Appendix A

History of Construction

December 2019

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

This Appendix presents the construction history of the Vale S.A. ("Vale") Córrego do Feijão Mine Dam I ("Dam I") in Brumadinho, Brazil and of certain relevant features near Dam I. The appendix describes the site and surrounding area, as well as the design and construction of the dam and its appurtenant structures. It also summarizes significant operations procedures that were used to manage the tailings and key events that may inform an understanding of the structure and characteristics of the dam.

The data and descriptions provided in this Appendix are based on documents and records obtained from Vale and/or third parties. In some cases, certain information was not available, which is understood to be due to multiple factors, including the more than 40-year history of Dam I, Vale's acquisition of the dam from a previous owner, Ferteco Mineração S.A. ("Ferteco"), in 2001, and the loss during the failure of certain paper records stored in an office near Dam I. Nonetheless, the available information is sufficient to provide a description of the major features of the Dam I design and construction.

2. FACILITY DESCRIPTION

2.1 <u>Location</u>

Dam I is part of the Córrego do Feijão Mine at the Paraopeba Complex located in Brumadinho, Minas Gerais, Brazil. The site location is presented in Figure 1.

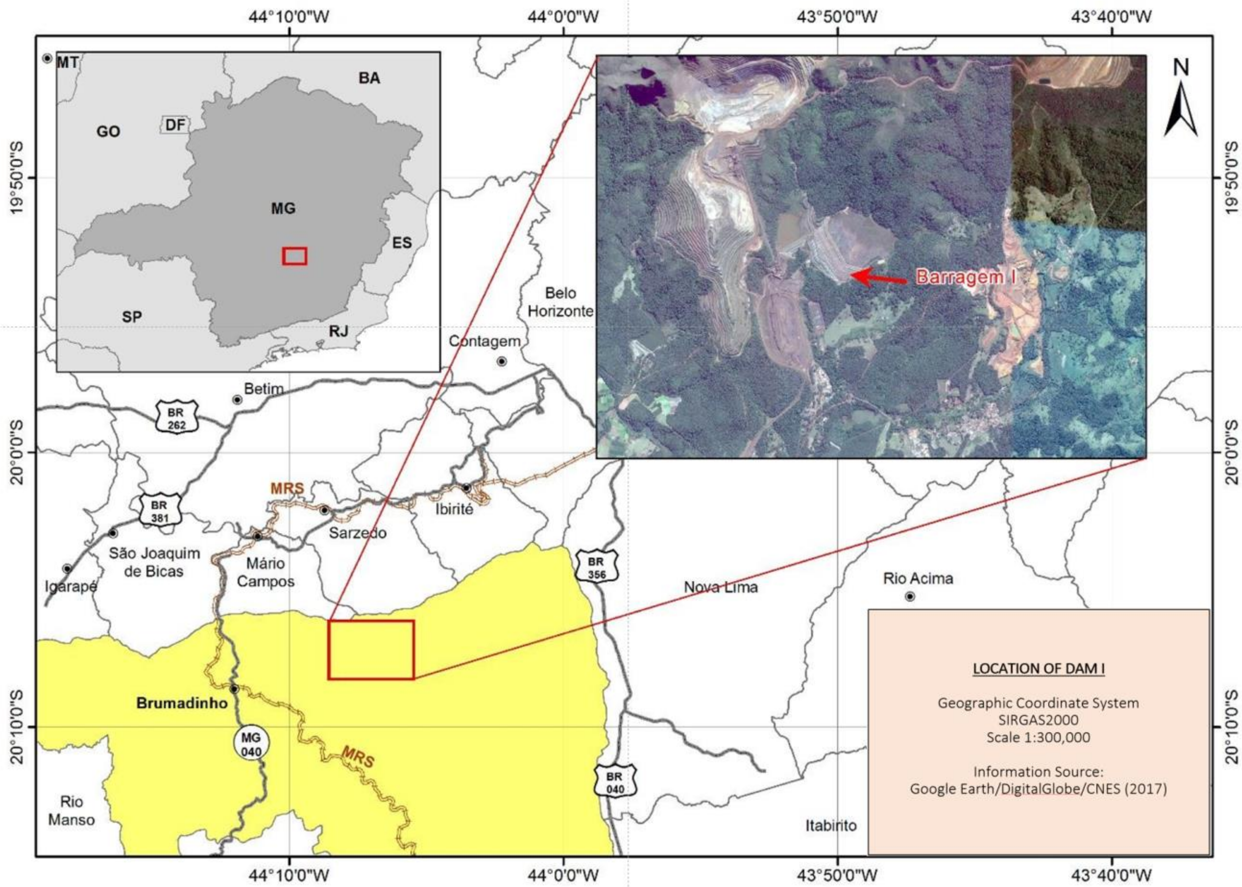

Figure 1: Location Plan for Córrego do Feijão Mine[1]

2.2 Facility Overview

The Córrego do Feijão Mine, including Dam I, was owned and developed by Ferteco until April 27, 2001, when Vale acquired Ferteco. Figure 2 presents an aerial view of the Córrego do Feijão Mine facility. Dam I and Dam VI are identified in Figure 2. At the time Vale purchased Ferteco in April 2001, the dam had been raised to have a crest elevation (El.) 916.5 meters above mean sea level (m msl), with a maximum height of 60.5 m, to the configuration discussed in Section 5.8 of this Appendix (i.e., Sixth Raising).

The byproducts of mining included waste rock and tailings, which were produced during crushing of the unprocessed ore and the subsequent ore concentration process. For the ore concentration process, water was used to assist in the gravity separation of ore particles from the rejects (i.e., tailings). This water was also used to transport tailings through a system of sluice pipes to the Dam I area. Although the unprocessed ore was subjected to the concentration

[1] Periodic Review of Dam Safety of the Córrego Feijão Mine – Dam I Technical Report (TÜV SÜD 2018) ("2018 TÜV SÜD Periodic Safety Review") (translated from original Portuguese).

process, varied amounts of ore still remained present in the tailings. The tailings dam itself was developed to dewater the sluiced tailings and to store them.[2] The Dam I reservoir also served to reduce turbidity and improve the quality of outflows.[3]

In July 2016, a dry method of ore processing operations began within the Córrego do Feijão Mine ore-treatment plant, eliminating the need to process ore using water. At that time, Dam I ceased to receive tailings.

