

DIAPHRAGM WALL BEHAVIOUR UNDER THICK ALLUVIUM LAYER CONDITIONS

Idrissa OUEDRAOGO^{1,*}, Latifa OUADIF¹, Anas BAH¹

¹ L3GIE Laboratory, Mohammed V University, Mohammadia Engineering School, 10090 Agdal Rabat, Morocco.

* Corresponding author: idr.oued07@gmail.com

Abstract

Diaphragm walls play an important role in the majority of structures built on non-rocky ground. In the particular case of dams, one way of dealing with tightness issues in alluvial valleys lies in the installation of a diaphragm wall, making it possible to connect the watertight components of the main structure to the rock foundation. This article analyses the specific case of a Concrete Face Rockfill Dam, resting on approximately 35 m of alluvium. To overcome tightness issues caused by alluvium at the bottom of the valley, a plastic concrete diaphragm wall was built. The objective of this paper is to analyse the behaviour of this diaphragm wall during all phases of the dam's construction. It turns out that the plastic concrete wall undergoes a double behaviour: on the one hand, it deforms laterally with the alluvium which settles and widens on both sides of the axis of the Dam; and on the other hand, it withstands these displacements in term of stresses, limiting the large deformations. It emerges from this study that the plastic property of the concrete used in the diaphragm wall plays a major role in the consistency of the entire structure.

Keywords:

Diaphragm wall;
Alluvium;
Concrete Face Rockfill Dam;
Plastic concrete;
Modeling;
Behaviour.

1 Introduction

A diaphragm wall is defined as “a wall constructed in a trench excavated mechanically from the surface” [1] (Konrad Hruban, 1979). Almost every field of civil engineering uses different diaphragm walls for many purposes, depending on the kind of effects waited from that diaphragm wall. These structures may be used to “form land cofferdams intended for preventing seepage” according to [1] (Konrad Hruban, 1979), which cites four usages of diaphragm walls. Many studies have been conducted to ensure the stability and safety of those structures. A method to investigate lateral earth pressure applying to a circular diaphragm wall has recently been studied and proposed by [2] (Jingwu Zhang, Mingdong Li, Lijun Ke, Jinxiang Yi, 2022). The method, based on analytical solution, has been compared with the Beam on a Nonlinear Elastic Foundation (BNEF) method proposed by [3] (Wei He, Chaoyun Luo, Jianfeng Cui, Janren Zhang, 2017) and measurement made on site. This comparison made it possible to assess the validity of the analytical method proposed by [2] (Jingwu Zhang, Mingdong Li, Lijun Ke, Jinxiang Yi, 2022).

Leakage problems of diaphragm walls have also been investigated by recent studies. For this instance, Fiber Bragg Grating (FBG) sensing technology has been analysed by [4] (Yong Shao, Congxin Chen, Zude Lu, Yun Zheng, Yapeng Zhang, 2022) for leakage points detection. This issue is of interest in the present study, which carry an analysis about a soft diaphragm wall, used to prevent seepage through dam foundation. Dams, constructed as artificial barriers in watercourses, serve a crucial role by creating reservoirs for various purposes, including agricultural production, energy generation, daily life, flood control, and social activities [15] (Dong Kim HANH, Dinh Duc TRUONG, Kien-Trinh Thi BUI, 2024).

Our study focuses on the special case of a plastic diaphragm wall crossing a 35m layer of alluvium. The study is carried on Rhiss Dam, which is located in North-East Morocco. The geological investigations, consisting in geophysical techniques (as presented in [5] (Mahmut Sari, Aysel Seren,

Selcuck Alemdag, 2020)), drilling, etc., have led to the knowledge that the valley is filled with alluvium at a maximum depth of thirty-five (35) meters, laying on the bedrock (Fig. 2). Although "alluvial soils are some of the world's most useful and productive soil resources" [6] (J L Boettinger, 2005), this layer of alluvium causes at least two (2) challenges:

- Tightness: Alluviums are permeable, making them unsuitable as Dam foundation,
- Deformability: by their cohesionless nature ([7] (J.T. DeJong, A.P. Sturm, M. Ghafghazi, 2016), these materials are very deformable, inducing high deformations on the foundation, with consequences on the dam structures. For this challenge, [14] (Bziaz Mohamed, Bahi Lahcen, Ouadif Latifa, Bahi Anas And Sabihi Abdehak, 2023) has studied by vibro-compaction and recommended parameters to adopt in order to overcome liquefaction of sand foundation in a port site in Morocco. Liquefaction is a dangerous and temporary phenomenon whereby water-saturated soil loses all or part of its strength [16] (Badr BERKAT, Ahmed AKHSSAS, Latifa OUADIF, Anas BAHY, 2024).

The study consists in using numerical analysis to calculate different fields of interest in the body of the diaphragm wall: displacements field, distortion field, and constraints (stresses) field. Analysing these fields will help to mitigate the deformations and stresses in the diaphragm wall, in order to maintain optimal usage conditions.

2 Materials and methods

To meet our initial goal, we followed a precise pattern consisting of geological and geotechnical investigations, bibliography, and numerical analysis.

2.1 Site: location and investigations

This study is carried out in the Rhiss Dam site located in north-east Morocco, in the province of Al Hoceima. Fig. 1 shows the dam site in satellite image.



Fig. 1: Rhiss Dam site on satellite map

The dam site is mainly located in:

- The marly-limestone and marly flysch with limestone banks of Beni-Bou Yacoub (Cretaceous upper - medium) [11] (Ressources en eau, J-P Thauvin, 1971);
- The gray-black sericite shales with sandstone-quarzitic beds of Kétama (Albian-Aptian) [11] (Ressources en eau, J-P Thauvin, 1971).

Further investigations made it clear that the narrowing of the valley at the site is mainly due to the presence of quarzitic layers within the fine schists drawing an anticline whose borders are composed of

marls and marl-limestones from the Upper–Middle Cretaceous. The quartzite bars and schists are arranged in a direction sub-transverse to the axis of the valley (N90 to N105) and dip regularly downstream from 40° to 45°.

