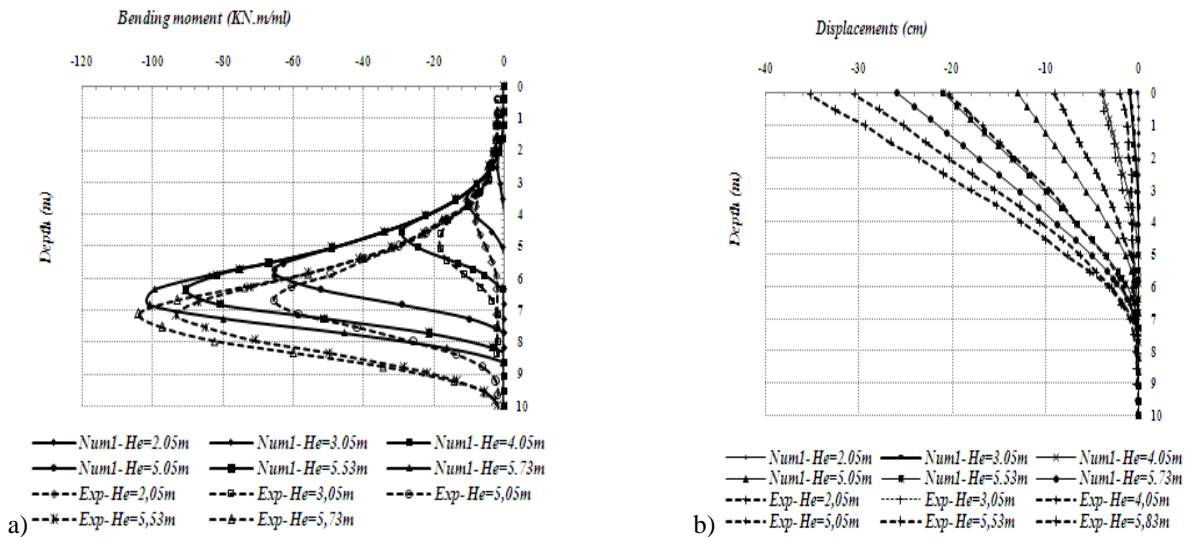


Figure6. (a) Bending moments and (b) horizontal displacements profiles obtained with k_h derived by the combined methods (KERISEL & ABSI Tables and CHADEISSON)

4. 2.1. Interpretation and comments

The main conclusion drawn from these computations are that:

- (a) As, the two heights of excavation (5.73m and 5.83m) obtained during experiments are correctly verified, the diaphragm wall behavior is also correctly transcribed and it is generally consistent with experimental observations;
- (b) The values of maximum bending moments are conform with experimental results and form part of the ranges of experimental values but they are underestimated at the beginning of excavation and especially for the first four stages;
- (c) Maximum displacements corresponding to excavation heights 5.73 and 5.95m, having respectively values of 27.20cm and 33.6cm, are underestimated of less than 20% and 32.80% of the experimental results (34cm and 50cm).

Although, this method of subgrade reaction coefficient appears able to transcribe the wall behavior, it seems limited to estimate accurate lateral displacements.

5. Numerical modeling of the freestanding diaphragm wall using finite element method.

The numerical modeling of the retaining wall was performed using the PLAXIS D.V8.5 software. The ground was

modeled using elastoplastic model with MOHR- COULOMB criteria; the diaphragm wall was modeled by “beam” element and not massive element as it has been used by GAUDIN, 2002. There is place to note that the numerical model dimensions replicate those of the prototype structure and not the reduced-scale centrifuge model submitted to 50g acceleration. The geometrical dimensions chosen for this model, are those advised for modeling in plane strain of an unsupported excavation with the maximum sizes (CHEANG, 2008). These dimensions remain smaller than those established by MESTAT, 1997 who recommended a distance behind the wall of greater than six times the excavated height and a depth underneath the wall equal to four times the excavated height. The horizontal displacement for the vertical boundaries of the numerical model is nil ($u=0$), as well as the vertical displacement along the lower boundary ($v=0$). Four different kinds of meshes from coarse to dense were used to insure the reproducibility and minimize the divergence of the results. The computations are performed by stages excavation with a height of 1m except for the two or the three last stages according to the simulated cases. This height is fixed according to the convergence of calculations or the height of desired excavation- (Figure7). The reduction factor of interaction soil-wall (R_{inter}) has been chosen equal to 0.80; 0.88 and 1.