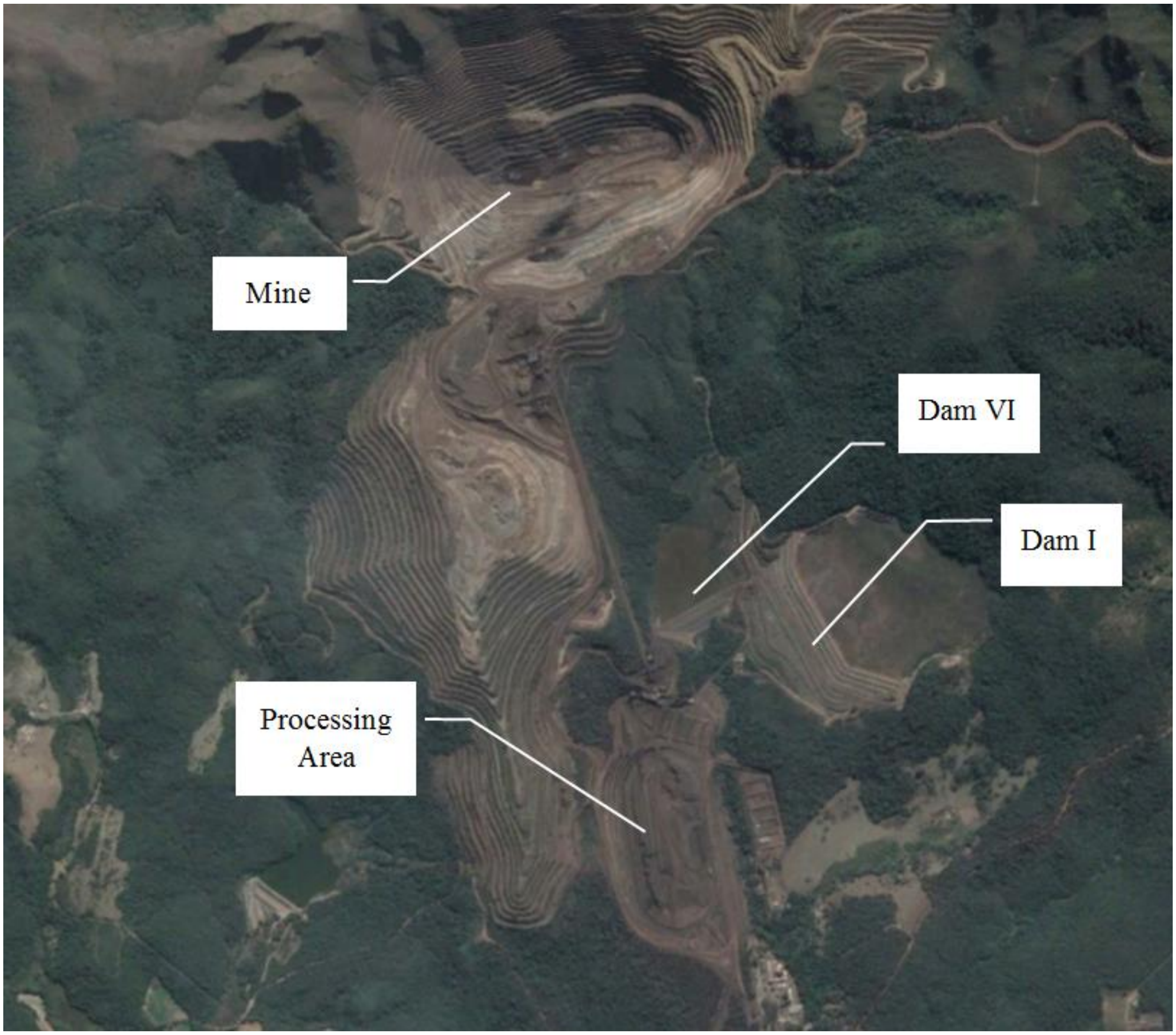

Figure 2: Córrego do Feijão Mine[4]

2 2018 TÜV SÜD Periodic Safety Review.
3 Supplementary Technical Review – Stability Analysis Under Undrained Loading Conditions (Geoconsultoria).
4 Google Earth. (n.d.). Aerial view of Córrego do Feijão Mine. From July 2018. Text and graphics added by authors.

3. SETTING

A summary of the regional and local geological conditions, based on information provided in the initial design documents from the First Raising of the dam[5] and reports of regional geology of the area,[6] is presented below.

3.1 Geology

3.1.1 Regional Geology

The Córrego do Feijão Mine is located along the alignment of the Serra do Curral mountain range, which delimits the northern section of the Quadrilátero Ferrífero, a region rich in iron ore. Except for a small rupture movement in the Cretaceous geological period, only weathering and erosion have been recorded in this region. The southwestern segment of the Serra do Curral is made up of a thick lithological sequence of the Minas Supergroup, which is made up of quartzites and phyllites of the Caraça Group, a ferrous formation of the Itabira Group, and metapelites of the Piracicaba Group. The Córrego do Feijão Mine, where Dam I is located, is situated in an area dominated by residual gneiss soils and lateritic colluvial soils.

3.1.2 Local Geology

Dam I was constructed in an area underlain by banded gneisses bedrock, which is overlain by saprolite, residual, and colluvial soils. These soils displayed good bearing capacity and low permeability.[7] Historical information regarding the geotechnical parameters for the in situ foundation soils is limited. In general, foundation soils consist of silty-clay material that were overlain by weathered soils.[8] Dam I was constructed within the streamshed of the Córrego do Feijão. The side valley (Córrego da Pedra Grande) narrows at the dam location due to a ridge consisting of hard rock that protrudes from the valley floor, referred to as the "Kanga Ridge." Fine silty sand on the valley floor and lateritic soils along the valley slopes join directly to the rock ridge, which is weakly fragmented and allows seepage.

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[5] Design Documents, First Raising (Christoph Erb 1976) ("Erb Design Documents") (translated from original German).

[6] Alkmim, F.F., & Marshak, S. (1998). Transamazonian orogeny in the Southern São Francisco Craton Region, Minas Gerais, Brazil: Evidence for paleoproterozoic collision and collapse in the Quadrilátero Ferrífero. *Precambrian Research*, *90*(1-2), 29-58; Doerr, J.V. (1969). Physiographic, stratigraphic and structural development of the Quadrilatero Ferrifero Minas Gerais, Brazil (U.S. Geologic Survey Paper 641-A). Washington, DC: U.S. Government Printing Office.

[7] 2018 TÜV SÜD Periodic Safety Review; Dam I Tailings Recovery Plan (Vale 2010).

[8] 2018 TÜV SÜD Periodic Safety Review; Erb Design Documents.