Core drillings have shown that the bed of the wadi is filled to a depth of 17 to 35 meters by coarse alluvium with low silty-gravelly terraces at the edge of the banks. Fig. 2 shows the geological section of the axis of the Dam.

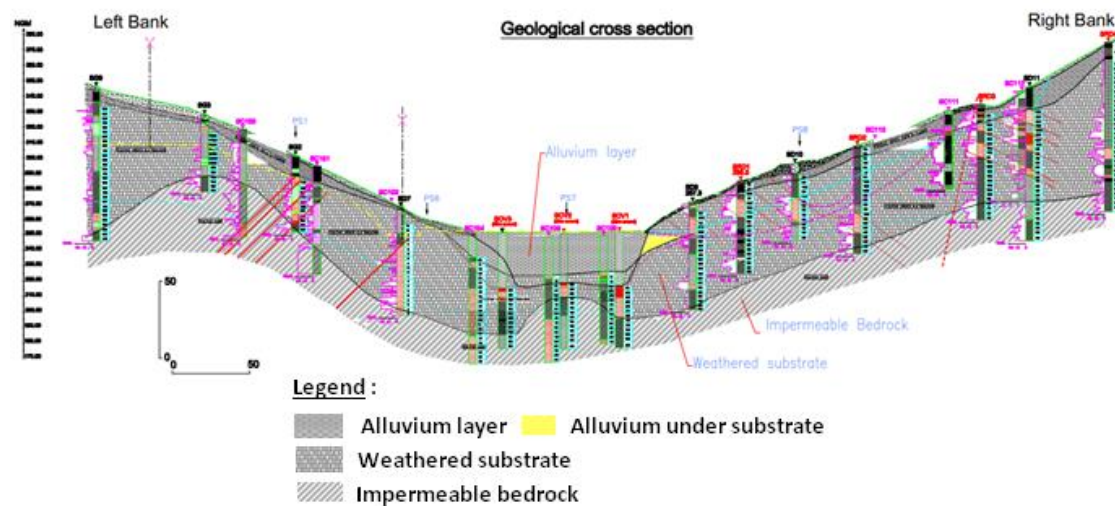


Fig. 2: Geological section at the dam site

2.2 Design of the Dam

The dam subjected to our study is a CFRD Dam (Concrete Faced Rockfill Dam). It has been designed according to the recommendations of International Commission On Large Dams (ICOLD), and CFBR (Comité Français des Barrages et Retenues). The typical section of the dike includes from upstream to downstream:

- The upstream concrete mask, connecting at the upstream foot of the dam with the plinth and at the crest to the sole of the parapet wall.
- A layer of alluvium clipped to 50 mm between the upstream mask and the alluvium of the dike.
- Natural alluvium of the wadi.
- A chimney drain with a thickness of 5.00 m, with a carpet constituting the downstream outlet.
- Small protective riprap with a width of 1.00 m, placed across the entire downstream face of the dam.
- Riprap to protect the downstream foot of the dike, placed up to the level 256.00 NGM.

These design elements have been done according to the indications given by CIGB-ICOLD ([9] (CIGB Bulletin 141, 2010), [10] (CIGB-ICOLD, Bulletin 51, 1985), [12] (CIGB-ICOLD, Bulletin 150, 2018) and [13] (CFBR, 2015)).

2.3 The Diaphragm Wall (DM)

The design of the diaphragm wall adopted at the stage of the execution studies to ensure the foundation of the plinth at the bottom of the valley, as well as the watertight break, is a plastic concrete wall over the entire height of the alluvial filling (35m). The diaphragm wall is topped with a head mass which acts as a foundation for the plinth.

It is 1.20 m thick and anchored 1 m (at least) in the bedrock, with a 6 m high plastic concrete mass at the top, located directly under the plinth. This choice makes it possible to mobilize the foundation stop and thus reduce the forces applied to the diaphragm wall.

The diaphragm wall has been executed by the method of alternating primary and secondary studs of approximately 7 m in width each, with a stair-step base ensuring a minimum anchorage of 1 m in the bedrock.

In addition, to ensure waterproofing at the joints between studs, there are two solutions:

- Either adopt side formwork equipped with water-stop joints between blocks.
- Or by nesting the studs together: the secondary studs bite into the primary studs over a certain width of around 1 m.

The excavation and concreting of the head mass is carried out according to the same principle as for the diaphragm wall itself, that is to say: excavation under bentonite-based grout to hold the walls of the trench, then concreting by dip tubes and simultaneous pumping of the bentonite mud.

In order to overcome possible construction difficulties due to the relatively large thickness of the head mass, it can be produced using the same method of alternating panels as the diaphragm wall, although with smaller widths.

The concrete head block is protected upstream by a fine (filtering) material, which is coated by a layer of clayey silts, then a layer of coarse alluvium, and then a layer of alluvium. Fig. 3 gives a Cross-Section of the Diaphragm Wall

