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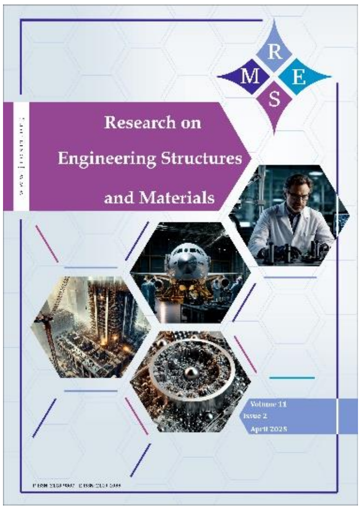

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Research Article

Stabilization of excavation using the diaphragm wall method in Karbala

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Abstract

Construction of historical and religious projects around the world is often highly sensitive, particularly when they involve sites of great spiritual significance. The Shrine of Imam Hussain (AS) in Karbala is a prime example as one of the most important centers for Muslims. Due to its religious importance, any development plan for this site requires exceptional care and precision. This article examines the excavation stabilization method used in the shrine's development project. After evaluating various techniques, the diaphragm wall method was chosen and implemented to stabilize the excavation pit. PLAXIS 2D and SAP200 software are used for modeling, it was demonstrated that this method effectively controls pit crown deformations, keeping them within an acceptable limit of 4 cm. Additionally, the diaphragm wall provides excellent water sealing, preventing seepage from behind the pit making it a highly reliable and advantageous solution.

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1. Introduction

Rehabilitation and reconstruction of historical and religious projects is always highly sensitive, especially when they involve sites of deep spiritual significance. The Shrine of Imam Hussain (AS) in Karbala, one of the most revered places for Muslims, is no exception. As part of the shrine's development plan, a 15-meter-deep excavation and its stabilization became a necessary undertaking. Globally, the geotechnical engineering community employs various excavation stabilization methods, each selected based on the specific geotechnical conditions of the site, along with economic, technical, and practical considerations. One of the most critical concerns in projects with a high groundwater table is ensuring effective excavation waterproofing. Among the methods available, the diaphragm wall technique stands out as one of the most efficient. It not only controls lateral displacement of excavation walls but also significantly reduces groundwater seepage.

Extensive research both numerical and experimental has been conducted on diaphragm wall performance. For instance, Bolton and Pourier [1] used centrifuge modeling to evaluate the performance of diaphragm walls and demonstrated their long-term effectiveness in excavation stabilization. Similarly, Pakbaz et al. [2] studied the diaphragm wall performance in five metro stations in Ahvaz. Their findings showed that finite element models predicted wall displacements in the range of 0.005 to 0.007 times the excavation depth, while actual observed displacements were even lower. They also noted that nearby structural settlements ranged from 0.0025 to 0.0035 times the excavation depth. In another case study, Jasmine and Muttharam [3] analyzed a 14-meter-deep excavation in Noida, India. Using finite element modeling and field monitoring with

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inclinometers, they observed that actual displacements were approximately 67% of those predicted by the models.

Numerous previous studies have investigated the stabilization of various soil types using a wide range of materials, including chemical additives, geopolymers, nanomaterials, magnetized water, and biotreatment methods. These stabilizing agents can be applied through soil mixing, injection, or deep mixing techniques. The choice of material and method largely depends on factors such as the type of soil, available treatment time, project budget, loading conditions, and the integration of sustainability principles. The effectiveness of the soil treatment is typically evaluated based on improvements in settlement behavior and bearing capacity [4-10]. In the present study, numerical modeling has been used to investigate the stabilization of the 15-meter excavation for the development of the Shrine of Imam Hussain (AS) in Karbala. A T-shaped diaphragm wall was designed for this purpose. This design eliminates the need for temporary supporting elements such as anchors, struts, or braces an especially important consideration for this project. Given the site's challenging conditions, including loose, collapsible soils and a high groundwater level, the T-shaped diaphragm wall offers a technically and operationally sound solution, overcoming many of the difficulties associated with traditional stabilization methods.

2. Study Area and Geotechnical Description

The arial photo, the project site plan and the condition of the surrounding buildings are presented in Figure 1. This picture shows the status of the project's neighbors. Based on the results of geotechnical studies and exploratory boreholes, the log of the obtained boreholes is presented in Figure 2. In general, except for above 8 meters, the dominant texture of the soil is sandy. At a depth of 37 to 50 meters, there is a continuous layer of clay, which has been found suitable for waterproofing and controlling seepage. Based on the presented results, layering and values of parameters measured by tests and also considering engineering judgment, the geotechnical profile was finalized according to Table 1. These values are used as the basis for calculations and modeling. According to the information of the site, in the modeling and design, from the depth of 50 onwards, dense sand layer similar to L8 layer is considered.