3.2 Hydrogeology

Groundwater in the region exists in the bedrock, weathered rock, and residual soils. Groundwater in areas having the geology described in Section 3.1 typically is recharged from precipitation and flows to surface drainage features (e.g., creeks) from underlying rock and soils. In the area of Dam I, groundwater likely flowed into the Feijão creek from the underlying bedrock and soils. The construction of Dam I and the impounded tailings likely eliminated the creek as a discharge point for groundwater, resulting in an increase in groundwater elevations in the vicinity of the dam and tailings as development proceeded. Further, the water in the tailings is likely hydraulically connected to groundwater in the area, limiting the potential for dewatering of the tailings by drainage downward into the ground. Drainage of the tailings would likely have occurred only through lateral drainage through the dam, not vertically downwards into the underlying bedrock and soils, as well as by evaporation from the surface.

3.3 Hydrology

Dam I is located within an 84.4-hectare (ha) drainage basin of the Córrego do Feijão.[9] The impoundment behind the dam covers an area of approximately 25 ha, or about 30% of the entire contributory catchment area of the dam. The ground surfaces of this mountainous area slope steeply towards the impounded area of the dam. About two thirds of the catchment area is covered by dense vegetation, and the remainder is covered by impounded tailings and vegetative undergrowth. Precipitation data for the site is provided in Appendix C.

4. OVERVIEW OF DAM I

4.1 Dam Description

Dam I was constructed in 15 stages (10 raisings, with early raisings constructed in stages) between 1976 and 2013. It was constructed primarily using the upstream method of construction, in which the dam is raised in stages by constructing berms on top of previously deposited and dewatered tailings. This method results in movement of the upstream crest of the dam over time, as shown in Figure 3, which increases the tailings retention capacity of the dam. Using this method, the dam was constructed to a maximum height of 86 m, with a final crest elevation of 942 m msl and a crest length of 720 m.[10] The 15 stages and 10 dam raisings are identified in Table 1. The term “raising” was used by Vale and the design engineers listed in Table 1 to denote certain periods of dam development, some of which involved one or more stages of vertical berm construction events. The correlation between raisings and stage(s) are

[9] 2018 TÜV SÜD Periodic Safety Review.
[10] 2018 TÜV SÜD Periodic Safety Review.

identified in Table 1, and the raisings and stages for Dam I are illustrated in the cross-sectional view in Figure 3.

The materials used to construct the dam stages were generally obtained from the tailings retained by the dam itself. As discussed in Section 5, the construction materials were intended to be obtained from the coarse-grained fraction of the tailings that had previously been deposited in the facility. According to available documents, discussed below, constructed dam stages were comprised of materials having permeability ranges of 10^{-8} to 10^{-7} m/s, with some as low as 10^{-9} m/s.

The operation of the tailings disposal process was designed to result in the segregation, by hydraulic settling, of the coarse fraction from the fine-grained fraction to isolate the coarse-grained tailings for use in future berm stage construction.[11] This construction was to be accomplished by discharging the sluiced tailings from the top of the dam crest onto the upstream side of the berm, with the intent of the coarse tailings fraction settling near the berm and the fine-grained fraction flowing with the sluice water to a lower elevation some distance from the berm (see pond location in Figure 4). The area of coarse material deposition near the dam that was expected to result from this process is referred to as the "beach." The beach properties are highly dependent on the maintenance of a sufficient height of the dam (i.e., too low a height would result in impounded water and fine-grained material deposition near the dam instead of the formation of a coarse-grained beach) and the operation of the sluice process (e.g., velocity of discharge, height of drop from the sluice pipe to the beach, etc.).

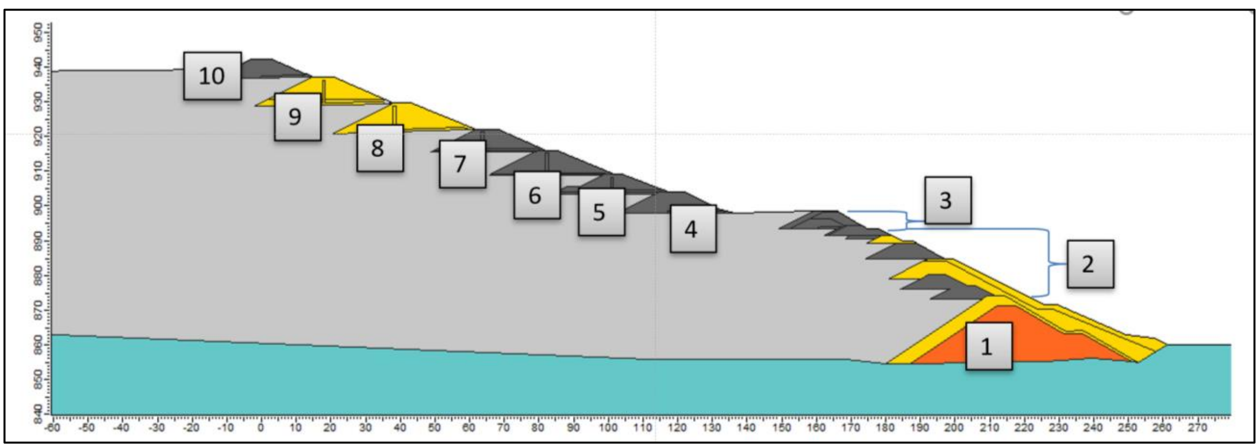


Figure 3: Dam I Cross-section Showing Raisings and Stages of Construction[12]

[11] Construction Specifications, Second Raising (Tecnosan 1981) ("Tecnosan Second Raising Construction Specs").

[12] 2018 TÜV SÜD Periodic Safety Review. Numbering added by authors.

