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Forensic Geotechnical Evaluation of Challenges in Constructing the Flood Protection Infrastructure (Wall) at Kadana Powerhouse, Gujarat, India

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Abstract: This paper presents a forensic geotechnical evaluation of the construction challenges and long-term stability of a 34-meter-high flood protection (FP) infrastructure (wall) at the Kadana Powerhouse, Gujarat, India, built within a geologically complex terrain. Following a critical design shift from underground draft tube tunnels to a cut-and-cover system necessitated by construction delays and adverse rock mass conditions previously confined slopes were exposed, revealing weak fault and foliation planes dipping unfavorably toward the excavation face. Emergency stabilization measures, including pre-stressed anchors and Perfo-bolts, were deployed to arrest progressive instability during construction. Nearly four decades later, a back-analysis incorporating 2D stability modeling, based on historical performance data was conducted to evaluate the long-term efficacy of these interventions. The analysis confirmed a notable improvement in the Factor of Safety (FOS) from 1.099 to 1.578 for sliding and from 3.053 to 4.048 for overturning, validating the adequacy of the original design response. The novelty of this study lies in its integration of retrospective analysis and performance monitoring to assess legacy infrastructure a rarely documented forensic geotechnical case of FP wall from India. The findings underscore the importance of integrating geological insight with adaptive engineering during both the design and construction phases and offer valuable guidance for future infrastructure development in weak or structurally disturbed rock masses.

Keywords: Forensic Geotechnical Analysis; Flood Protection Wall; Stability Assessment; Fault and Foliation Planes; Kadana Powerhouse; Remedial Measures.

1. Introduction

The Kadana Hydro-Electric Project (KHEP), located in the Panchmahal district of Gujarat, India (23°18′30″N, 73°49′45″E), is a major infrastructure development aimed at hydroelectric power generation and flood control along the Mahi River (Fig. 1). The project features a 65-meter-high composite dam, a 106-meter-long powerhouse housing four 60 MW turbo-generator units (two reversible and two conventional), and several critical ancillary structures, including draft tube tunnels and a flood protection (FP) wall. The FP wall was specifically designed to prevent flooding of the powerhouse service bay from tailrace channel (TRC) discharges [1].

Initially, the design included underground draft tube tunnels. However, in the early 1980s, the layout was revised to a cut-and-cover configuration due to the presence of shallow, sheared rock cover. This condition had caused collapses in the underground structures, primarily due to delays between excavation and construction. The design modification enabled safer and more practical construction of the turbo-generator units, but it also removed the natural lateral confinement provided by surrounding rock. As a result, it increased the risk of instability in adjacent ancillary structures.

The most critical geotechnical challenge emerged during the 1982 construction of the 36-meter-high flood protection wall, which was founded on highly foliated and faulted phyllite and quartzite rocks belonging to the Aravalli Supergroup [2]. These geological conditions are known to significantly compromise the stability of both surface and underground structures [3–6]. As excavation and construction progressed, the flood protection wall experienced severe geotechnical issues, including sliding and tilting, driven by the altered site conditions and inherent weaknesses of the rock mass. A forensic geotechnical re-evaluation was undertaken to analyze the effectiveness of stabilization techniques. The stability of wall was governed by vertical and horizontal driving forces as well as resisting forces at the base [7–9].

A review of the available literature indicates that, to counteract destabilizing forces, prestressed anchors were installed to provide horizontal resistance, while Perfo-bolts were used to stitch the foliation planes and fractures within the foliated phyllite rock mass. These bolts not only increased shear resistance but also contributed to sealing the fractures through associated grouting, thereby reducing the risk of water ingress and further weakening of the rock mass [10–12]. The effectiveness of such support systems in weak rock conditions has also been documented in various tunnel and mining projects [13, 14].

Although these stabilization measures were based on the geotechnical knowledge and engineering practices available at the time [8, 15, 16], modern tools and systematic rock mass classification methods such as the Q-system [17] and Rock Mass Rating (RMR) [18] were not applied. The use of these classification systems, had they been available or widely adopted, could have significantly improved the understanding of rock mass behavior. This, in turn, would have enabled more rigorous and optimized planning of support strategies, including the design of anchoring systems, during the staged construction of the wall u nder adverse geological and changing site conditions [19].