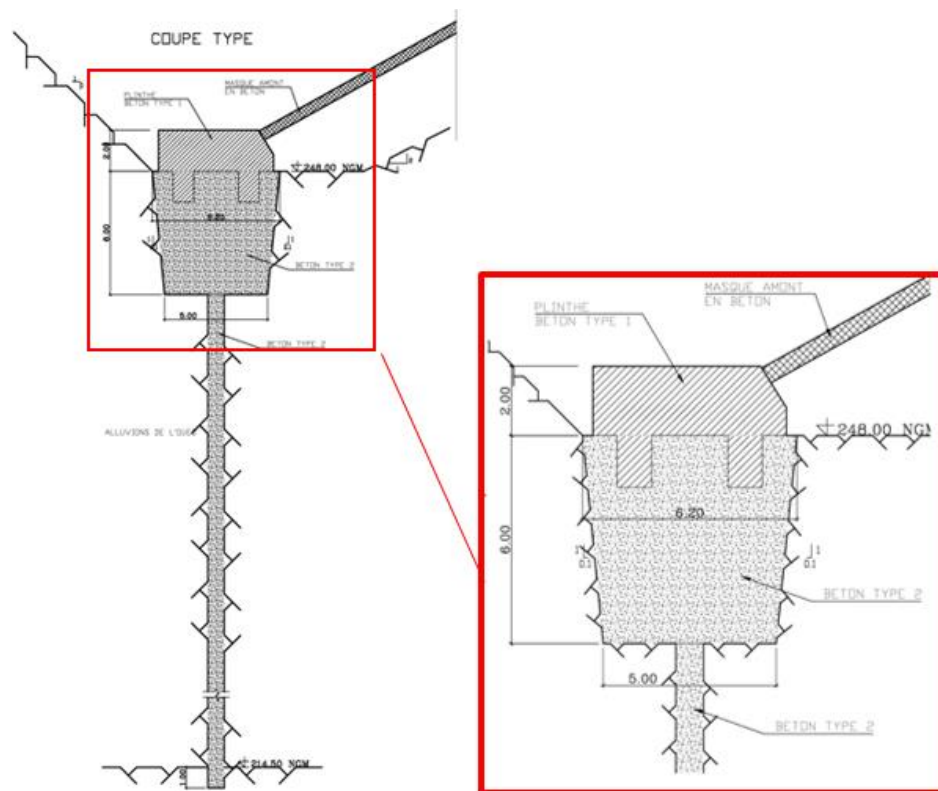


Fig. 3: Typical section of the diaphragm wall

2.4 Numerical analysis

In order to assess the capability of the diaphragm wall to undergo the constraints it is subjected to, we have used numerical analysis based on finite elements methods.

The following loads have been considered:

- Water load:

The water in the reservoir exerts a hydrostatic pressure equal to:

$$P_w = W \cdot H \quad (1)$$

where:

P_w is the hydrostatic pressure,

W is the volume mass of water (1t/m^3),

H is hydraulic load.

Downstream of the diaphragm wall, the hydraulic load depends on the permeability of the different horizons and the anchoring of the watertight break in the bedrock. It is calculated automatically in the model.

It is almost constant between the downstream of the diaphragm wall and the downstream toe of the dam, and equal to the water level downstream of the dam, because of almost total head losses in the bedrock.

• Ground push and stop: the stresses applied by the ground on the diaphragm wall are calculated automatically based on the deformation and the limit resistance of the materials. Thus, the thrust and Stop forces depend on the following parameters:

- 1) Ground deformation moduli.
- 2) Cohesion and friction angle of the soil.
- 3) Inertia of the wall.

• Earthquake: the diaphragm wall has been verified under the action of the project earthquake defined in the seismic hazard assessment note, established within the framework of the pre-studies.

The calculation of the seismic forces applied to the diaphragm wall is carried out using the pseudo-static method according the equation:

$$F = \alpha \cdot m \cdot a g \quad (2)$$

where:

F is the seismic force,

α is the Pseudo-static coefficient taken equal to 2/3,

m is the mass of the element.

ag is the seismic intensity, equal 0.367g (SMS) during the operation phase and 0.15g during the construction phase (SP).

Hence, the pseudo static magnitude for SP and SMS are given in table 1.

Table 1: Horizontal pseudo static magnitude (a_h) for SP and SMS.

Pseudo static magnitude	SP [g]	SMS [g]
a_h	0.1	0.245

Thus, the resulting pseudo static acceleration, due to its scalar character making it possible to define a direction of propagation of the seismic waves, is introduced into the model in two phases: one where it is taken positive (SP+ and SMS+) and the other, where it is negative (SP- and SMS-).

The base of the plinth is assumed to be horizontal. Calculations are carried out for the unit width ($L = 1\text{m}$). The calculation assumptions are presented below:

- The modeled section corresponds to the maximum height of the dam (85m), and the maximum depth of the alluvium of the wadi (35m);
- Soil and plastic concrete have perfect elastoplastic behaviour (Mohr Coulomb Model);
- The mask and the plinth are modeled as elastic materials (Elastic Linear Model);

The materials used in the model are in Table 2, while Fig. 4 shows the model.

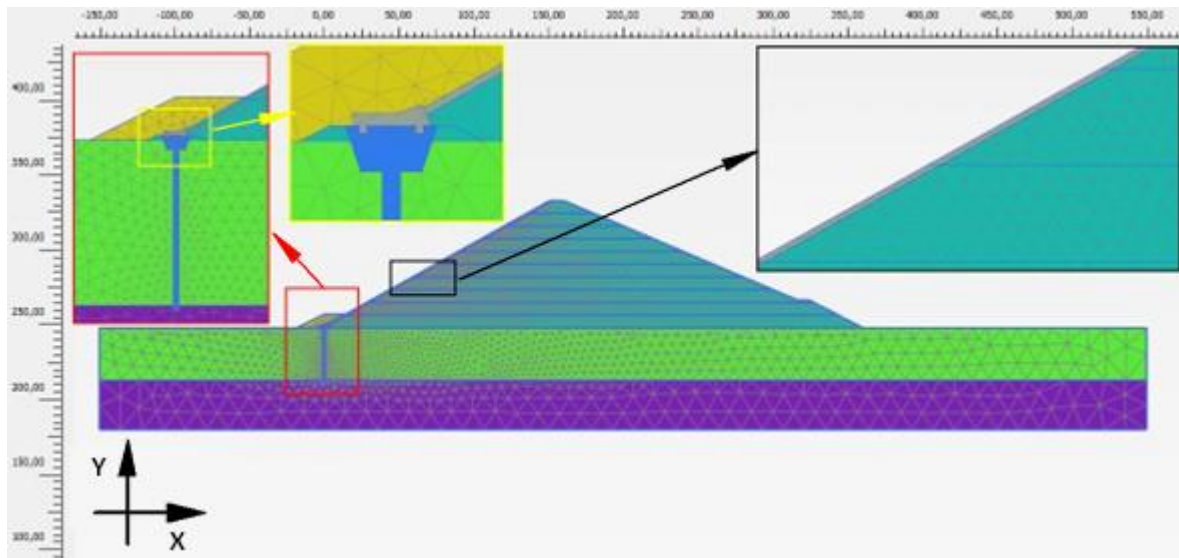


Fig. 4: Cross section of the model (colored rectangles show a zoom of the area as indicated by the rows)

Table 2: Materials used in the model and their parameters.