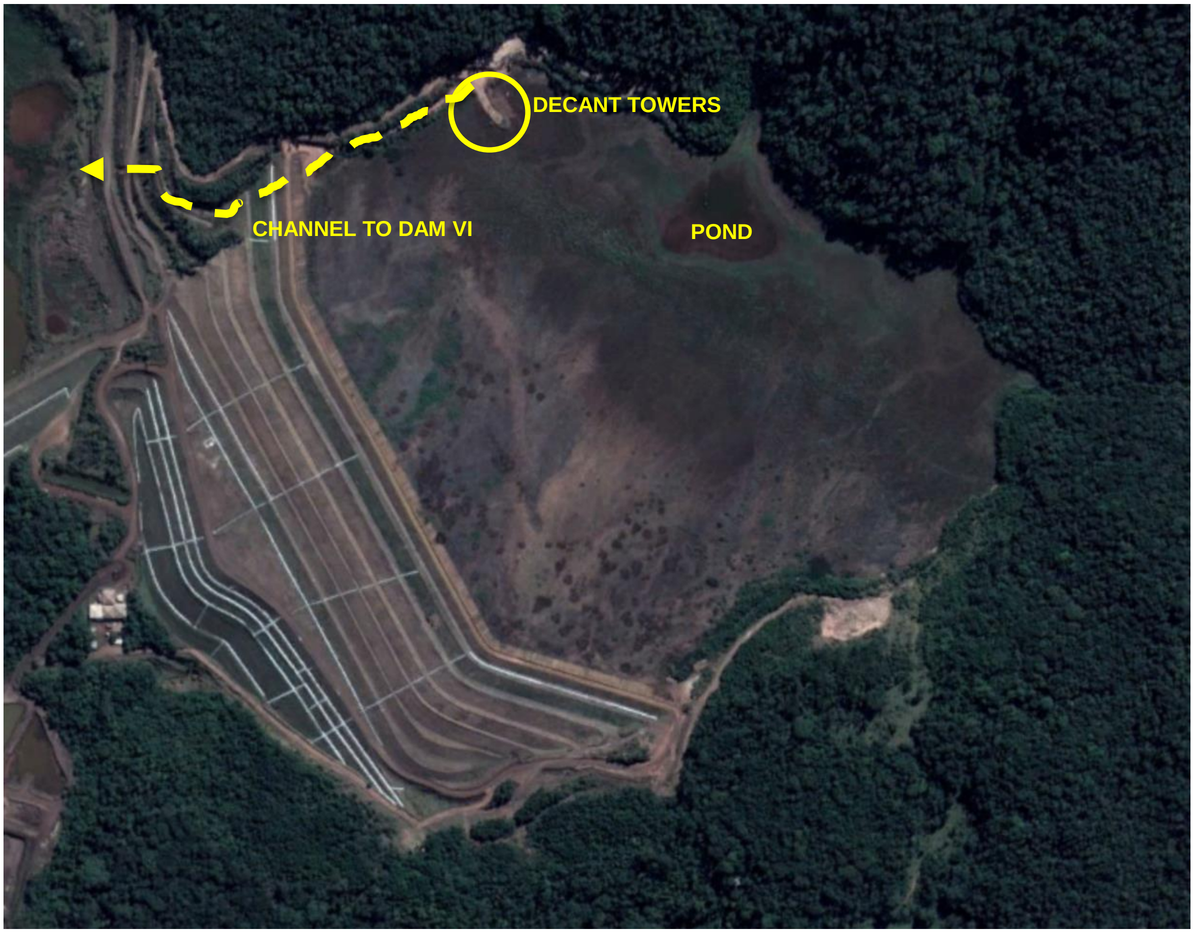


Figure 4: Aerial view of Dam I[13]

[13] Google Maps. (n.d.). Aerial view of Dam I. From June 2017. Text and graphics added by authors.

Table 1: Dam I Construction Details[14]

Stage	Year	Raising ID	Top El. (m msl)	Height max (m)	Project Design Firm	Construction Company
1	1976	Starter Dam (First)	874	18	Christoph Erb	Emtel
2	1982	Second	877	21	Tecnosan	Tercam
3	1983		879	23		Unknown
4	1984		884	28		Construtora Sul Minas
5	1986		889	33		Unknown
6	1990		891.5	35.5		Unienge Com. e Constr. Ltda.
7	1991	Third	895	39	Chammas Engenharia	Construtora Sul Minas
8	1993		899	43		Unknown
9	1995	Fourth	905	49	Tecnosolo	CMS Constr. S.A.
10	1998	Fifth	910	54		U & M
11	2000	Sixth	916.5	60.5		Constr. Dragagem Paraopeba
12	2003	Seventh	922.5	66.5		Construtora Impar Ltda.
13	2004	Eighth	929.5	73.5		Integral
14	2008	Ninth	937.0	81.0	Geoconsultoria	Integral
15	2013	Tenth	942.0	86.0	Geoconsultoria	Salum Enga.

4.2 Water Management Features of the Dam

As noted in Section 2.2, the function of Dam I was to store the tailings with the intent that the dam and dewatered tailings would be a permanent and stable disposal site for the tailings. This depended upon the management of the sluiced water and other water (i.e., run-on, precipitation, and groundwater inflow) so that the water did not destabilize the dam structure. As described in

14 Modified from 2018 TÜV SÜD Periodic Safety Review (translated from original Portuguese).

general in Section 2.3 and in more detail in Section 5, materials and operational practices were specified throughout the dam's development history for draining the water from the tailings deposited near the dam without destabilizing the dam. Water management practices specified during design and construction involved both *surface water* management and *tailings water* management.

- The *surface water* management features for Dam I generally consisted of a series of lateral and vertical surface canals that were designed to route the surface water run-on, precipitation, and sluiced water to the Córrego do Feijão beyond the toe of the dam. The purposes of the surface water management system included preventing overtopping of the dam and promoting proper development of the tailings beach (discussed in Section 4.1).

- The *tailings water* management features for Dam I were designed to release the tailings-laden sluice water into the impoundment area to create favorable conditions for water drainage, both vertically and laterally, from the tailings and thereby promote proper formation of the beach against the dam (discussed in Section 4.1). The formation of an exposed beach would result in a coarse layer of relatively high-permeability material directly behind the upstream face of the dam, promoting consolidation and desiccation to support the next raising and also providing a source of coarse-grained material for the construction of the next berm stage. Due to the fact that it forms as a slope (having a grade, ideally, of several percent), the beach would also promote the flow of sluiced water or precipitation laterally towards the pond. Tailings water management also included the construction of lateral drains and subsurface drains in the raisings, typically as drainage blankets of higher permeability material at the base of the raisings, to route water from the retained tailings to the dam's surface to prevent buildup of hydraulic pressure. Flows from these drainage features were collected in surface drainage channels and pipes, and were monitored by means of flow meters. The measurements obtained using flow meters are discussed in Section 7 of Appendix C, and subsurface drains within the dam structure are discussed in Section 5 for each stage of dam construction.

As the height of the dam was raised, there was a need to direct water from the impoundment to the stream at the toe of the dam. To facilitate this passage of water, a channel was constructed around the right abutment of the dam. Prior to the construction of the Ninth Raising, a culvert was constructed in natural soils adjacent to the Eighth Raising berm to transmit this water, and the Ninth and Tenth Raising berms were constructed over this culvert. Also, at that time, three intake towers were constructed at the right abutment of the dam to control and direct flows to the channel and, eventually, into the impoundment behind Dam VI.