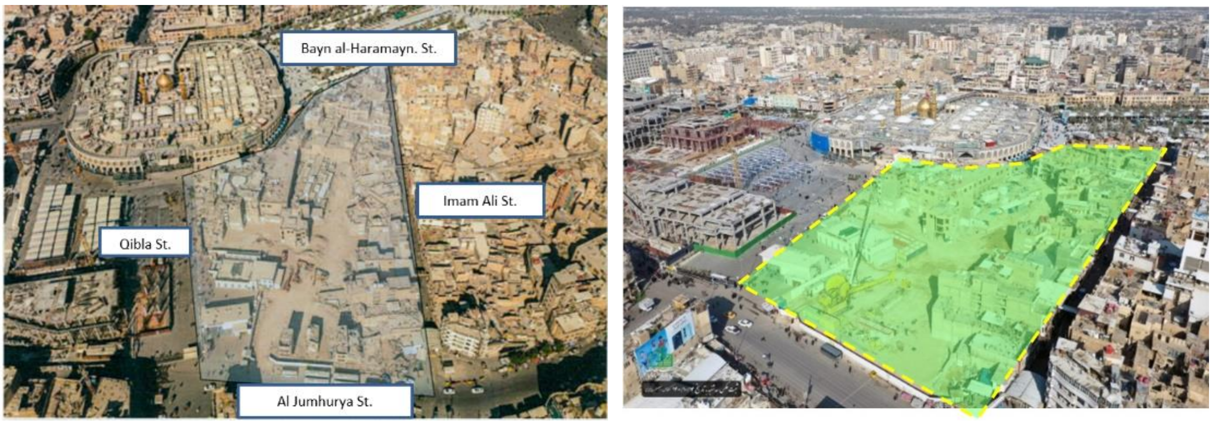


Fig. 1. Site location and boundary

During drilling operations, all formations' high-pressure fluids and gases of the earth are controlled by borehole pressure, which consists of hydrostatic pressure of drilling mud, pump pressure, and friction pressure loss in the annulus. If, for any reason, the borehole pressure falls below the formation fluid/gas pressure, the formation fluids/gases will enter the hole and a pressure "kick" will occur. If a kick cannot be controlled properly, uncontrolled formation fluids/gases will reach to surface where the drilling rig is located. Such a catastrophic event is known as blowout [1]. To prevent formation fluids/gases to reach the surface of the well, blowout preventers are used as safety valves. When they are activated, they are supposed to close off the wellbore and seal it (in some cases, the sealing pressures are 20,000 Psi which is 1360 bar) in an emergency to control and balanced formation fluids and gases [2,3].

In a blowout preventer stack, two types of blowout preventers are used; annular and ram. Annular BOPs are used in combination with hydraulic system that can seal off different sizes of annulus whether drill pipe is in use in the wellbore or not. Upon command, high-pressure fluid is directed to the closing hydraulic ports positioned in the lower side of the piston. This causes the operating piston to move upward; therefore, the moving piston compresses the packer [2-43]. Because of a cap at the top of annular blowout preventer, the packer can only move toward the center of the wellbore to pack off a drill pipe or seal off the wellbore.

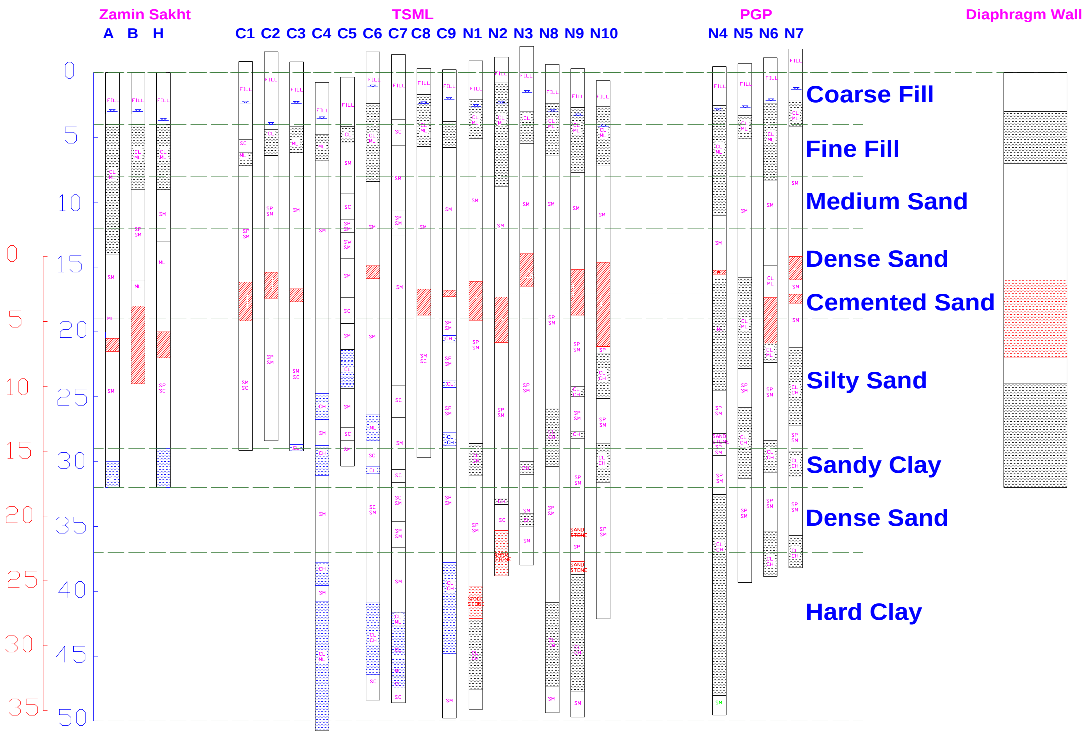


Fig. 2. Borehole log and soil profile

Table 1. Soil layers parameters

Definition	Parameter	L1 0 -4	L2 -4 -8	L3 -8 -12	L4 -12 -17	L5 -17 -19	L6 -19 -29	L7 -29 -32	L8 -32 -37	L9 -37 -50	Unit
Saturated unit weight	γ_{sat}	17.0	17.0	18.0	18.5	19.0	19.0	19.5	19.5	19.5	kN/m³
Unsaturated unit weight	γ_{un}	17.5	19.5	19.5	18.0	18.0	18.0	16.0	16.0	16.0	kN/m³
Effective cohesion	c	2.0	12.0	7.0	7.0	50.0	10.0	25.0	10.0	35.0	kPa
Effective angle of internal friction	φ	25.0	20.0	30.0	34.5	38.5	36.0	27.5	36.0	25.0	°
Reference secant stiffness in standard drained triaxial test	E_{50}^{ref}	10	12	50	135	135	100	70	150	90	MPa
Reference tangent stiffness for primary oedometer loading	E_{oed}^{ref}	10	12	50	135	135	100	70	150	90	MPa
Unloading/reloading stiffness	E_{ur}^{ref}	30	36	150	405	405	300	210	450	270	MPa
Reference stress for stiffness	p_{ref}	20	56	59	68	71	101	166	143	253	kPa
Reference shear modulus at very small strains ($\varepsilon<10^{-6}$)	G_0^{ref}	120	40	110	90	110	100	80	80	80	MPa
Power of stiffness formula	m	0.5	0.5	0.5	0.5	0.5	0.5	1.0	1.0	1.0	-
Poisson's ratio for unloading-reloading	υ_{ur}	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	-
K_0-value for normal consolidation (default $K_0^{nc}=1-\sin\varphi$)	K_0^{nc}	0.58	0.66	0.50	0.43	0.38	0.41	0.54	0.41	0.58	-
Hydraulic Conductivity	K	8.64	8.64	8.64	8.64	0.864	8.64	8.64	8.64	0.009	m/day
Over consolidation Ratio	OCR	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-