The objective of this study is to carry out a forensic geotechnical evaluation of the flood protection infrastructure. It aims to examine the challenges faced during its construction and evaluate how effective the implemented remedial measures were. The study also seeks to draw lessons that can help in the design, assessment, and repair of similar structures in complex geological conditions. This work is important because it shows how a retrospective analysis of a major hydropower structure still in service after more than forty years can offer useful insights. It helps us understand how well the stabilization strategies have performed over the long term. These findings can also guide future infrastructure projects in geologically complex and structurally weak areas. Forty to forty-five years ago, modern techniques for rock mass classification and evaluation were not widely available. Advanced tools such as three-dimensional Finite Element Analysis, which could have enhanced predictive capabilities for infrastructure construction on complex foundations, were either in their infancy or not in use [20, 21]. Similarly, Machine Learning techniques were not yet developed and therefore could not be applied to optimize the design of the flood protection wall under varying site conditions. Despite these limitations, the Kadana case highlights the enduring value of simple two-dimensional stability analysis. When guided by practical field experience, such traditional methods can still provide reliable insights and serve as a foundation for future infrastructure development in complex geological settings. projects in geologically complex terrains.

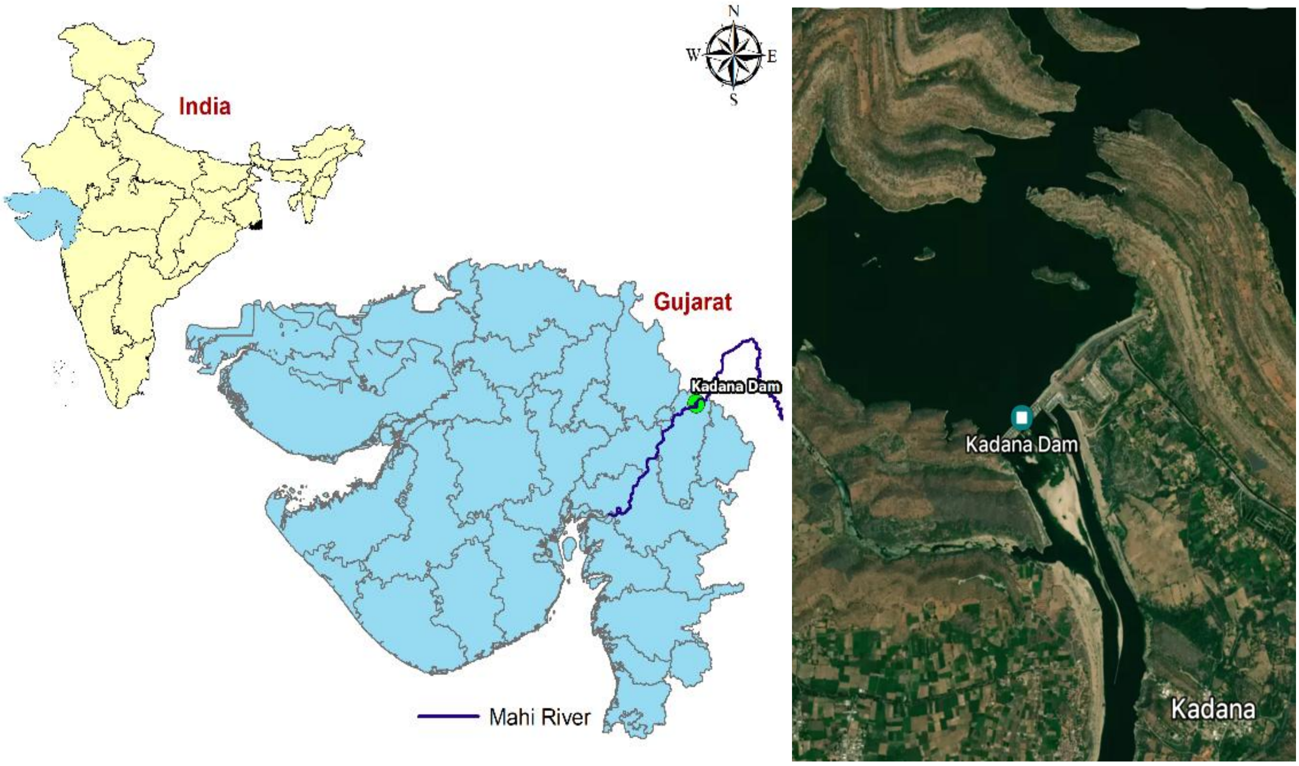

Fig 1. Location Map of Kadana Dam

2. Study Area

2.1 Geological and Geotechnical Setup