5. DAM I CONSTRUCTION

5.1 Preexisting Conditions

As discussed in Section 4.1, the construction of Dam I began in 1976. At that time, the site was undeveloped and had tropical features and a mountainous topography.[15] The topography of the site before construction began is shown in Figure 5.

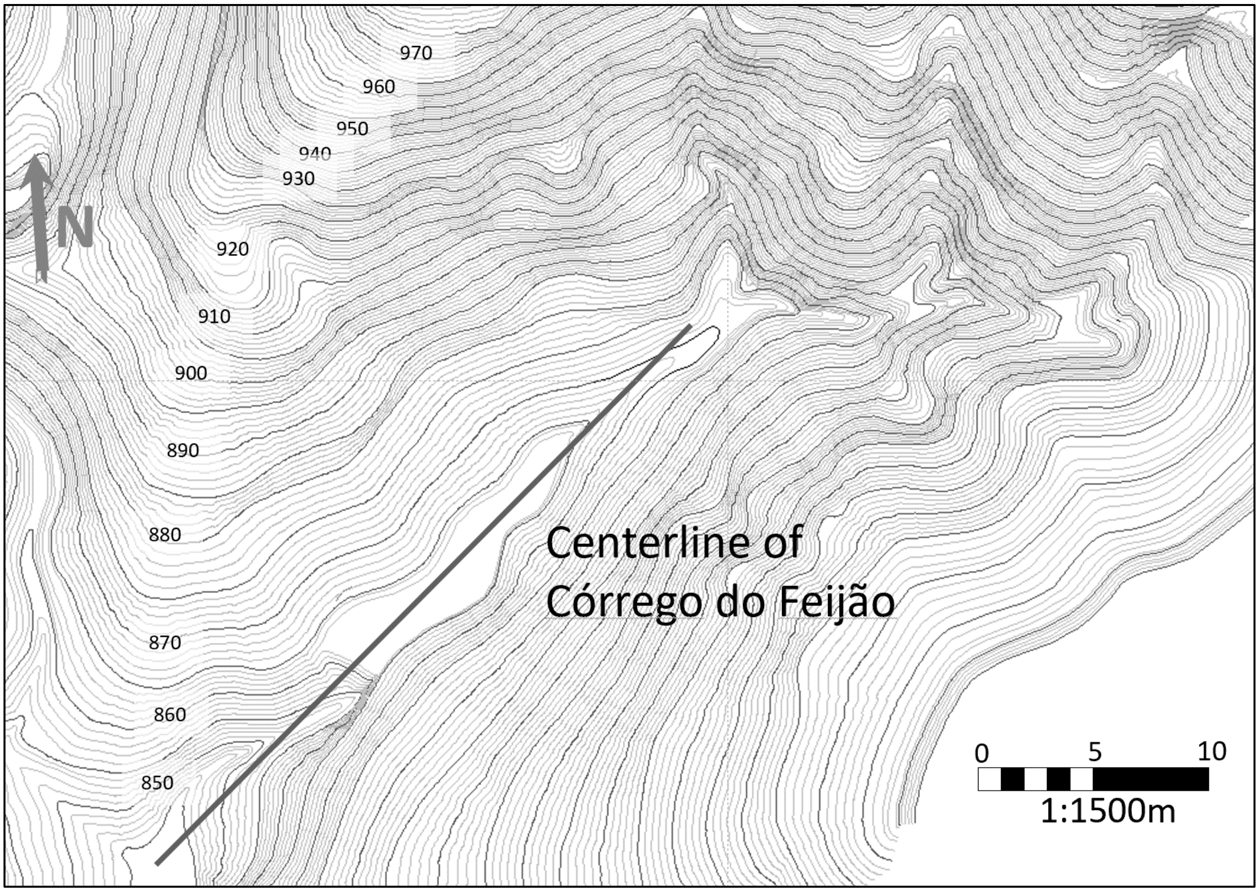


Figure 5: Topography in Dam I Area Before Construction[16]

Documents describe the site before it was developed as a valley having a creek flowing through the valley floor. The valley floor is further described as being about 30 m wide and having marshy conditions resulting from the relatively flat slope of the creek in that area; the marshy area is shown in Figure 6 along the alignment of the creek. The valley slopes are characterized

15 The pre-construction site features are discussed in the Erb Design Documents and in the inspection reports for the site preparation for initial construction prepared by Engosolo. Site Inspection Reports (Engesolo 1976).

16 Design File Topography Map of Dam I. Text and graphics added by authors.

as being covered "without exception" by stiff laterite "red earth" soils having a high proportion of silt. The description further suggests that the laterite soils were covered with a layer of broken quartz and ore rubble having a thickness of less than 0.3 m. The site was characterized as good for building an impoundment because the surface was covered with water-holding cohesive soils having a thickness of at least 0.5 m.[17]

A key feature of the site before development began was the presence of an outcropping of hard rock, referred to as the "Kanga Ridge" and shown just to the south of the "Dam Axis" label in Figure 6. The ridge was described as moderately to weakly fragmented and permeable.[18] The Córrego do Feijão creek flowed through the ridge at a narrow cut, leaving the ridge as a natural "support structure" feature along which to build Dam I.

[17] Erb Design Documents.
[18] Erb Design Documents.