The limitations of current study can be summarized by the following points:

- The numerical analyses were performed using two-dimensional plane strain assumptions, which idealize the excavation and diaphragm wall as longitudinally uniform. While this approach is appropriate for evaluating global behavior along representative sections, it does not capture three-dimensional effects such as corner stiffness, spatial variability of soil properties, or interactions with nearby irregular structures. These effects may locally influence deformation and stress distribution.

- The soil stratigraphy and material properties were derived from a limited number of site investigation data points and were assumed to be laterally homogeneous. Natural soils may exhibit heterogeneity, anisotropy, and spatial variability that are not fully represented in the model, potentially affecting displacement and seepage predictions.
- The study relied primarily on numerical modeling results for displacement and seepage assessment. At the time of the analysis, comprehensive field monitoring data (e.g., inclinometers, piezometers, or pumping tests) were not available for direct validation. Consequently, the reported deformation (approximately 4 cm) and seepage quantities should be interpreted as design-based predictions rather than measured values.
- Simplified constitutive soil models and idealized soil–structure interface parameters were adopted to ensure numerical stability and practicality. These simplifications may influence the accuracy of stress–strain response, particularly under complex loading or long-term conditions.

As a result of these limitations, the findings of this study should be considered site-specific and primarily applicable to deep excavations in similar geological and hydrogeological conditions. The results are intended to demonstrate the feasibility and effectiveness of the proposed diaphragm wall system rather than to serve as universal design values. Future work incorporating three-dimensional modeling, enhanced site characterization, and field monitoring data would further improve the robustness and general applicability of the conclusions. The predicted maximum lateral deformation of the diaphragm wall, obtained from PLAXIS 2D analyses, is approximately 4 cm at the crown for the final excavation stage. According to standard design guidelines for deep excavations in cohesionless and partially saturated soils, the permissible lateral deformation for similar diaphragm walls is typically in the range of H/200 to H/150, where H is the wall height. For the 15 m-deep walls in this study, this corresponds to approximately 7.5–10 cm. Thus, the margin of safety between the predicted and permissible deformations is roughly 3.5–6 cm, indicating that the diaphragm wall is expected to perform well within acceptable deformation limits under the design conditions.

3. Seismic Analysis and Excavation Stabilization

To determine the seismic acceleration for analysis, it is important to consider that Karbala is situated in a low-risk seismic zone with very low earthquake intensity. Based on this, the horizontal acceleration coefficient (k_h) for quasi-static analysis was calculated using a peak ground acceleration (PGA) of 0.12g, in accordance with FHWA guidelines [11–13]. Given the high importance of the structures surrounding the excavation area, a conservative value of k_h = 0.12 was adopted for the quasi-static analysis.

3.1. Excavation Stabilization

Several alternative methods were considered for stabilizing the 15 m-deep excavation near the Shrine of Imam Hussain, including secant pile walls, soldier pile walls, and sheet pile walls. Secant piles were initially considered; however, they were rejected due to their higher cost, longer installation time, and the potential for excessive groundwater inflow in the saturated soil layers. Soldier piles and sheet piles were deemed insufficiently rigid to control lateral deformations within the strict allowable limits (≤4 cm). The T-shaped diaphragm wall was selected because it provides high stiffness, effective groundwater cutoff, and reduced excavation deformation, meeting both structural safety and cultural heritage protection requirements in this sensitive site.

To stabilize the excavation in this project, and considering structural requirements, it was decided to separate the retaining wall from the main structure. This design ensures that no lateral soil pressure except at the foundation level is transferred to the main structure. Consequently, the permanent earth pressure, including seismic loading, is entirely supported by the surrounding retaining wall, with the main structure engaged only at the foundation level. In this context, diaphragm walls were employed both as a cutoff wall and a stabilizing element. The primary advantage of diaphragm walls is their ability to provide effective waterproofing, significantly reducing groundwater inflow during dewatering operations. Based on comprehensive studies and analyses, the wall length was optimized to minimize the inflow rate to a level that can be effectively managed through pumping over the project's operational life.

A one-meter-thick diaphragm wall was constructed to stabilize the excavation. The diaphragm wall in this project was designed in a T-shaped configuration (buttressed retaining wall). This design includes a series of buttress walls installed at variable intervals of 2.6 to 5.2 meters. These buttresses are constructed as separate diaphragm panels oriented perpendicular to the main cutoff wall and are connected to the waterproof wall panels through shear connections. At the elevation where the foundation of the main structure intersects the diaphragm wall and the buttresses, the thickness of the foundation remains unchanged from that of the main structure. This is because the foundation at this level is designed to resist seismic loads only, while the earth pressures are borne by the base of the buttress elements.

In the design of the excavation stabilization system, the diaphragm wall was modeled as reinforced concrete with material properties selected based on standard engineering practice and site-specific design requirements. The following parameters were used in the numerical analyses:

- Elastic modulus (E): 30,000 MPa
- Poisson's ratio (ν): 0.2
- Compressive strength (f′c): 30 MPa
- Tensile strength: Assumed negligible in PLAXIS 2D but accounted for in SAP2000 structural analysis through reinforcement design.