Materials	γ_{sec} [KN/m ³]	n [%]	γ_{sat} [KN/m ³]	K [m/s]	E [KN/m ²]	C [KN/m ²]	Φ [°]	ν
Alluvium	21.0	10.00	22.0	4.00E-05	2.00E+04	0.25*	39	0.25
Dike (3A)	23.2	10.00	24.0	1.00E-04	1.00E+05	0.25*	39	0.3
Stringers	17.1	1.00	17.2	1.00E-08	2.60E+04	50	26	0.19
Bedrock	21.0	1.00	21.1	1.00E-09	3.00E+05	70	37	0.3
Diaphragm wall	23.2	1.00	23.3	1.00E-08	4.00E+05	145	32	0.3
Plinth	24.5	0.10	24.51	1.00E-11	2.20E+07	-	-	0.2
Concrete face	24.5	0.10	24.51	1.00E-11	2.20E+07	-	-	0.2

The load cases considered assume the construction of the diaphragm wall before the construction of the dam embankment. Taking into account the phasing presented above, displacements, stresses and strains are calculated by considering the following cases:

- Case 1: "Construction of the diaphragm wall and the plinth"

The earth pressures and the hydrostatic pressures are balanced on both sides (Upstream-downstream direction) of the diaphragm wall. The diaphragm wall is thus subjected to its own weight, the weight of the plinth, and friction with the alluvium.

- Case 2: "Case 1 + completion of the dam and the mask in 10 layers"

The installation of the embankments for the construction of the dike is done gradually until reaching the crest of the dam.

The thrust of the earth due to the weight of the dam downstream of the diaphragm wall induces a movement of the diaphragm wall upstream, in addition to settlement in the alluvial foundation.

- Case 3: "Case 2 + Installation of the upstream toe embankment"

The placement of stringers at the upstream foot of the dam on the head of the diaphragm wall and the plinth constitutes an additional load for the diaphragm wall, causing settlement of it and the foundation.

- Cases 4 and 5: "Case 3 + Construction site earthquake"

This is the case of the empty reservoir under construction earthquake (corresponding to the advent of an earthquake during the end of construction of the dam before its commissioning).

Two calculations are carried out:

- 1) Case 4 (SP+): where the acceleration of the construction site earthquake directed upstream.

2) Case 5 (SP-): where the acceleration of the construction site earthquake directed downstream.

- Case 6: “Case 3 + Impoundment of the dam up to the level RN 325.00 NGM”

To the forces due to load case 3, are added the hydrostatic weight of the retainer above the plinth and the hydrostatic thrust resulting from the difference in the hydraulic load between the upstream and downstream of the diaphragm wall.

- Cases 7 and 8: “Case 6 + Operating earthquake”

In this case, we consider that the reservoir is full (corresponding to the occurrence of the design earthquake during the operation phase of the dam).

The calculations carried out are as follows:

- 1) Case 7 (SMS+) where the acceleration of the maximum safety earthquake directed upstream.
- 2) Case 8 (SMS-) where the acceleration of the maximum safety earthquake directed downstream.

In accordance with the phasing given above, calculations have been made using Plaxis. [8] (Razvan ignat, Sadek Baker, Stefan Larsson, Sven Liedberg, 2015) has used Plaxis to analyse excavations supported with a diaphragm wall formed with rows of dry deep mixing columns.

3 Results and discussion

3.1 Results

3.1.1 Deformations

The maximum displacements and deformations in the diaphragm wall are summarized in table.3 below.

Table 3: Calculation results -Deformations-

Load case	Corresponding Phase	Ux [cm]	Uy [cm]	Px [cm]	Py [cm]	U [cm]	γ_{xy} [%]
1	Diaphragm Wall and plinth	0.0137	-1.151	-	-	-	0.03
2	Dike and Mask	-52.85	-13.97	-	-	53.53	-12.04
3	Foot Embankment	-51.64	-20.47	-	-	54.11	-12.00
4	SP+	-155.9	-20.13	-	-	156.4	-23.13
5	SP-	-	-24.42	46.97	-5.295	24.62	-11.8
6	Impoundment	-	-112.7	96.75	-94.26	121.2	-12.4
7	SMS+	-	-143.7	-156.1	-32.16	187.2	*-25.03
8	SMS-	-	-85.72	118	26.59	189.4	*52.53

* see §3.2.2, Fig. 8, and Fig. 9.

Ux is the horizontal total displacement, Uy is the vertical total displacement, Px is the horizontal phase displacement, Py is the vertical phase displacement, U is the total displacement and γ_{xy} is the distortion.

3.1.2 Constraints

The maximum stresses in the diaphragm wall are summarized in table 4 below.

Table 4: Maximum constraints in the diaphragm wall

Load case	Corresponding Phase	τ_{max} [KPa]	$\sigma'1$ [KPa]	$\sigma'2$ [KPa]	$\sigma'3$ [KPa]
1	Diaphragm wall and plinth	317	542	234	194
2	Dike and Mask	749	1849	721	513
3	Foot Embankment	754	1869	728	513
4	SP+	755	1965	731	550
5	SP-	697	1708	881	714
6	Impoundment	1207	3246	1234	891
7	SMS+	1211	3266	1238	980
8	SMS-	1158	2887	1725	1226

3.2 Discussion

3.2.1 Displacements

Given the high deformability of the alluvial foundation ($E=20\text{Mpa}$), the diaphragm wall withstands large deformations throughout the construction of the dam and in extreme cases of exploitation and earthquakes:

1) Load cases relating to construction (construction phases and SP project earthquake).