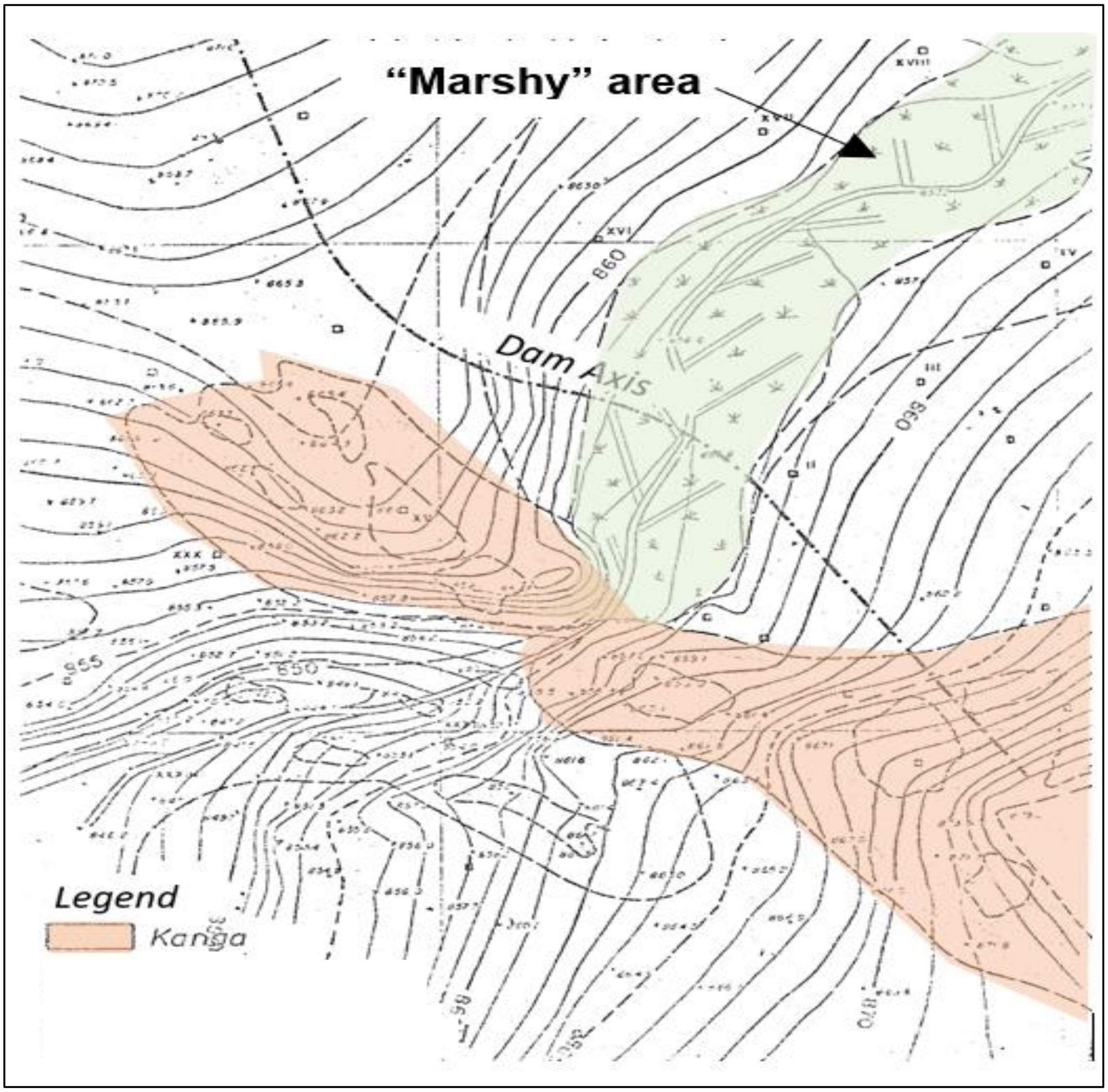
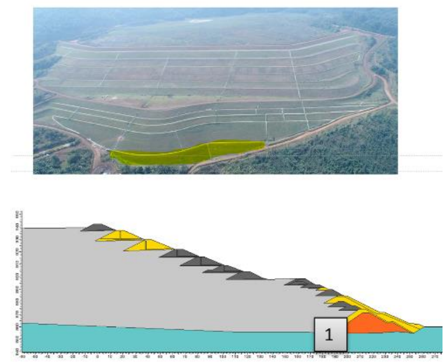


Figure 6: Site Conditions Before Dam I Construction[19]

19 Erb Design Documents. The colored areas identified in Figure 6 as "Kanga" and "Marshy Area" were added based on the descriptions in the Erb Design Documents; the figure itself was not available in a resolution that allowed correlation between the legend and the figure contents.

5.2 First Raising: Starter Dam, 1976



The First Raising (location shown in the index figure to the right), also referred to as the "Starter Dam," was constructed in 1976. It was designed by Christoph Erb in 1975 to create a dam with a maximum height of 18 m and having a crest elevation of 874 m msl. Documents available from the First Raising do not include a formal design plan. However, several documents are available that describe the design and construction of this dam.[20] In general, they include descriptions of the site conditions before construction began (Section 5.1), the process of selecting the site and designing the Starter Dam, the design of the dam itself, and observations made during the construction of the dam.

5.2.1 Geotechnical Investigations

Prior to the construction of the Starter Dam, a geotechnical investigation was performed consisting of test pits and laboratory tests of potential dam construction materials. Available laboratory test results on potential dam construction materials consisted of grain-size tests on five samples of "sand-sized ore grains" obtained from the Feijão mine processing operation. During construction, numerous test pits and boreholes in the area of Dam I were performed.[21] These investigations generally confirm the characterization of the conditions of the site prior to Dam I construction provided in Section 5.1.

5.2.2 Design Approach

The design approach involved the construction of a dam across the Córrego do Feijão creek bed and to the northeast of the exposed rock "Kanga Ridge" shown in Figure 6. The design included assessments of the dam's surface water management system, stability, internal drainage, and construction specifications, as described below.

5.2.2.1 Geometry

The Starter Dam was designed to a maximum height of 18 m and a 5-m wide crest at El. 874 m msl. The upstream slopes were designed to have a slope of 1.5 horizontal to 1 vertical (1.5H:1V), and the downstream slopes were designed to have a slope of 1.75H:1V. An approximate 4-m thick layer of laterite soil was designed for the upstream slope of the dam

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[20] Erb Design Documents; Site Inspection Reports (Engesolo 1976).
[21] Erb Design Documents.

(Figure 7), which was to be embedded approximately 1.2 m into the bordering natural cohesive soils at the bottom of the Starter Dam to form a seepage cutoff wall.

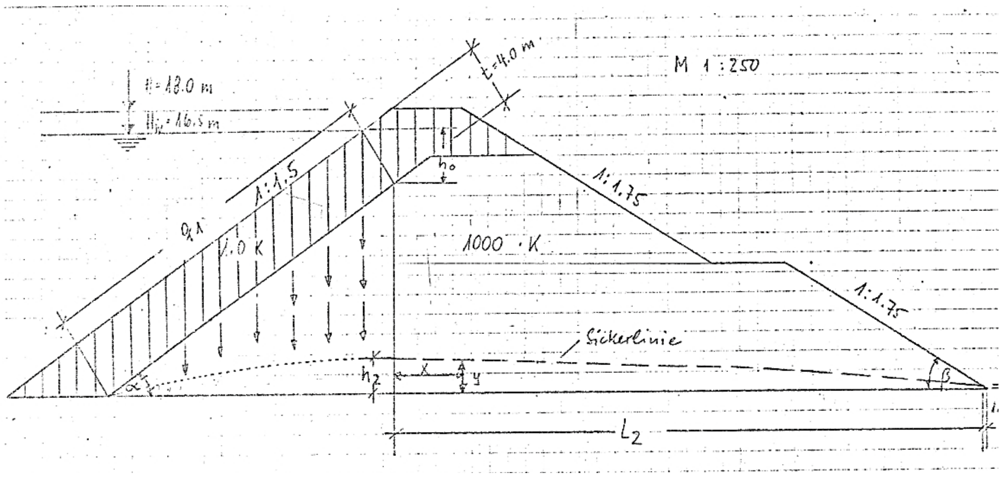

Figure 7: Conceptual Design Cross-section for Dam I[22]