The concrete material was assumed to behave linearly elastic in PLAXIS 2D for global deformation evaluation, while the wall's bending moments and stresses were verified in SAP2000 considering reinforcement requirements. Considering the soil characteristics and environmental conditions, Portland cement resistant to chloride and sulfate attack was selected. Due to the high groundwater level at the construction site and the presence of chemical agents that could compromise the durability of the concrete and surrounding soil, micro silica (silica fume) was incorporated into the mix. Specifically, 8% of the cement weight was replaced with micro silica to enhance durability and reduce water permeability through the wall. Micro silica also improves the bond between fresh concrete and adjacent surfaces, reduces segregation of concrete components, and further minimizes water infiltration. Because of the low water-to-cement ratio and the inclusion of micro silica, a high-range water-reducing admixture (superplasticizer) was used to achieve a target slump of at least 18 cm, ensuring proper workability.

For the numerical analysis of the excavation stabilization system, PLAXIS 2D (version 20) software was utilized. The Hardening Soil Model with Small-Strain Stiffness (HSS) was adopted for simulating soil behaviour, while a linear elastic model was applied for concrete. Structural elements, such as plates available in PLAXIS, were used to represent the diaphragm wall and other components. Both static and pseudo-static (seismic) analyses were conducted. The geometry and boundary conditions of a sample model are shown in Figure 3. A 15-node triangular element mesh was used to ensure higher accuracy, with an example of the generated mesh presented in Figure 4. Loading is considered according to Table 2. Due to having dead (D), live (L) and earthquake (E) loads, the load combinations used according to the American concrete code ACI318M-14 is as follows:

Table 2. Load combinations based on ACI318 [14]

Primary Load	Load Combination
D	$U = 1.4D$
L	$U = 1.2D + 1.6L$
E	$U = 1.2D + 1.0E + 1.0L + 0.2S$
E	$U = 0.9D + 1.0E$

The interaction between the diaphragm wall and surrounding soil was explicitly represented using interface elements in PLAXIS 2D. These elements simulate the shear and normal behavior along the wall–soil contact and allow relative displacement while transferring shear stresses. To model the shear connection, the interface elements were assigned a strength reduction factor (Rinter= 0.7), which reduces the soil shear parameters (cohesion and friction angle) at the contact to account for imperfect bonding between the soil and the concrete wall. This approach allows controlled mobilization of shear resistance along the interface, capturing realistic soil–structure interaction under excavation loading. The normal stiffness of the interface was chosen sufficiently high to prevent unrealistic penetration between the soil and wall, while the tangential (shear) stiffness was governed by the reduced shear parameters. This method effectively highlights the progressive transfer of lateral loads from the soil to the wall and the corresponding wall deformations.

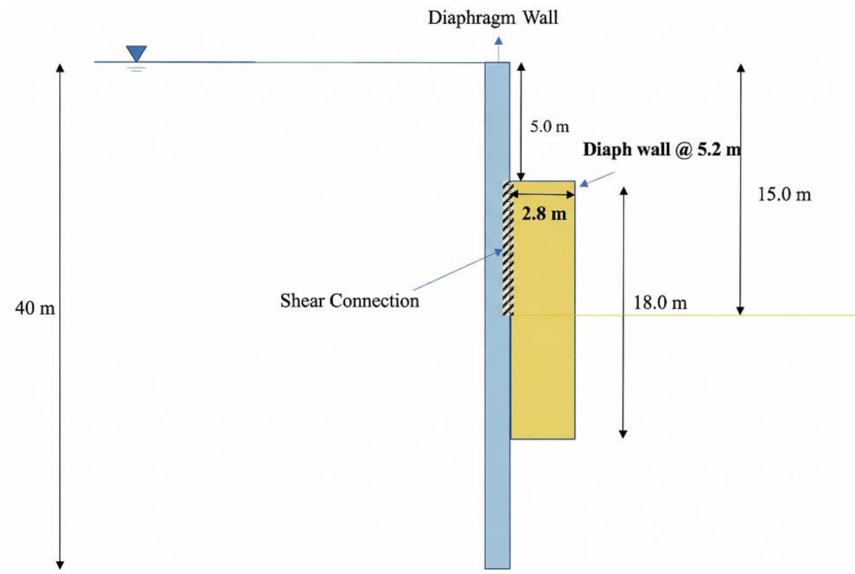


Fig. 3. Section of buttressed retaining wall

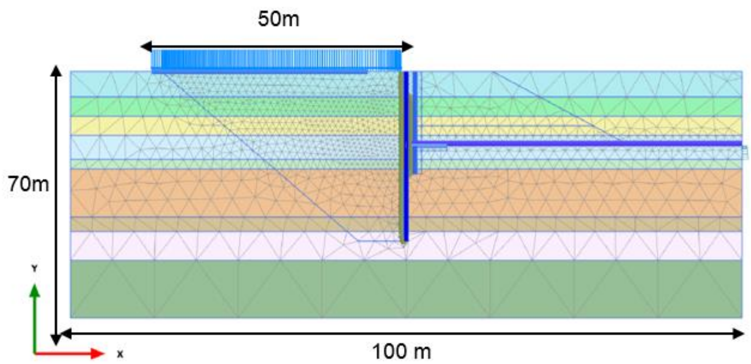


Fig. 4. PLAXIS Model of retaining system

3.2. Controlling Displacement and Damage Level Nearby Structures

The Mair and Burland criterion [15] was applied to evaluate wall movements, adjacent building responses, and potential structural damage. As illustrated in Figure 5, the angular distortion (δ/L) is calculated in accordance with the recommendations of Eurocode 7. To obtain a more accurate measure, the rigid rotation of the foundation (tilt) is subtracted from the total angular deformation, and the net value of δ_{max} is used in the calculation. Neglecting the tilt component leads to an overestimation of δ_{max}, which may result in a more conservative assessment. However, due to uncertainties in the actual behavior of surrounding structures, both approaches considering and neglecting the foundation tilt were evaluated in this study. Of the two, the latter (excluding tilt correction) typically yields the more critical results.