The Kadana Powerhouse site is situated within the Kadana Formation of the Lunawada Group, belonging to the Paleo-Proterozoic Aravalli Supergroup. The area is geologically complex, comprising tightly folded and faulted phyllite, quartzite, meta-subgraywacke, and mica schist [2] (Fig. 2). The rock mass is structurally disturbed, characterized by a major fold axis plunging SSW to WSW and foliation planes trending NNW–SSE, with steep dips (70°–80°) towards the NNE and SSW [2, 22]. The site exhibits closely spaced joints, a combination of low- and high-angle faults, and crisscross shear zones, all of which contribute to highly unfavorable tunneling and foundation conditions. As per the Q-system classification conducted at other projects [17], tunneling in such highly jointed and sheared rock masses demands robust and customized support measures to ensure stability. A critical fault zone, located approximately 62 meters downstream of the powerhouse, consists of about 0.5 meters of shattered rock and clay gouge. It dips upstream at ~23°W and poses a significant stability risk to the FP wall and nearby structures [2]. Consideration of such weak zones occurring in the foundation is essential in the design [3] for safe construction of surface and underground structures. The Kadana Powerhouse is situated between the Masonry Dam and the Earthen Dam (Fig. 3). A flood protection wall is positioned on the left side of the tailrace channel, adjacent to Draft Tube Tunnel No. 1. Originally designed as an underground structure, the draft tube tunnel was later modified to an open cut-and-cover configuration. The construction of the flood protection wall necessitated a deep foundation key at R.L. 72 m, which is 8 meters below the general foundation level of R.L. 80 m. During previous construction phases, excavation depths reached up to 30 meters due to tunnel collapses, resulting in the removal of lateral confinement. This led to observable tilting and widening of the flood protection wall in February 1982 [2]. Furthermore, instability was aggravated by the saturation of the surrounding rock mass, likely associated with the impoundment of the upstream reservoir.