5.2.2.2 Internal Drainage

The design of the Starter Dam did not include any internal drainage system features. Regarding the ability of the Starter Dam to transmit seepage, the designer noted that the low permeability coefficient of the laterite compared to the permeability of the tailings should limit the seepage that could be expected through the core of the dam,[23] suggesting that any seepage that flowed through the 4-m thick laterite soil layer on the upstream slope of the dam would flow through the tailings of the dam core without buildup of water. This is illustrated in Figure 7, which shows a presumed seepage line at a very low level in the Starter Dam. No structural feature appears to have been included in the design of the dam's downstream toe to facilitate drainage of seepage from the core to prevent erosion or piping issues.

The dam's design also required the construction of a layer of laterite soil on the downstream slope. This feature was judged necessary to retain the tailings fill of the embankment, to provide a suitable layer for growth of vegetation, and to prevent erosion of the downstream face of the berm. There is no record of the design or construction of drainage features to transmit seepage from within the core of the Starter Dam through this downstream laterite layer.

22 Erb Design Documents.
23 Erb Design Documents.

5.2.2.3 Surface Water Management

The design called for the discharge of the surface water entering the impoundment area from both the catchment area and the sluice discharge pipes to Córrego do Feijão creek through a channel constructed at the northwest valley slope at the right abutment of the dam.[24] The channel was designed to convey all the water expected to flow into the impoundment. The drainage area of the catchment basin was estimated to be 1,050 square kilometers (km^2), which correlates well with the area estimated in later studies. Surface drainage controls consisted of a berm which was constructed on the downstream dam slope at an approximate elevation of 864 m msl (10 m below the crest of the Starter Dam). Additionally, a trapezoidal channel 150-m long, 3-m wide having an approximate 0.2% slope was constructed on the Starter Dam through which surface water was discharged from the impoundment to the original Córrego do Feijão creek bed at the toe of the dam.

5.2.2.4 Selected Material Parameters

Material parameters for the tailings and laterite were provided in the initial design and are presented in Table 2. The Starter Dam was designed to be constructed of free draining "ultrafine" ore (i.e., tailings) from the Feijão mine. The anticipated tailings parameters were based on tests performed on samples from the offsite Fábrica pit, and the laterite parameters were based on test samples collected from the Forquilha settling pond of the Fábrica pit.

Table 2: Starter Dam Material Parameters[25]

Material	Specific Weight (kN/m^3)	Friction Angle (°, ϕ)	Cohesion (kPa)	Permeability (m/s)
Ultrafine Ore	25	35	0	1.2×10^{-6}
Laterite	20	29	30	1.2×10^{-9}

5.2.2.5 Geotechnical Stability

The upstream and downstream slopes of the Starter Dam were evaluated for geotechnical stability. The design configuration was reported to be stable under static conditions with factors of safety (FS) of 1.53 or greater, which was considered by the designer to be in compliance with relevant standard practice at the time. The importance of the compaction of tailings and laterite was discussed in the design documents and described as a requirement for achieving the required level of stability.

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24 Erb Design Documents.

25 Erb Design Documents; 2018 TÜV SÜD Periodic Safety Review.

5.2.2.6 Construction Specifications

Construction specifications were provided within the text of the design report.[26] The report specified compaction requirements for the tailings and laterite. Compaction was specified at 100% of the standard Proctor maximum dry density ("standard Proctor"), except for the downstream side of the Starter Dam where 95% standard Proctor compaction was considered acceptable for the laterite.

The design documents specified the following regarding site preparation:

- all vegetation in the Dam I and surrounding areas must be removed, including the layer of topsoil and any organics;
- weak surface soils need to be dried and drained (with trenches) before clearing;
- the rock along the "Kanga Ridge" must be cleared, and cracks in the rock face should be filled with sand to prevent internal erosion; and
- the Starter Dam should be sealed on the upstream face with laterite soil that is keyed into the surrounding soils.

There was no mention of any preparation of, or improvement to, the base of the impoundment area to achieve any particularly high or low permeability.

5.2.3 Complications and Variances

As-built information is not available for the Starter Dam. However, the design documents from the Second Raising, which provide a summary of the Starter Dam, do not identify any variances from the planned design.[27]

Testing and inspections were performed during construction, which generally show that reported test results met the requirements of the construction specifications.[28] No information was provided regarding the sources of construction materials, their parameters, nor whether they met the requirements or design assumptions. The testing reports include records of test boreholes and accompanying laboratory geotechnical tests. Among other things, these reports note:

- the presence of the wet surface within the valley (January 20, 1976); and
- the performance of soil compaction along the rock (i.e., the Kanga Ridge) using mechanical equipment (February 6, 1976).

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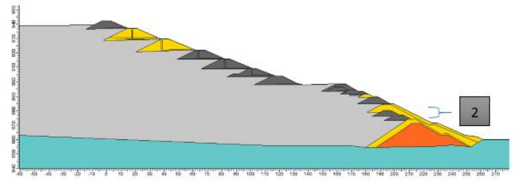
[26] Erb Design Documents.
[27] Design Documents, Second Raising (Tecnosan) ("Tecnosan Second Raising Design Documents").
[28] Site Inspection Reports (Engesolo 1976).

5.3 Second Raising: Initial Design, 1982 through 1983



The Second Raising (location shown on the index figure to the right) was constructed between 1982 and 1990. The initial Second Raising design called for raising Dam I by 15 m from El. 874 m msl to a crest elevation of 889 m msl. However, after the dam was raised to El. 892 m msl, the design was changed, as discussed in Section 5.4. In this section, only the initial design of the Second Raising is discussed. Various documents describe the design and construction of the initial design of the Second Raising,[29] including (i) the initial and revised designs, slope stability analyses, and construction technical specifications;[30] (ii) reports of the construction inspection;[31] and (iii) drilling and laboratory testing reports performed during the Second Raising.[32] The initial design of the Second Raising was produced in 1980 by Tecnosan (discussed in this Section 5.3), and a revised design was produced in 1983, also by Tecnosan (discussed in Section 5.4).