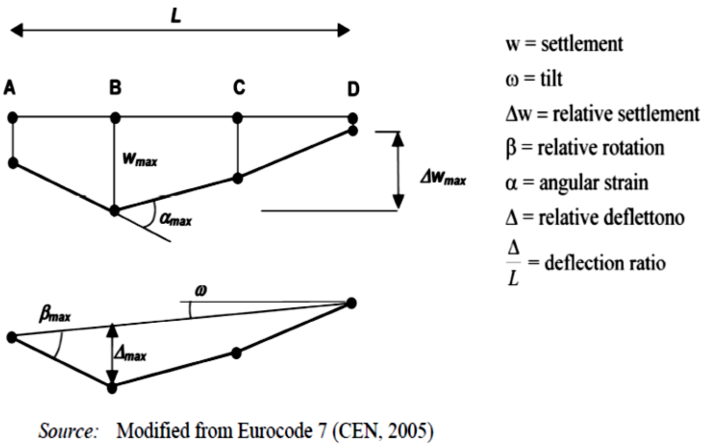


Fig. 5. The effect of foundation rotation on controller displacement ratio [11]

To control the displacement of the excavation wall, the guideline provided in FHWA-IF-99-015 [12,16] was followed. According to this standard, a value of 0.005H is used to estimate the maximum horizontal displacement of the wall in dense sandy soils under active earth pressure conditions. For an excavation depth of 14.2 meters, this corresponds to a maximum allowable displacement of $\delta_{max,h}$= 0.005H = 71 mm. To monitor and manage displacement throughout the excavation process, the maximum horizontal displacement of the wall was tracked at various excavation stages. Figures 6 and 7 illustrate the progression of wall displacement during excavation, showing how it evolves in response to excavation depth and construction activity.

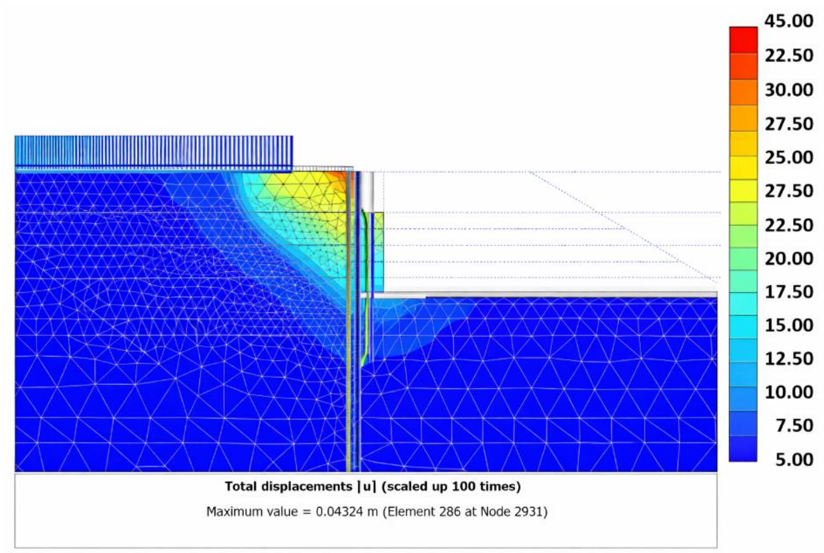


Fig. 6. Total displacement contour in Plaxis numerical model

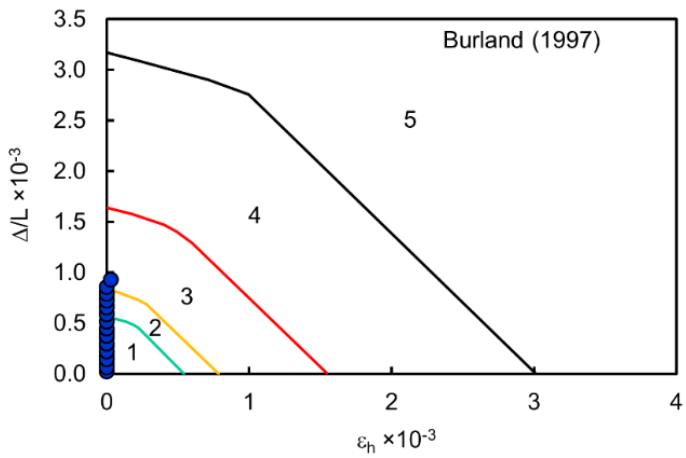


Fig. 7. Retaining wall and adjacent structures displacement control [15]

3.3. Hydraulic Gradient and Seepage Control

Due to the depth of 40 meters of the wall, the amount of inlet flow rate is low and there are no critical conditions in terms of hydraulic gradient and seepage. The results of seepage and gradient are almost the same for all sections and are as shown in the table below. Also, the water flow vectors during pumping are shown in Figure 8.

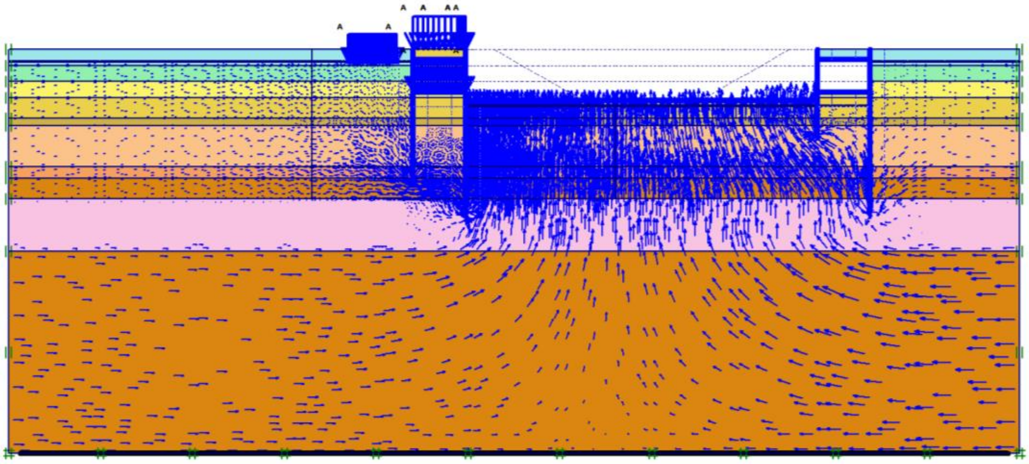


Fig. 8. Water flow vectors during pumping (Flow field, extreme velocity 19.33×10^{-3} m/day)