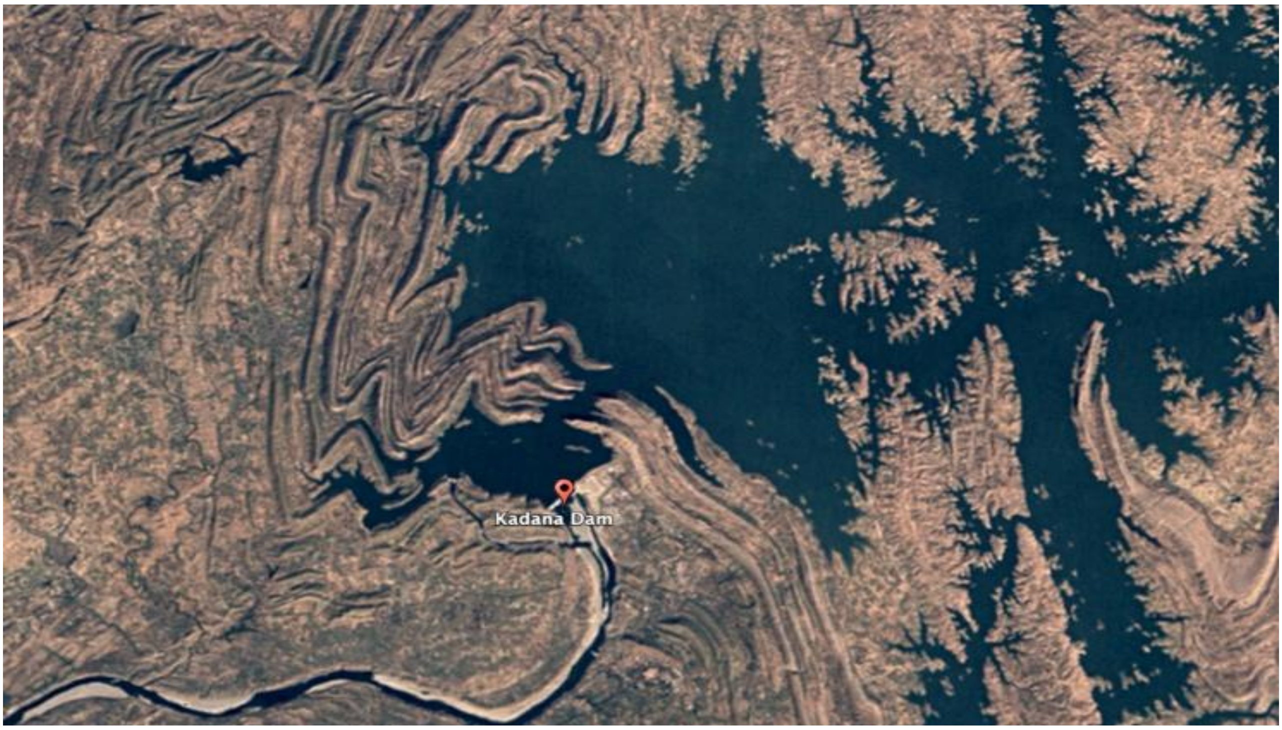

Fig 2. Satellite picture showing folded Quartzite and Phyllite Rocks in and around Kadana Project Site

Fig 3. Layout of Kadana Powerhouse

The rock mass in the powerhouse area was extensively fractured and sheared, rendering it geomechanically fragile. Excavations for the draft tube tunnels and wall foundations further disturbed this already weakened geological setting, triggering roof collapses within the tunnels and destabilizing adjacent structural elements [23]. As emphasized by [17], site-specific geotechnical assessments are crucial for evaluating rock mass quality and for developing appropriate engineering interventions to mitigate such risks in complex geological environments.

2.2. Hydrology/Hydraulics of F.P. Wall

The flood protection wall was designed to safeguard critical infrastructure, particularly the powerhouse service bay, from backwater effects and flood-induced tailrace channel (TRC) inundation. Hydrologically, the tailrace channel forms the lower reservoir in the pumped-storage scheme, receiving discharge from both conventional and reversible turbines. Based on available data, the TRC is designed to handle a maximum water level of 114 m during peak turbine discharge or flood events. The flood protection wall has a crest elevation of 116 m, thereby incorporating a freeboard of approximately 2 m to accommodate hydraulic surges, wave run-up, and potential backwater effects under transient conditions. The hydraulic design of the wall ensures that even under simultaneous operation of all four turbines and high downstream levels, the TRC water remains below critical levels, preventing overtopping and ensuring structural and operational safety. This configuration reflects standard flood protection practices in hydropower installations with pumped storage capabilities and high tailwater variability.

3. Methodology

The forensic geotechnical evaluation of the flood protection wall at the Kadana Powerhouse was conducted through a comprehensive, multi-phase approach designed to investigate construction challenges and assess the long-term performance of stabilization measures. The methodology included the following key components.

3.1 Historical Review and Data Compilation

Archival project documents, design drawings, and construction records from the 1980s were systematically reviewed to understand the original design intent, prevailing site conditions, observed anomalies, and the rationale for the adopted remedial measures. This review established the baseline for subsequent forensic assessment.

3.2. Field Investigations and Geological Mapping

During the original construction phase, extensive field investigations were carried out to characterize the exposed foundation geology. This included mapping weak zones and recording the orientation of fault and foliation planes. Engineering geological cross-sections were developed using observed lithological units and structural features particularly the sheared phyllites and intercalated quartzite with clay-rich fault gouge zones.

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3.3. Geotechnical Assessment and Analytical Modeling

Original slope stability and foundation analyses were revisited using two-dimensional analytical models. Key geotechnical parameters such as unit weight, cohesion, and internal friction angle were adopted from site-specific investigations and supported by published references (e.g., [3]; IS: 456-2000). Deterministic sliding and overturning analyses were conducted to calculate the Factor of Safety (FOS) before and after the implementation of treatment measures.