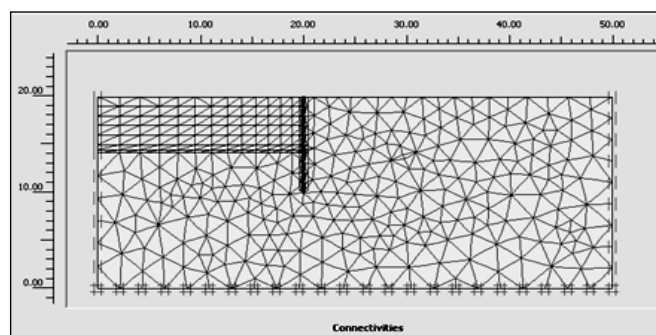


Figure7. Geometries and mesh used for the numerical model

5.1. Properties of soil

The soil consists of a single homogeneous layer of Fontainebleau dry sand. The analyses assume fully drained conditions throughout the profile and model the sand behavior using linearly elastic, perfectly plastic model with MOHR- COULOMB criteria. Two different values of cohesion C have been used and one value of internal friction's angle of soil ϕ . The parameters of E_{ref} , C_{ref} , ψ and R_{inter} are variables according to the simulated case as indicated in the table 2.

Table2. Properties of the soil layers and interfaces.

E_{ref} (MPa)	ν	C_{ref} (kPa)	ϕ (°)	Ψ (°)	R_{inter}
10	0.275	0 or 2.60	39.40	16.70	0.80,0.88 or 1.00

5.2. Properties of the diaphragm wall

The diaphragm wall is modeled by elastic beam elements. The properties of reinforced concrete are:

D (m) = 0.152; E (MPa) =22350; $\nu=0.3$; H (m) = 10.

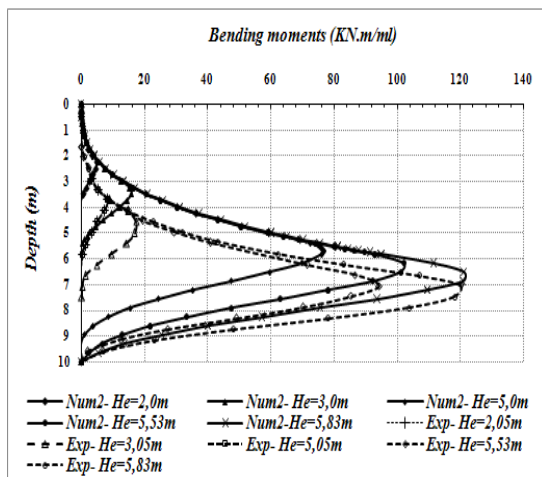
Table3. Compared results of numerical simulations

R_{inter}	Mesh type	Maximum bending moment	Calculated	Maximum horizontal displacements	Calculated	
			result/experimental Result (%)		result/experimental Result (%)	
E=10MPa	1	Coarse	112.8	94%	34.11	90%
		Medium	115.54	96%	33.73	89%

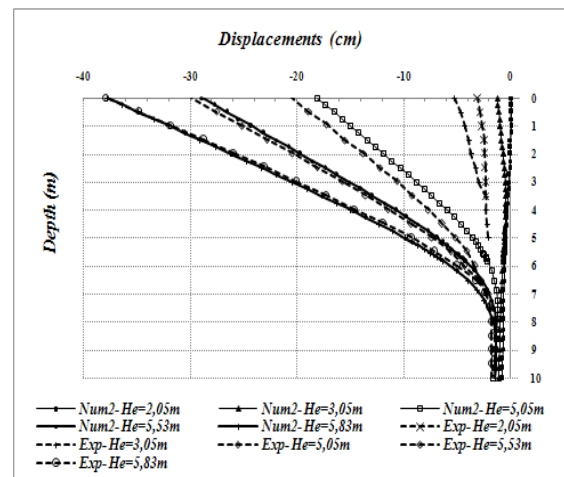
$C_{ref}=0\text{kPa}$		Fine	117.90	98%	33.95	89%
		Dense	120.8	101%	35.81	94%
	0.8	Fine	121.98	102%	40.59	107%
		coarse	119.22	99%	38.7	102%
	0.88	medium	121.22	101%	37.8	99%
		fine	117.80	98%	33.95	89%
$E=10\text{MPa}$	1	Coarse	58.54	49%	11.02	29%
		medium	60.34	50%	11.34	30%
$C_{ref}=2.60\text{kPa}$	0.88	medium	62.88	52%	15.28	40%
		fine	65.50	55%	15.28	40%