During the construction of the dam until the installation of the upstream foot embankment (phase 1 to 3), the diaphragm wall moves upstream, absorbing the thrust due to the horizontal deformation concomitant to the settlement of the alluvial foundation. At the end of the construction of the dike and the upstream concrete face, this deformation reached almost 53cm upstream. We note a slight regression after the installation of the silts at the upstream foot of the dam. This could be explained by the fact that by settling under the additional weight of the stringers directly placed on the head mass crowned by the plinth, the head of the wall (which is site of the greatest displacements at this stage) records a slight movement downstream, of a few millimeters.

In the case of the occurrence of an earthquake of intensity 0.1g (SP+), we witness a surge in the upstream movement of the diaphragm wall. The recorded horizontal displacement then triples and reaches more than 155cm upstream.

Furthermore, the diaphragm wall experiences settlement throughout the project phases, the importance of which remains less pronounced than the horizontal movement. Indeed, until the end of the installation of the embankment and the upstream mask, the settlement is of the order of 14cm. When the stringers are installed, this value increases to a little more than 20cm.

When the SP+ earthquake passes, it is noted a regression of a few mm. On the other hand, the seismic case SP- generates an additional settlement of around 4cm (i.e., 24cm of settlement calculated at the end of this phase). Considering the fact that this seismic case causes a movement in the downstream direction unlike the rise of the embankment which pushes it towards the upstream, the movement commented on concerns the increment of displacement due to the phase. Thus, when the SP- earthquake passed, we noted a movement of the diaphragm wall downstream by approximately 47cm. At the same time, the wall absorbs a vertical movement of 5cm downwards. This movement is mainly absorbed by the downstream part of the head massif.

2) Load cases relating to operation (watering and operating earthquakes).

When impounded, the settlement of the diaphragm wall increases, reaching 112cm (i.e., 94cm induced by impoundment). This settlement is consumed in the body of the diaphragm wall, which is deviated from its axis downstream. Also, when water is filled, we see an accentuation of the horizontal movement of the diaphragm wall. In fact, the hydraulic gradient between the upstream under the load of the reservoir and the almost drained downstream causes the wall to move widely downstream, thus exceeding its original axis. The calculated value is approximately 96 cm downstream.

In the event of an SMS+ earthquake occurring during operation, the head of the wall is projected far upstream, and the movements become excessive. Indeed, this phase induces 156cm of displacement upstream as the earthquake passes. At the same time the wall is sunk by approximately 32cm.

The greatest elongation of the diaphragm wall is observed during the passage of the SMS- earthquake which is associated with the impoundment. In fact, the wall is driven further downstream by 118cm. This corresponds to a movement of the diaphragm wall of 176cm relative to its original axis. This large movement causes the diaphragm wall to be torn from its anchoring (which explains why the settlement has decreased by approximately 26cm (see Fig. 5 and Table 2)).

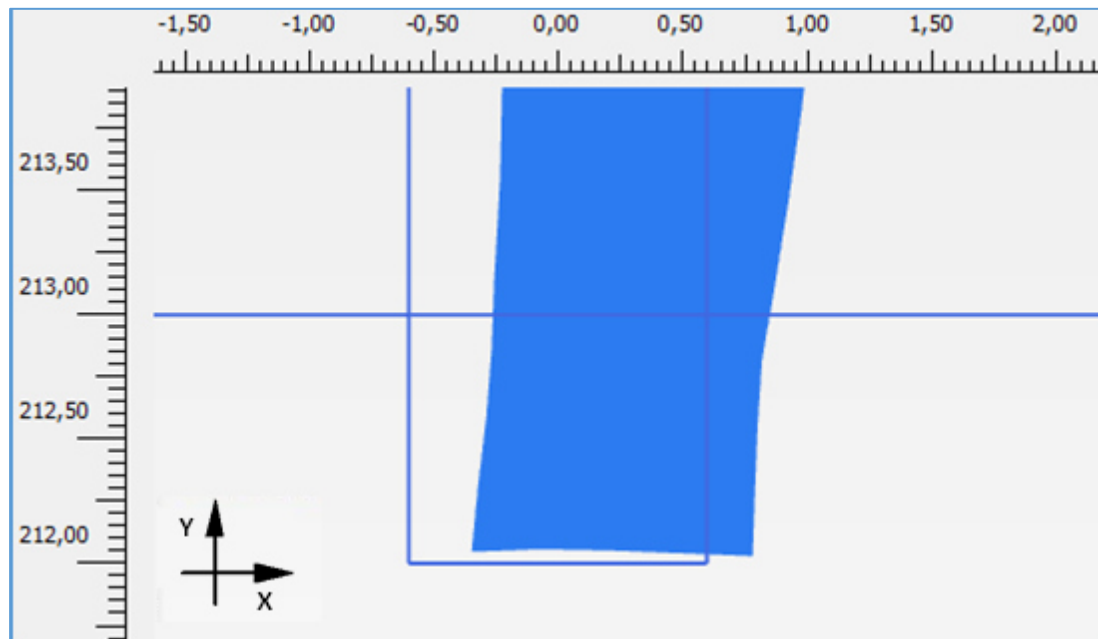


Fig. 5: The base of the diaphragm wall torn from its anchoring.

3.2.2 Strains

Displacements of the diaphragm wall are inevitably accompanied by distortions (strains) which are accentuated as the wall deforms. Indeed, the greater the displacements observed in the wall, the greater the calculated distortion. On the other hand, we note a preferential accentuation of the distortion in critical places on the wall, particularly the anchoring zones, which visibly absorb the strongest angular deformations.