3.4. Evaluation and Design Review of Stabilization Measures

The adequacy of stabilization techniques implemented during construction was critically examined. This included pre-stressed rock anchors, Perfo-bolts, struts, staged concrete construction, and drainage systems. The original design approach an integration of empirical methods and analytical modeling was reviewed in light of the prevailing geological complexities. The objective was to assess whether these measures were technically sound and sufficiently robust to ensure long-term structural integrity.

3.5. Forensic Back-Analysis, Validation, and Lessons Learned

A forensic back-analysis was undertaken nearly four decades after construction to evaluate the enduring effectiveness of the stabilization measures. Sensitivity analyses were performed to assess the influence of critical geotechnical parameters, particularly the friction angle (ϕ), on the global stability of the structure. This phase integrated historical performance data, analytical modeling results, and field observations to validate the original treatment strategies. The findings were benchmarked against established geotechnical standards and translated into actionable lessons for future infrastructure projects especially those involving foundations on faulted and foliated rock masses under complex geological conditions.

4. Case Study of Geotechnical Challenges and Treatment During Construction

4.1. Geotechnical Challenges of Dam and Powerhouse

The construction of the flood protection wall and related structures at the Kadana Dam and Powerhouse encountered serious geotechnical challenges due to weak geological features such as sub-horizontal to low dipping faults and steeply dipping foliation planes in the foundation rock. These discontinuities raised critical concerns of sliding and instability during excavation and construction of dam and ancillary structures. To mitigate these issues, concrete shear keys were installed in the dam and power dam blocks to resist sliding. Additionally, a network of underground draft tube tunnels was provided to increase passive resistance.

Draft tube tunnels of powerhouse were partially excavated in 1976, were left incomplete due to administrative delays. When work resumed in 1982, it was noticed that the rock mass deteriorated with time resulting in roof collapses. Consequently, the remaining tunnels were redesigned as cut and cover structures. Thus, under altered site conditions draft tube tunnels were reconstructed with reinforced concrete. The sudden change in the design necessitated deep open excavation, which in turn created instability in nearby structures, including the service bay columns and the FP wall. This FP wall, standing 34 meters high and constructed in 1982 adjacent to Draft Tube Tunnel No. 1, intersected several shear zones as well as a major fault filled with shattered rock and clay gouge. During excavation, signs of structural distress became evident most notably, a horizontal separation of 39 mm and tilting of 17 mm at RL 92 m, along with additional tilts of 45 mm and 32 mm observed at RL 100 m.

Initially, excavation was restricted to a depth of 8 meters. However, following a tunnel collapse, it had to be extended to approximately 30 meters, reaching below the base of critical structures. This deepening created steep, unsupported slopes, exacerbating the instability. Due to the surrounding structural constraints, reconfiguring these slopes was not feasible, ultimately necessitating a temporary suspension of excavation activities. Subsequently, based on the 2D stability analysis, stabilization measures to prevent collapse of the FP wall were adopted.

4.2. Stabilization and Remedial Measures of Flood Protection Wall of Powerhouse

The stabilization of critical infrastructure in adverse geological conditions requires a combination of analytical evaluation, empirical knowledge, and real-time monitoring. In the case of the Kadana Dam FP wall, an integrated set of geotechnical treatments was employed to counteract sliding, tilting, and potential failure associated with fault- and foliation-related instability (Fig. 4). These interventions reflect the application of rock engineering principles as outlined by [6, 24], and case studies by [13]. A combination of active and passive support systems was implemented to stabilize the wall and adjacent rock slopes:

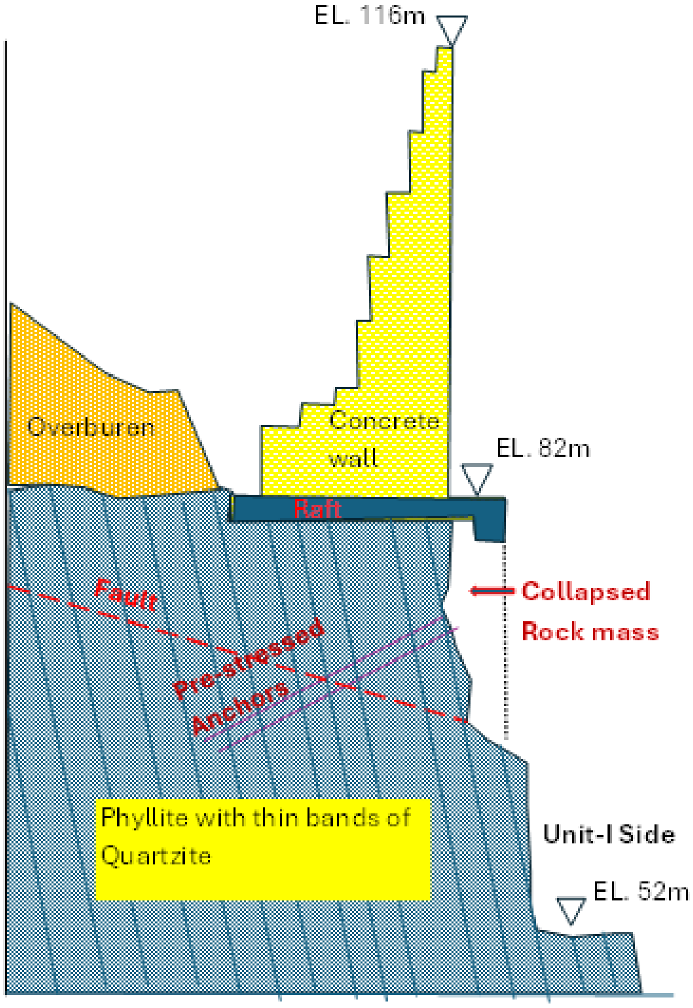

Fig 4. Typical cross-section of Flood Protection Wall and Foundation during Progressive Construction.

Pre-stressed Rock Anchors: Three rows of 100- tonne capacity anchors (10 m length) were installed at 3 m intervals, inclined at 20° across the fault zone. These anchors were tensioned to 80% of their capacity to tie the footwall to the hanging wall of fault, thereby actively resisting sliding of Foundation of FP Wall.

Staged Excavation: The excavation was carefully conducted in 2 m stages, with concrete lifts cast in synchrony to avoid sudden loss of confinement.

Grouting: To seal open joints and consolidate the weak foundation rock, low pressure grouting was carried out in the foundation.

Drainage Measures: Weepholes and an internal drainage system were installed to reduce pore pressures, control uplift, and drain seepage water. This step was vital to maintain long-term integrity, particularly where open joints, foliation fault were present.

5. Back Stability Analysis of Flood Protection Wall

5.1. Geotechnical Parameters used in the Stability Analysis

An accurate assessment of wall stability under adverse geotechnical conditions requires a rational evaluation of geometry, material properties, and failure mechanisms. The analysis was conducted using standard geotechnical engineering principles and input values derived from laboratory testing, empirical correlations [25], and in-situ observations.

Main Parameters:

- Geometry: Base width = 20 m, top width = 2 m, height = 50 m (34 m above foundation), area = 550 m².
- Material Unit Weights: Concrete = 24 kN/m³, Phyllite = 26 kN/m³ [26].

Discontinuity Parameters:

- Fault plane: ϕ = 25°, c = 0 (typical for gouge-filled, sheared zones).
- Fault plane: ϕ = 25°, c = 0 (typical for gouge-filled, sheared zones).
- Foliation plane: ϕ = 30°, c = 0 (steeply dipping, open).

These conservative assumptions were used for worst-case analyses prior to and after stabilization.

5.2. Sliding and Overturning Stability Analyses

The effectiveness of stabilization efforts can be quantified through stability analyses that assess factors of safety (FOS) against sliding and overturning. The analyses here follow methods outlined by [6], evaluating both pre- and post-treatment scenarios.

5.2.1. Sliding Stability

Sliding failure occurs when the horizontal force acting on the wall exceeds the resisting frictional force at the base. The resisting force is calculated using the following formula (Eq.1):

Resisting Force = μ × F_vertical (1)

Where: μ = tan(φ), which is the friction coefficient derived from the angle of the fault plane.

F_vertical is the total vertical force acting on the wall (600 t per meter).

The Factor of Safety against sliding is calculated as (Eq.2):

$$FOS=\frac{R}{T} \quad (2)$$

where R = N · tan(ϕ) + Rbolts (resisting force, with Rbolts = 0 before treatment, 981 kN/m after), and T = W · sin (23°) − Fa,parallel (driving force). Overturning stability was assessed about the toe, with stabilizing moment Ms = N · x and overturning moment Mo = T · y. The foliation plane was analyzed similarly for local stability [24].