5.3. Calculations and results

We present only here after, the results obtained by the simulations performed to verify the convergence of calculations during the phase's excavation until reaching the height of 5.83m obtained during experiments. In the following figures, the inserted abbreviations below are used.



a)



b)

Figure8. a) Bending moments and (b) horizontal displacements profiles versus heights of excavation with $E_{soil}=10\text{Mpa}$, $C=0\text{kPa}$ and $R_{int}=0.88$

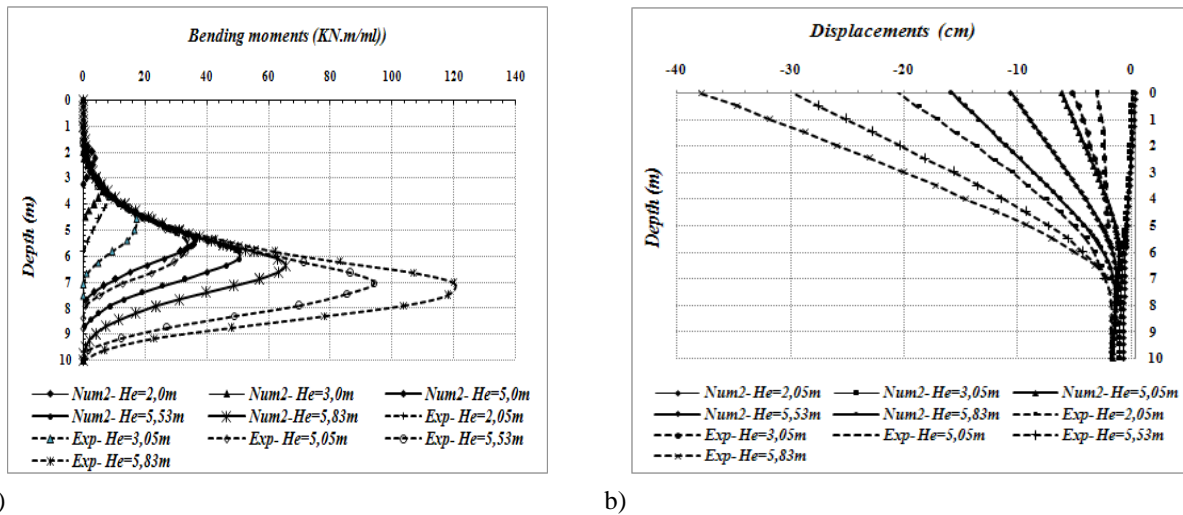


Figure 9. a) Bending moments and (b) horizontal displacements profiles versus heights of excavation with $E_{soil}=10\text{Mpa}$, $C=2,60\text{kPa}$ and $R_{int}=0.88$

5.4. Interpretation and comments

The main results drawn from these computations are like following:

- (a) The experimental height of excavation (5.83m) is correctly estimated.
- (b) The diaphragm wall behavior is correctly transcribed and it is over all in conformity with the one observed in the experimental tests (Profiles of the horizontal displacement and bending moments).
- (c) The profiles of displacements at the head illustrate clearly the level of embedment length which is located between 6m and 10m. Indeed, the deformation affects only the party above the bottom of the excavation.