Table 3. Seepage analysis results

V (m/day)	K (m/day)	I	Q (m^3/day/m)	P (m)	Q (m^3/day)	Q (lit/sec)
0.010	8.64	0.0012	0.30	1000	300	3.5

4. Structural Analysis

After analyzing the interaction of the soil and the retaining structure in Plaxis software, the axial and bending forces of the sections are obtained. For concrete design, according to ACI, the external forces are multiplied by a factor of 1.6 in the static state and by a factor of 1 in the pseudo static state. Then the concrete cross-section is obtained with the amount of reinforcement required for the load resulting from the analysis. The forces resulting from the analysis for different sections of the retaining structure used in the design are shown in the following diagrams. To obtain the final forces of the section, the existing forces, described below, must be combined with each other, notations are shown in Figure 9.

$$P = P_{W1} + P_{W2} \tag{1}$$

$$V = V_{W1} + V_{W2} \tag{2}$$

$$M = M_{W1} + M_{W2} - P_{W1}d_1 + P_{W2}d_2 \tag{3}$$

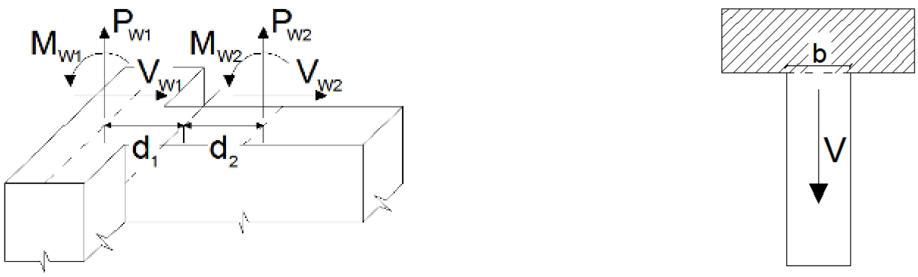


Fig. 9. Notations used in the analysis

The shear force for the design of reinforcements is calculated from the following equation. The reinforcements are designed for each surface of one square meter, so the design force will be equal to ʈ.

$$\tau = \frac{V.Q}{I.b} \tag{4}$$

The variation of bending moment, shear force, and axial loading for the cut-off wall are shown in Figure 10. Also, the variation of bending moment and shear force for the Buttress wall are shown in Figure 11. The shear force of reinforcements at the junction of the buttress and the cut-off wall obtained from Plaxis software is shown in Figure 11.

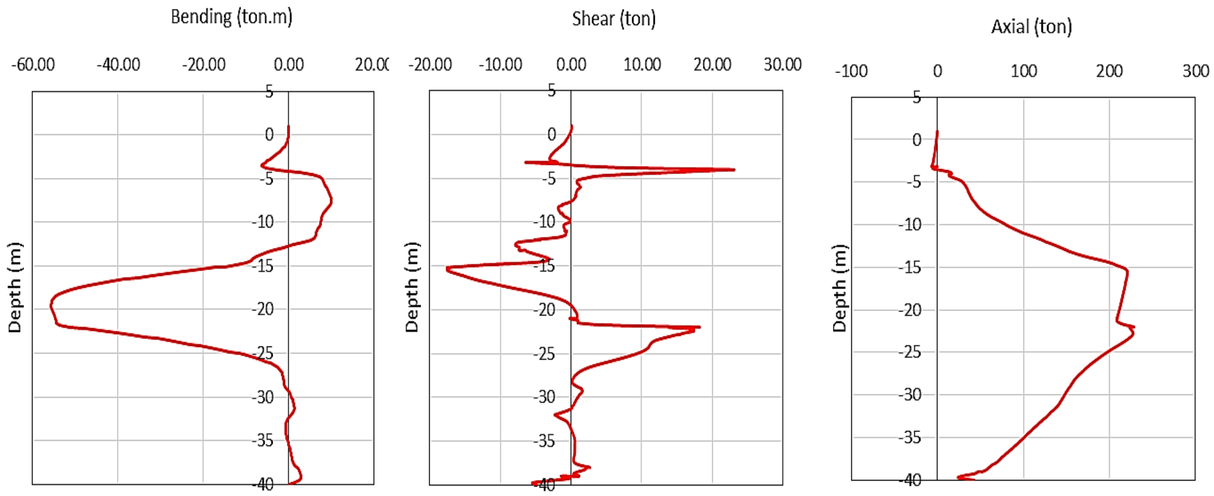
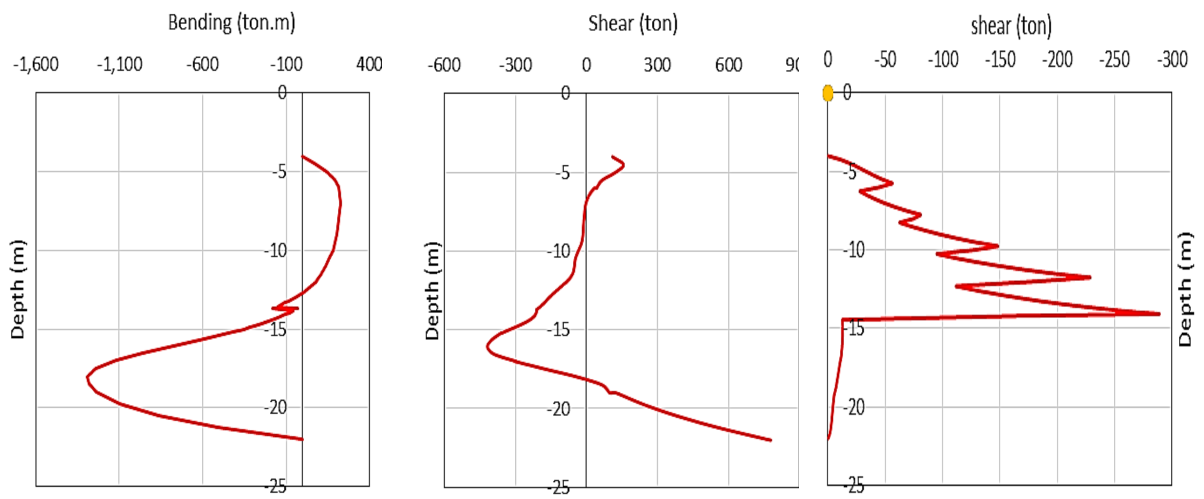