1) Load cases relating to construction (construction phases and project earthquake)

Quantitatively, the observed and significant values range from 12 to 23%. More precisely, until the end of the construction phase, the maximum distortion calculated is 12.04%, observed at the end of the placement of the backfill and the mask. Indeed, as the wall is pushed upstream by the foundation which settles under the weight of the backfill, the mass of the head mass opposes a greater counterthrust than the body of the wall, so that a difference in displacement is created between the head mass and the rest of the wall. Thus, the junction between these two entities absorbs a significant traction, which is dissipated by distortion (see Fig. 6). Thus, this junction zone, which deserves special inspection, is the site of the greatest deformations that can affect the diaphragm wall.

Still in the project phase, the advent of an earthquake could prove to be aggravating for the diaphragm wall, depending on the direction of propagation of the waves. Indeed, as we can see in the results table (table 2), the distortion could reach 23% cumulative if tremors happened, of course depending on their intensity, to accentuate the movements of the dam mass and by extension of the diaphragm wall upstream (SP+). Indeed, for an SP+ earthquake, the distortion reached 23% (Fig. 6).

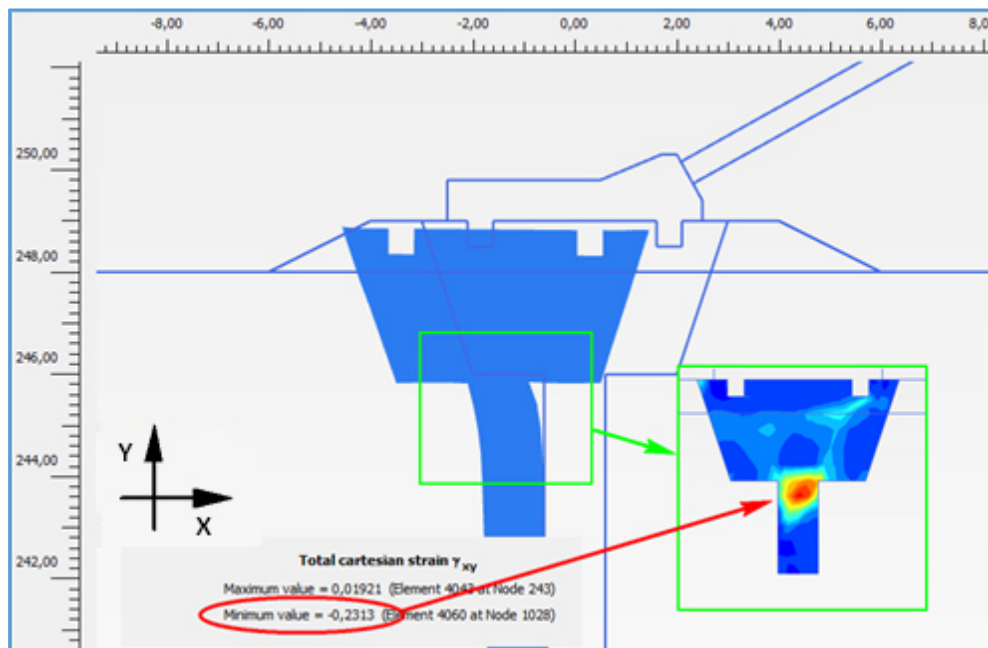


Fig. 6: Location of the maximum total distortion

On the other hand, the same earthquake in the opposite direction gives 11% deformation. However, it is important to note that this value is biased: at first glance, we go from 12% to 11%, which would seem advantageous; except that this regression is due to the fact that the massif is deformed in the opposite direction. Given the scalar nature of the distortion, the final value is then smaller. It is therefore important to take into consideration the cyclical aspect of the distortion.

We therefore carried out an in-depth analysis of the critical zone, observing the distortion specific to this seismic passage. We note (see Fig. 7) that the maximum incremental distortion to the right of the junction is +0.32% (the sign of this distortion is significant, testifying to the reversal of the thrust and therefore of the displacements). By adding this scalar value of 0.32% to the pre-earthquake value -12% we obtain -11.68% which is close to the -11.80% shown in table 2.