A. For a zero soil's cohesion

For soft contact ($R_{inter}=1$), the values of maximum bending moments are underestimated in the case of “Coarse, medium and fine” meshes. In the three cases, the differences do not exceed 6%. As against, in the case of a dense mesh, a light over-estimation has been recorded (about 1%) and more is dense the mesh more the computed values are overestimated. Indeed, for a sliding contact ($R_{inter}<1$), it appears that the values obtained according to the mesh type has a little influence on the results but the interaction coefficient of reduction directly affects the retaining wall behavior. In the other hand, the values of maximum horizontal displacements are ranged in an interval from -11% to 7% compared to the experimental result. However, for soft contact ($R_{inter}=1$), the values are underestimated from -11% to -6%.

B. For a nonzero soil's cohesion

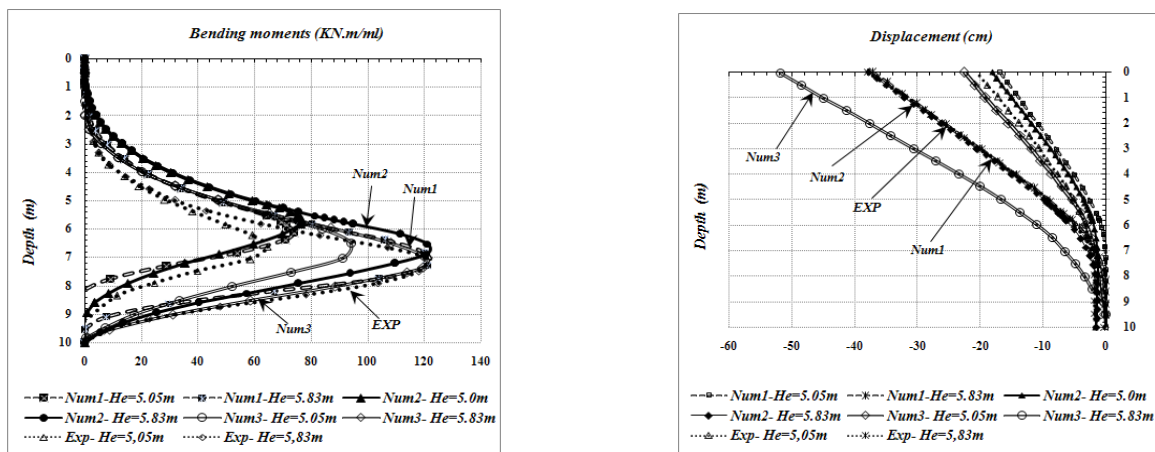
The maximum values of bending moments range in an interval of (-51 to -45%) when those for maximum horizontal displacements range also in an interval from (-71 to -60%) compared to experimental result. In the same way, more the mesh is denser more the underestimation of the results decreases. The elastic modulus and the interaction coefficient of reduction have little influence on the computation results.

In conclusion, for a soil cohesion “ $C=0\text{kPa}$ ” using a sliding contact ($R_{inter}<1$), the computation results are satisfactory and at least two results are consistent with experimental results. However, for the cohesion of the soil “ $C=2.6\text{kPa}$ ”, the results are strongly underestimated whatever the type of contact.

6. Confrontations of experimental results with numerical calculations

The confrontation of numerical results and experimental results allows to compare the performances of each of the two

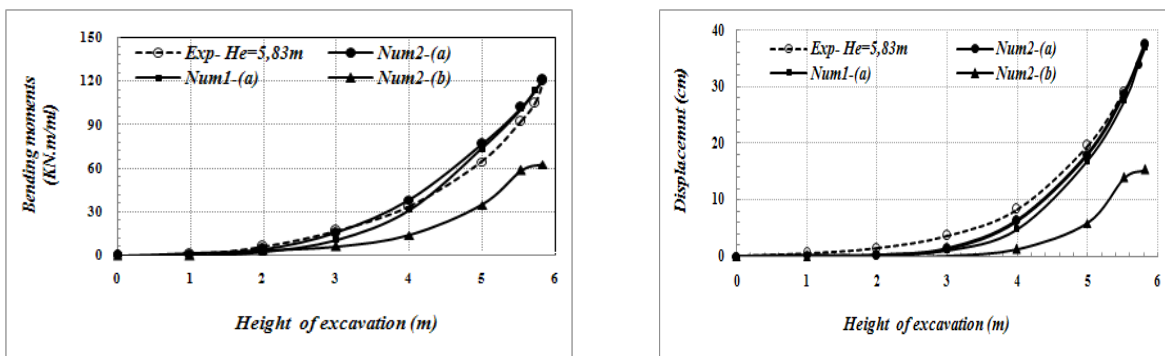
methods and to define their application. For this purpose, the various profiles of maximum displacement and bending moments resulting from the two methods are gathered and shown in figures 10 (a) and (b).