Fig. 10. Cut-off wall forces



(a) (b) (c)

Fig. 11. Buttress wall forces (A and B) and shear forces at the junction of the buttress and the cut-off wall (C)

SAP2000 software was utilized to calculate the stresses induced in the panels of the central cut-off wall. To accurately simulate the loading conditions, soil pressure data obtained from PLAXIS was imported and applied to the structural model in SAP2000. This approach allowed for a more realistic representation of soil-structure interaction. Figure 12 illustrates the stress distribution at various critical points under the most severe load combinations.

In the cross-section of Ha'er and Bein' Alharamain, due to the convex geometry of the wall, the type of force created in the elements, especially the waler beams, is different from other sides, and this side should be modeled separately. Therefore, according to this issue, this side has been modeled in 3D in SAP2000 software, and the results of its analysis are presented in Figure 13.

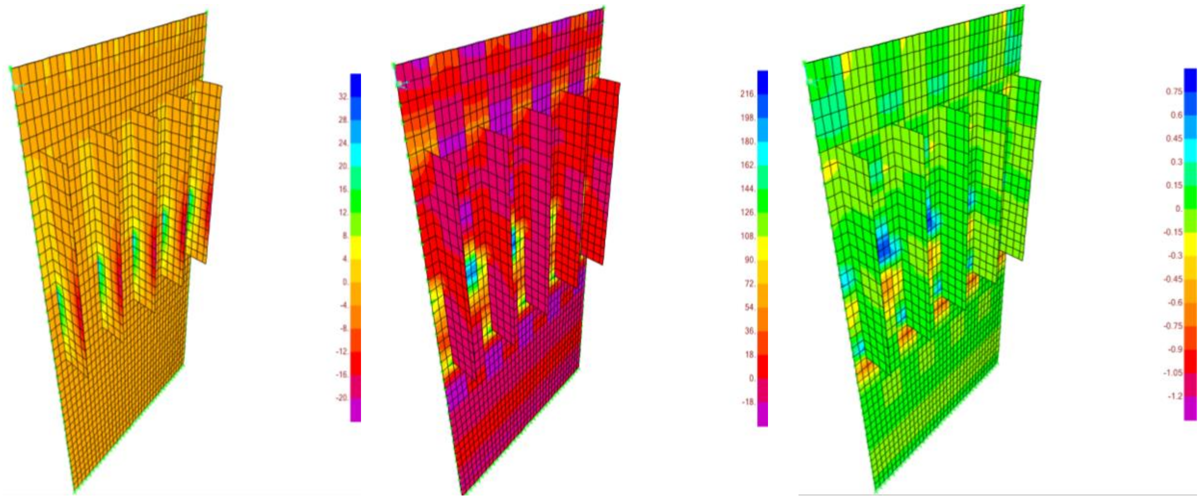


Fig. 12. The maximum stresses created in the model

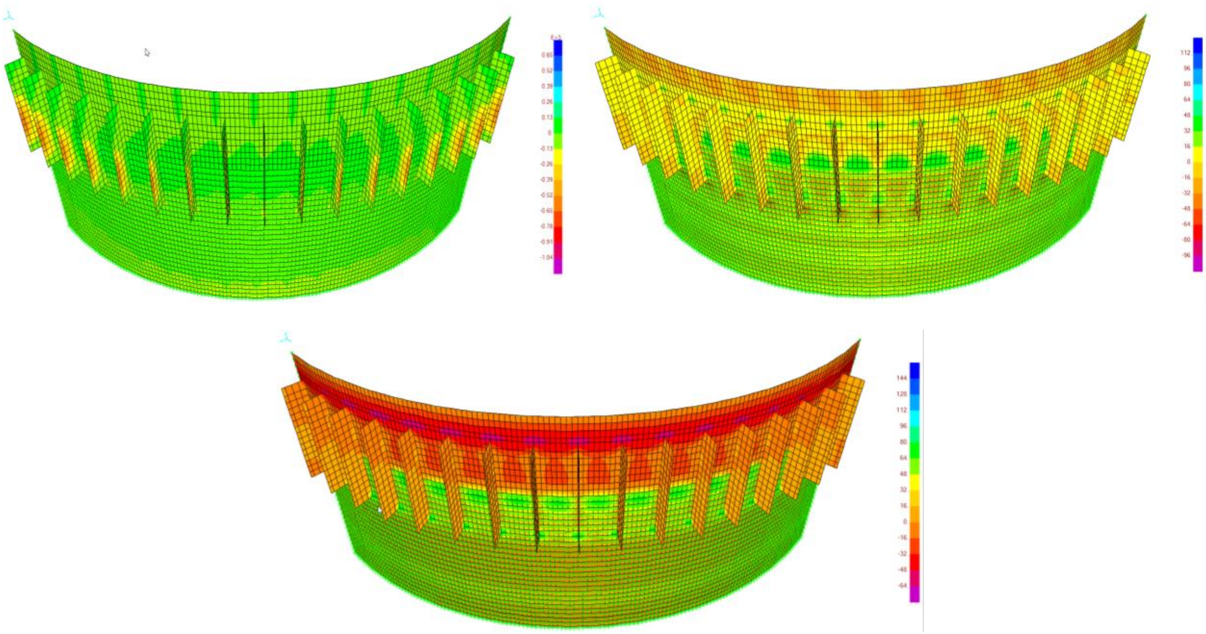


Fig. 13. The maximum stresses created in the Ha'er section

5. Instrumentation and Monitoring

In important excavation projects such as the current project, continuous monitoring of influential parameters such as deformations and pore water pressure is important and essential. In this project, it is necessary to use tools such as inclinometers to monitor the behavior of the diaphragm wall and piezometer in the inner and outer parts of the excavation due to the presence of high ground water level, see Figure 14. This issue is important both for stability and for controlling the adequacy of the drainage system. In the following, the details of the instrumentation plan will be discussed. One of the most important measurement parameters is the pore water pressure in the inner area of the well and also around the cut-off wall. At this stage, approximately 1 piezometer is considered for every 5000 square meters inside the excavation area. Also, for every 150 meters of wall length, a piezometer has been considered behind the cut-off wall to control the quality of its performance and also to determine the water pressure on the wall. These piezometers are installed in the perimeter section of the excavation and at two different depths of -10 and -20 meters. At this stage, 11×2 = 22 piezometers are required for monitoring. Three inclinometers are used to measure the horizontal movement of the wall. Also, tiltmeter is used to monitor deviations and tilts in surrounding structures, especially sensitive structures. Finally, it is appropriate to use surveying method to control surface displacements. The location of benchmark points around the site should be determined in such a way that, in addition to proper visibility, they are installed in areas that are not affected by movement caused by excavation, water pumping, and executive operations. The targets should be installed both in the area around the pit and on the edge of the diaphragm wall. On average, two targets on the diaphragm wall and two targets perpendicular to the wall (up to a distance of 30 meters) are predicted for every 10 meters of wall length. According to the approximate perimeter of 600 meters for the excavation area, about 120 targets on the wall and 120 targets behind the wall and outside the excavation area are expected.