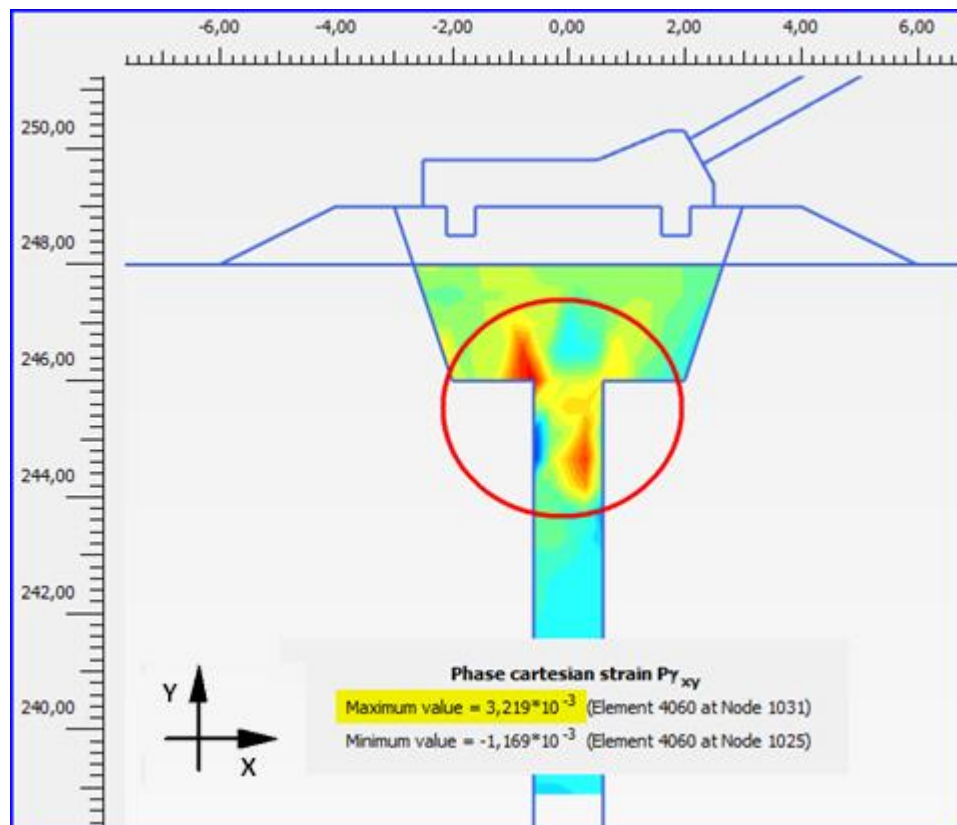


Fig. 7: Maximum incremental distortion located at the junction point

2) Load cases relating to operation (watering and operating earthquakes)

When the dam is filled with water, the entire mass as well as the diaphragm wall are propelled downstream under the load of the reservoir. A final maximum distortion of 12.40% was noted, always located at the junction with the head mass.

In the case of an SMS+ type earthquake in the exploitation phase, which in particular pulls the massif in the same direction as the impoundment, we note an accentuation of the deformation, which reaches 12.49% at the right of the junction with the head mass.

It is important to note that the greatest deformation is observed at the upstream corner of the massif, and we assume it to be a modeling peak. This is a large value of 25% which is not representative. Indeed, if it were to arrive or if a rupture occurred at this location, no vital prognosis of the wall is involved (see illustration on Fig. 8).

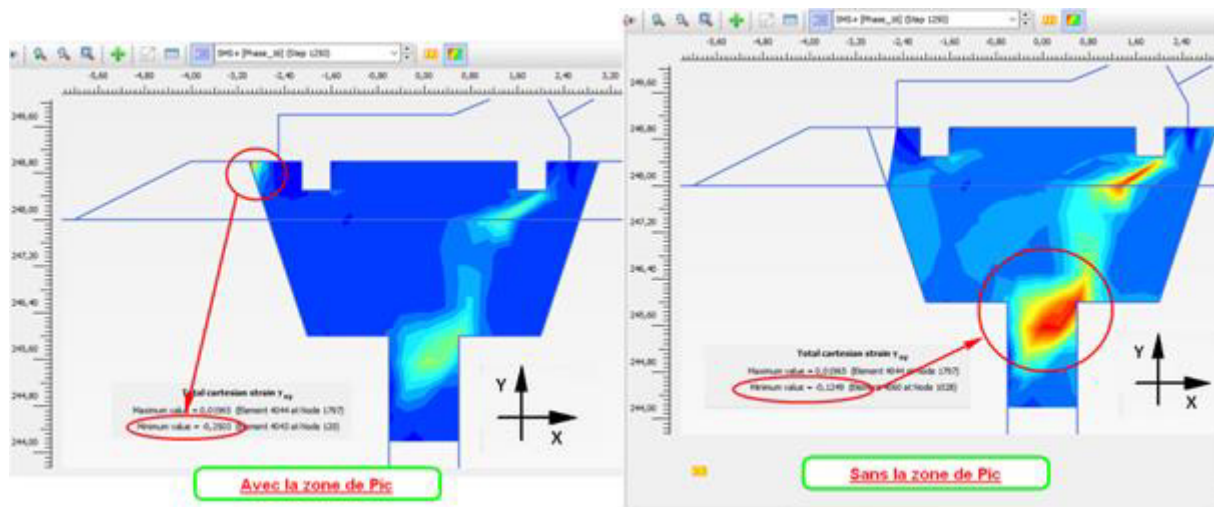


Fig. 8: Concentration points of distortion at the corner of the diaphragm wall (SMS+ case)

In the case of an SMS-, the same phenomenon is observed. We obtain more than 50% deformation at the upstream end of the massif. However, two critical values should be noted: 12.3% at the junction with the head mass and 16% at the anchorage with the bedrock (see Fig. 9). The explanations given above remain valid for this scenario.

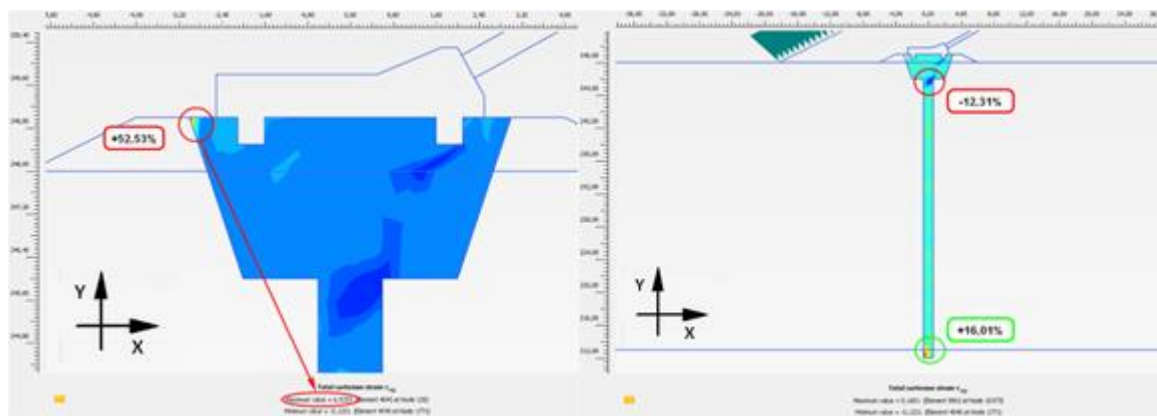


Fig. 9: Concentration points of distortion at the corner of the diaphragm wall (SMS- case)

3.2.3 Shear stress (τ_{max})

1) Load cases relating to construction (construction phases and project earthquake)

The shear stress calculated in the wall varies from 300Kpa to 750Kpa during the construction phase. Indeed, at the end of the different construction phases, the analysis of the state of stress given by the algorithm shows that the shear hardly reaches 800Kpa. As soon as the diaphragm wall and the plinth are in place, the stress τ_{max} reaches 317Kpa, a value which evolves during the rise of the embankment to stabilize at 754Kpa at the end of construction marked by the installation of the upstream foot embankment.