a)

b)

Figure 10. Comparison between profiles of bending moments (a) and displacements b) for different heights of excavation



a)

b)

Figure 11. Comparison between profiles of bending moments (a) and displacements (b) for He=5,83m

6.1. Interpretation and comments

The results obtained by numerical calculations Num 1 (reaction coefficient method with K-REA Terrasol) and Num3 (MEF using CESAR LCPC software more accurately), describe correctly the behavior of the flexible wall especially in the embedment length. Both computations produce bending moments in good consistency with experimental results below the bottom of the excavation and they are slightly underestimated above the bottom of that excavation. However, these moments are quantitatively underestimated by numerical calculations Num2 (MEF using PLAXIS software), see the shift observed on drawn profiles.

On the other hand, numerical calculations Num1 and Num2 estimate properly the horizontal displacements at least to the height of excavation (He = 5.83m) while they are overestimated by numerical calculations Num3.

7. Conclusion

It emerges from this parametric and comparative study the following conclusions:

- (a) For each excavation stage, a very good consistency was found between the calculated values and the values recorded in experiments; this observation is valid for the retaining wall behavior, lateral displacements and bending moments. However, it was noted a slight overestimation for maximum excavation heights compared to the experimental results.
- (b) About the methods used for estimation of the experimental excavation height, the results obtained are in good agreement with the experimental results. However, it is observed an underestimation of the results of bending moments especially in the first four stages and overestimation beyond the fourth stage. Similarly an underestimation of the horizontal displacements is obtained for all stages except the last one.
- (c) The finite element method with a zero cohesion, seems more powerful and it gave closer results to those obtained experimentally especially for the first four stages. At the time, the subgrade reaction method with RANKINE's theory satisfied similar slightly results but in paradox like finite element method (MEF), the results are strongly underestimated for a zero soil's cohesion. Unfortunately there is no explanation of this contradiction between the two methods.
- (d) The results obtained are in good consistency with those made by Yap et al. (2012). Indeed, the comparative results show that in terms of distribution and magnitude of active earth pressure, RANKINE's theory possesses the highest match to the PLAXIS and CESAR-LCPC analysis and also it has the highest compatibility to finite element analysis among all theories.

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Abbreviations

C_{ref}	Soil's cohesion
d	Equivalent thickness
E_{ref}	Young's modulus
Exp	Experimental results, (GAUDIN, 2002).
FEM	Finite element method
H_e	Height of excavation
K1	Numerical computations with Num1 using RANKINE method and K_h according to SCHMITT formula at H_e limited=4.90m
K2	Numerical computations with Num1 using RANKINE method and K_h according to CHADEISSON chart at. H_e =5.83m
K3	Numerical computations with Num1 using RANKINE method and K_h according to BALAY formula at H_e =5.83m
K_h	Subgrade reaction coefficient
Num1	Numerical computations with Subgrade reaction method using K-REA software
Num1 (a)	Numerical result with Num1 at H_e =5.83m
Num2	Numerical computations using PLAXIS 2D-v8.5 software
Num2 (a)	Numerical result with Num2 at H_e =5.83m for E =10MPa and c =0 kPa
Num2 (b)	Numerical result with Num2 at H_e =5.83m for E =10MPa and c =2.60 kPa
Num3	Numerical computations using CESAR LCPC software (GAUDIN, 2002).
Num3 (I)	Numerical result with Num3 at H_e =5.05m

Num3 (II)	Numerical result with Num3 at $H_e=5.83\text{m}$
R_{inter}	Strength reduction factor interaction
ν	Poisson's ration
γ	Soil unit weight
ϕ	Friction angle
Ψ	Dilatancy angle

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