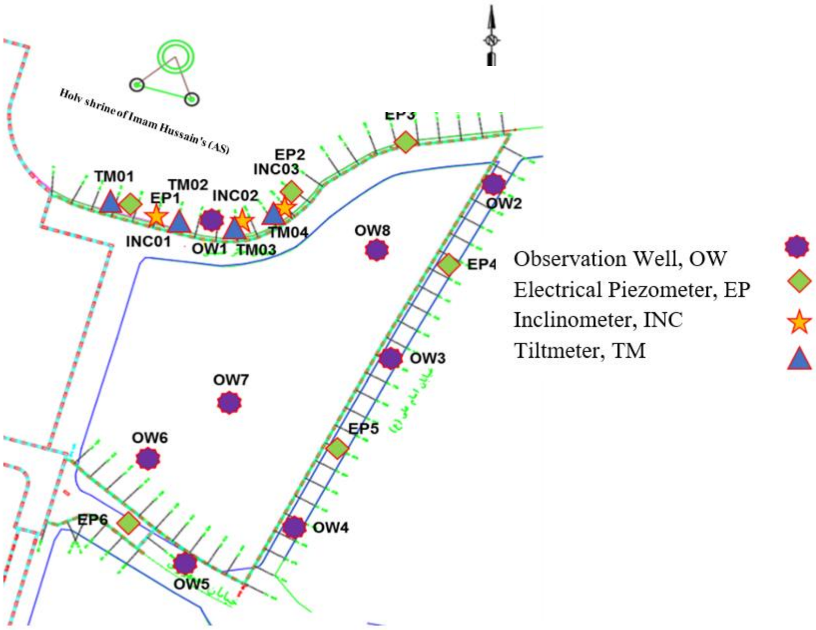


Fig. 14. Placement of the proposed precision instruments for Imam Hassan's (AS) courtyard

The excavation site was instrumented with inclinometers, piezometers, and surface settlement markers to monitor soil and wall behavior during excavation. Preliminary field measurements from the initial and intermediate excavation stages indicate lateral wall displacements and pore water pressures that are consistent with the trends predicted by the PLAXIS 2D and SAP2000 models. Inclinometer readings show maximum lateral deflections near the wall crown, matching the predicted range of approximately 4 cm. Piezometer readings confirm that the diaphragm wall effectively limits groundwater inflow, in agreement with the modelled seepage distributions. These preliminary data provide confidence in the model predictions while also highlighting areas where ongoing monitoring is critical. Full-scale measurements will continue throughout the excavation process to further validate and refine the numerical analyses. This addition helps demonstrate that the numerical predictions are supported by field observations, while also acknowledging limitations and the need for continued monitoring.

6. Conclusion

In this article, the stabilization plan of the excavation of Imam Hussain's (AS) holy shrine was presented. In this project, the diaphragm wall method was used to stabilize the excavation and simultaneously solve the problem of water seepage. As seen in the outputs taken from the models made using PLAXIS and SAP200 software, the stabilization plan provided by the diaphragm wall method is responsive both from the point of view of deformation and from the point of view of lateral soil forces and has the desired adequacy. In this research, it was seen that the diaphragm wall method, in addition to controlling the displacement of the retaining wall (in the range of 4 cm) and controlling the damage level of the adjacent structures at the permitted levels, can also control the problem of seepage of subsurface water into the excavation. Also, one of the important advantages of the diaphragm wall is the removal of the concrete wall of the basement and the use of the diaphragm wall as the permanent wall of the basement, which is cost-effective both in terms of the economy and the construction time of the project. With the help of implementing the diaphragm wall as a T-shaped section in this project (Buttress wall), the need to implement temporary elements such as anchors, struts, raker beams and etc. was eliminated. This, especially in this project, considering the past experiences and the non-suitable soil condition (loose and running sand) and the high level of ground water, can solve the technical and operational problems of implementing elements such as anchors.

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