At the advent of an SP+ earthquake, a new stress state is established, with a τ_{max} value of 755Kpa. For the SP- earthquake, the maximum value of τ_{max} is 697Kpa.

2) Load cases relating to operation (watering and operating earthquakes)

When the dam is impounded, the stresses increase and the new state obtained at the end of the impoundment stabilizes at $\tau_{max} = 1207Kpa$.

In the seismic phase, we observe a variation in these values, meanwhile 1211Kpa in SMS+ and 1158Kpa in SMS-.

Anyway, the value of τ_{max} must be interpreted with the state of stress of the diaphragm wall, in particular the effective stresses in compression and confinement (σ'_1 , σ'_2 and σ'_3). Indeed, the constraint τ_{max} is a function of the constraints σ'_1 and σ'_3 .

An analysis of the Mohr circle resulting from the different situations shows that certain zones of the wall reach the envelope of the intrinsic curve of the diaphragm wall resulting from the triaxial tests, although the shear stress is low, unlike certain zones far of the plasticity limit despite a greater τ_{max} (see Fig. 10).

However, in general it is observed by traversing the body of the diaphragm wall that the intrinsic crown of the wall is generally reached. This is interpreted as a reaching of the plasticity limit and thus the deformations obtained are irreversible.

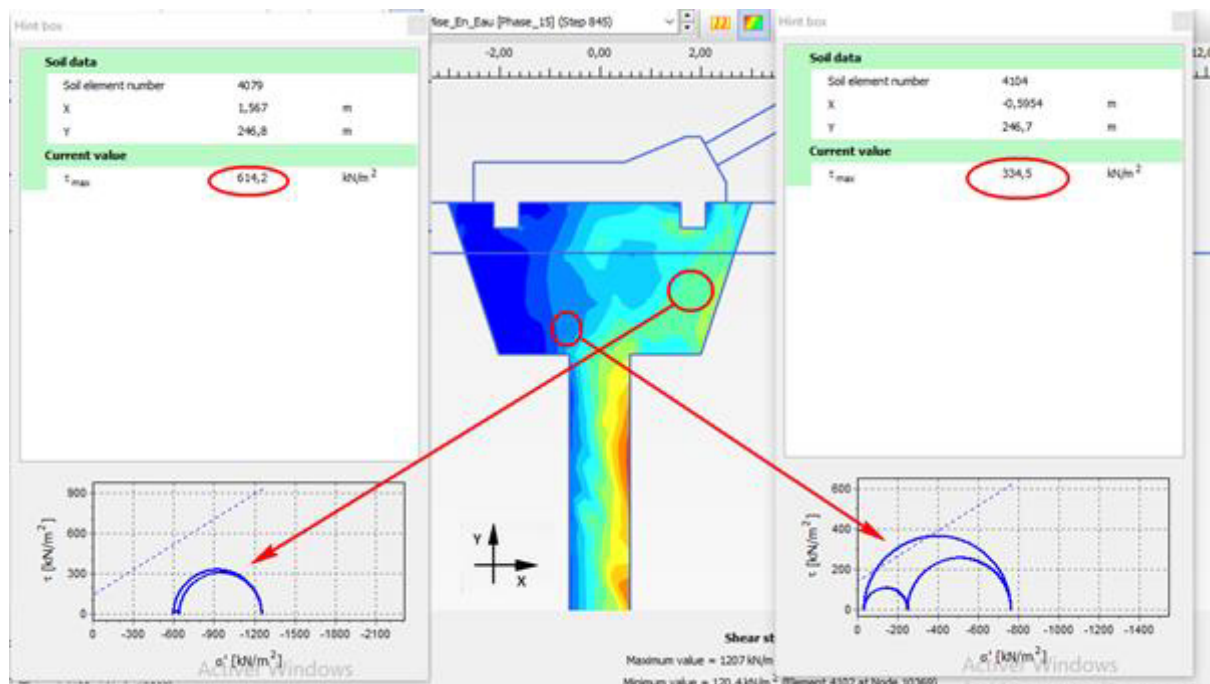


Fig. 10: Mohr Coulomb's circle and τ_{max} values

Conclusion

Based on numerical simulation, the behaviour of a diaphragm wall set upstream a high dam, and crossing a thick layer of alluvium, has been studied in this paper. It comes out from that study that the diaphragm wall withstands the earth pressure in two ways: by deforming accordingly to the alluviums and opposing a stop to the alluviums' thrust. Indeed, as the dam's embankment gets high, the diaphragm wall displaces upstream, due to the thrust of the alluvium layer settling downstream the diaphragm wall. The maximum upstream displacement is observed at the end of the embankment's construction.

In the other hand, when the dam is impounded, given the water pressure on the concrete face, the whole dam is deformed, and a general downstream displacement occurs. Hence, the diaphragm wall displaces downstream. The same behaviour is noticed in seismic charge cases: the diaphragm wall and the dam move accordingly to the seismic waves' orientation.

These displacements are made possible due to the plastic property of the diaphragm wall. Nevertheless, it comes out that some areas of the diaphragm wall withstand more distortions, due to the difference of displacements between the three parts of the diaphragm wall (the anchoring, the body, and the head mass). Indeed, distortions concentrate at the anchoring point in the bedrock and the junction between the body and the head mass. That state deserves a particular attention when designing a plastic diaphragm wall in similar conditions and use.

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