Before Treatment:

- Normal force = 12,150.6 kN/m
- Driving force = 5,157.24 kN/m
- Frictional resistance = 5,665.82 kN/m
- FOS = 1.099 (marginal)

After Treatment:

- Increased normal force = 12,486.1 kN/m (due to anchors)
- Additional resistance from perfo-bolts = 981 kN/m
- Reduced driving force = 4,309.85 kN/m
- FOS = 1.578 (safe)

5.2.2. Overturning Stability

Overturning stability was evaluated by comparing the resisting moment, which arises from the weight of the wall to the overturning moment caused by the horizontal force exerted by the anchors. The vertical height from the base of the wall to the point of application of the horizontal force is denoted as H.

The resisting moment was calculated using the expression (Eq.3):

Resisting Moment = F_vertical × (H / 2) (3)

The overturning moment was calculated as (Eq.4):

Overturning Moment = F_horizontal × H (4)

The Moment Safety Factor (MSF), which indicates the stability against overturning, was obtained by the ratio (Eq.5):

MSF = Resisting Moment / Overturning Moment (5)

Before Treatment:

- Ms = 111,846.27 kNm/m, Mo = 36,631.94 kNm/m, MSF = 3.053

These values show a marked improvement in stability due to anchor force addition and reduced driving moments.

5.2.3. Stability Against Foliation Plane-Induced Failure

Discontinuities such as steep foliation planes often represent potential slip surfaces in slope and wall stability analyses. As emphasized by [4], even when dips are steep (~75°), localized instability can occur if the rock mass cohesion is compromised or if the planes are open.

FOS Results:

- Before Treatment: FOS = 0.155 (highly unstable)
- After Perfo-Bolting: FOS = 0.232 (improved but still localized risk)

Perfo-bolts helped stitch the discontinuities and prevented their widening toward the free face. Additional bolting was necessary at critical locations to ensure adequate interlocking of rock slabs. The foliation plane (ϕ = 30°) yields a pre-treatment factor of safety (FOS) of 0.155, indicating instability. Installation of Perfo-bolts increased the FOS to 0.232, suggesting localized stabilization by pinning the fractures [4]. However, in the stability analysis, the orientation of the foliation planes at an angle of 30° to the alignment of the wall was not explicitly considered. These planes dip steeply at about 75°, reducing the likelihood of wall sliding. Nevertheless, to prevent the opening of these planes toward the exposed free face, additional local rock bolting was carried out alongside the Perfo-bolts.

5.2.4. Sensitivity Analysis: Evaluation of Friction Angle for Fault Zone

Given the variable nature of sheared and weathered rock, the friction angle (ϕ) is often uncertain. A sensitivity analysis was conducted to evaluate the impact of the fault zone friction angle (ϕ) on sliding stability, with ϕ varied from 15° to 35°, reflecting the range for sheared material (Table 1). The FOS was calculated for each ϕ value to assess how changes in material properties affect stability, providing insight into the reliability of the stabilization measures under varying geotechnical conditions. This aligns with slope engineering practices described by [6, 24]. The sensitivity analysis evaluated the FOS for sliding along the fault plane for ϕ values of 15°, 20°, 25°, 30°, and 35°. The analysis was performed by recalculating the resisting force ($R = N \cdot \tan(\phi) + R_{bolts}$) for each ϕ value, keeping other parameters constant, to determine the FOS (R/T). Results are presented in Table 1, showing that FOS increases with ϕ, with post-treatment values exceeding 1.5 for ϕ ⩾ 25°, confirming the critical role of accurate material characterization [27]. A sensitivity analysis was conducted to assess how variations in ϕ influence overall stability.

Findings:

- Friction angle range: 15°–35°
- FOS varied accordingly, confirming the criticality of the assumed ϕ = 25°
- Higher angles (e.g., ϕ ≥ 30°) ensured FOS > 1.5 Lower angles (ϕ ⩽ 20°) resulted in marginal or unstable conditions

Table 1. Sensitivity Analysis of FOS for Sliding vs. Friction Angle

ϕ (°)	R before (kN/m)	FOS (Before)	R after (kN/m)	FOS (After)
15	3,255.15	0.631	4,326.60	1.004
20	4,422.82	0.857	5,494.27	1.275
25	5,665.82	1.099	6,803.28	1.578
30	7,015.76	1.36	8,087.22	1.876
35	8,507.85	1.649	9,579.30	2.222

This analysis helped verify the robustness of the design under variable geological conditions and guided anchor spacing and load decisions.

6. Discussion

The post-treatment stability analysis revealed a marked improvement in both sliding and overturning resistance of the flood protection wall. For sliding along the fault plane, the anchoring system increased the normal force to 12,486.1 kN/m, while the Perfo-bolts contributed an additional resisting force of 981 kN/m. The horizontal component of the anchor force (847.39 kN/m) further reduced the net driving force to 4,309.85 kN/m. As a result, the total resisting force rose to 6,803.28 kN/m, yielding a factor of safety (FOS) of 1.578 satisfying the minimum recommended threshold [15, 24].

Regarding overturning stability, the initial overturning moment was 36,631.94 kNm/m, opposed by a stabilizing moment of 111,846.27 kNm/m, resulting in a pre-treatment FOS of 3.053. Following treatment, the stabilizing moment increased to 123,922.55 kNm/m while the overturning moment decreased to 30,612.80 kNm/m, improving the FOS to 4.048. This significant enhancement confirms the effectiveness of the implemented stabilization strategy [8, 9].

The wall's foundation rests on phyllite with pronounced foliation planes, inherently predisposing it to instability under load. To counteract this, Perfo-bolts were systematically installed at 10-meter intervals and 2-meter center-to-center spacing. These bolts, grouted into pre-drilled holes, effectively stitched together the foliated and fractured rock layers. Grouting also sealed open fractures, limiting water ingress and improving the overall integrity of the rock mass [12, 16].

This combined approach offered multiple benefits. In addition to sealing fractures, grouting enhanced cohesion across foliation planes, thereby reducing the potential for sliding. The Perfo-bolts provided supplementary tensile resistance, increasing the rock mass's shear strength and preventing further propagation of discontinuities. [11] and [10] have similarly emphasized the value of Perfo-bolts in improving the performance of foliated rocks such as phyllite and in mitigating hydraulic fracturing risks. Furthermore, [27] and [4] have documented similar reinforcement strategies in steep, unstable rock slopes, reinforcing the success of the Kadana approach.

Sliding and overturning analyses confirm the stability of the wall under current loading conditions. The integration of prestressed anchors and Perfo-bolts has proven to be an effective and durable design solution. By enhancing internal cohesion and sealing discontinuities, the treatment significantly reduced movement risks along pre-existing weak planes. These findings align with foundational geotechnical principles [7, 15] and demonstrate the value of integrating empirical design approaches with modern rock mechanics frameworks [5, 28].

7. Conclusion

This forensic geotechnical study of the flood protection wall at the Kadana Powerhouse highlights the impact of complex geological conditions on infrastructure stability. The site features low-dipping fault planes intersected by steep foliation within sheared phyllite and quartzite. These geological features became critical when natural confinement was removed during excavation.

The wall faced potential instability after a major design change from a tunnel to a cut-and-cover structure which exposed unfavorable rock mass conditions. Despite this, the timely implementation of stabilization measures, such as prestressed anchors and Perfo-bolts, significantly improved the wall's stability. Back-analysis conducted after four decades confirmed that these measures effectively increased safety margins against both sliding and overturning. The results, supported by two-dimensional stability assessments, demonstrate the wall's continued performance and long-term reliability. This case underscores the importance of detailed geological and geotechnical investigations, especially in areas with complex structural geology. It also emphasizes the value of anticipatory design approaches that account for potential subsurface challenges. Furthermore, the study shows that classical engineering solutions remain highly relevant, particularly when modern monitoring tools are limited or unavailable. Although the analysis was based on 2D methods and historical performance records due to limited instrumentation and real-time monitoring the findings still offer important lessons. They support the integration of forensic evaluation frameworks in future infrastructure projects. Such frameworks can help assess existing structures, guide adaptive design strategies, and proactively address potential stability issues in geologically sensitive environments.

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