5.3.1 Geotechnical Investigation

Geotechnical field investigations are not reported to have been performed before the development of the Second Raising initial design. However, in 1980, laboratory tests were performed on tailings to estimate the following geotechnical parameters: unit weight, grain size distribution, consolidation parameters, friction angle, and permeability.[33] Test results reported for the tailings are provided in Table 3. The locations from which these samples were obtained were not reported, and so it cannot be known whether these materials were planned for use as dam construction material. Tecnosan also mentions the results of direct shear tests performed on two tailings samples, which reported estimated friction angles of 29.5° and 31.5°, but for which no laboratory test reports are provided. Laboratory consolidation tests also were reportedly performed on the tailings samples, but values from the tests do not appear to have been included in the available documents.

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[29] Tecnosan Second Raising Design Documents.
[30] Tecnosan Second Raising Construction Specs.
[31] Second Raising Construction Inspection Documents.
[32] Pavisolos & Sondag Geotechnical Investigation Reports 1 and 2.
[33] Tecnosan Second Raising Design Documents.

Table 3: Laboratory Test Results on Tailings Samples (Second Raising, Initial Design)[34]

Sample	Actual Density of Grains (Specific Gravity) (g/cm³)	Grain Size Distribution					Coefficient of permeability (m/s)
		Clay	Silt	Fine Sand	Medium Sand	Coarse Sand	
A	4.90	-	7.0	90.0	3.0	-	3.43×10^{-6}
B	5.00	-	7.0	85.0	8.0	-	9.47×10^{-6}

Laboratory tests also were performed on the drainage layer material called "sinterfeed."[35] Sinterfeed was used for the construction of the drainage blankets underlying many of the raisings. According to the initial design, the two stages that would have used sinterfeed filters were the Second and Fourth stages of the Second Raising (Figure 8). A gradation curve was provided by Tecnosan for the sinterfeed (Figure 9a) and tailings (Figure 9b), indicating that the sinterfeed has the gradation of a fine sand to coarse silt. However, no other description of the sinterfeed nor the location or nature of the source of the sinterfeed were found in the initial design documents for the Second Raising.

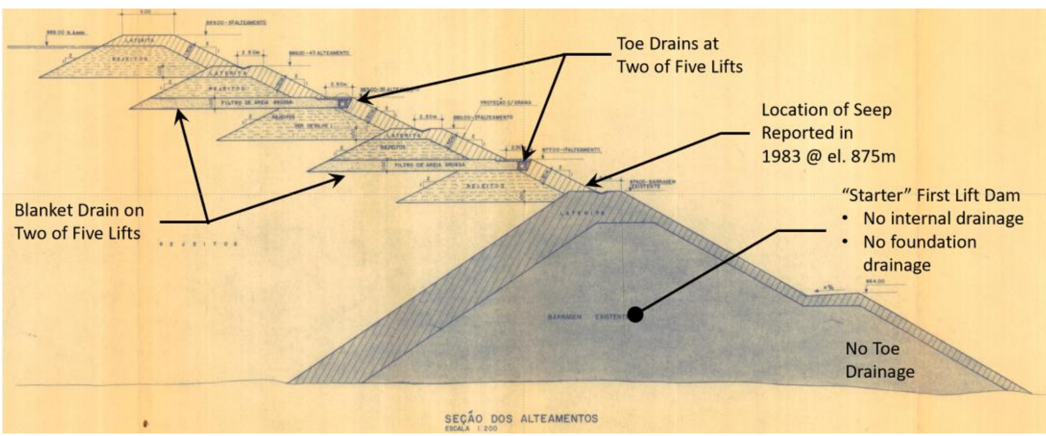

Figure 8: Second Raising Cross-section (Initial Design)[36]

34 Modified from Tecnosan Second Raising Design Documents (translated from original Portuguese).
35 Tecnosan Second Raising Design Documents.
36 Tecnosan Second Raising Blueprint. Text and graphics added by authors.

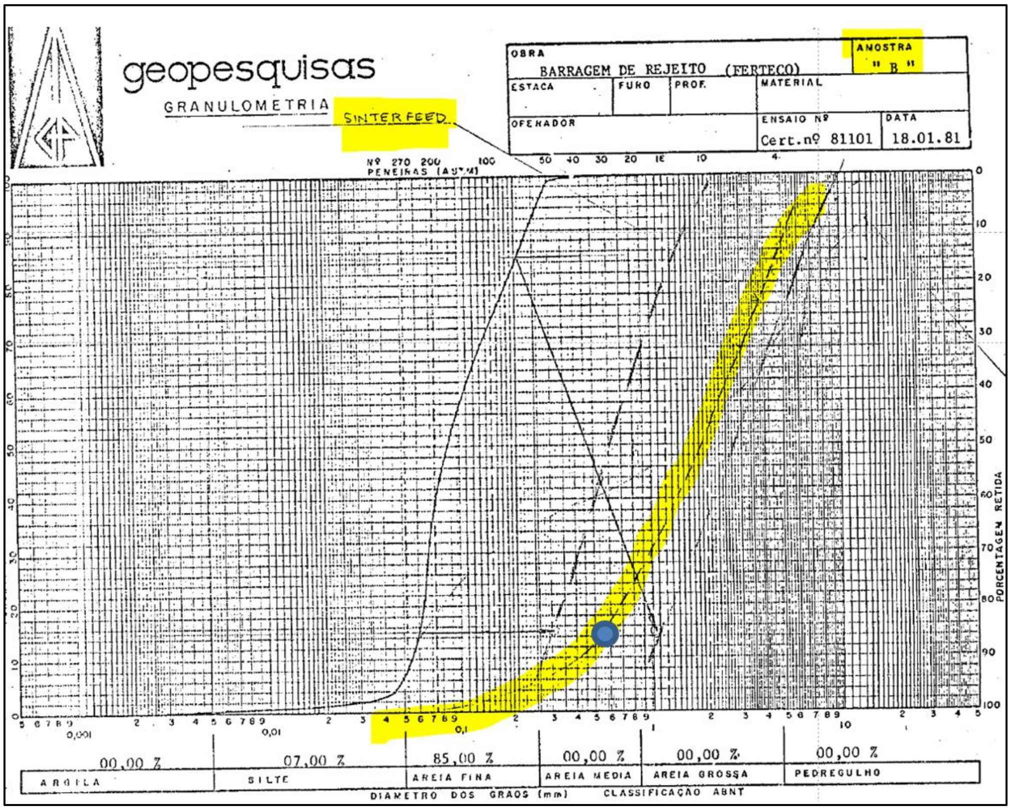

Figure 9a: Sinterfeed Gradation Curve[37]

[37] Tecnosan Second Raising Design Documents (highlighting added).

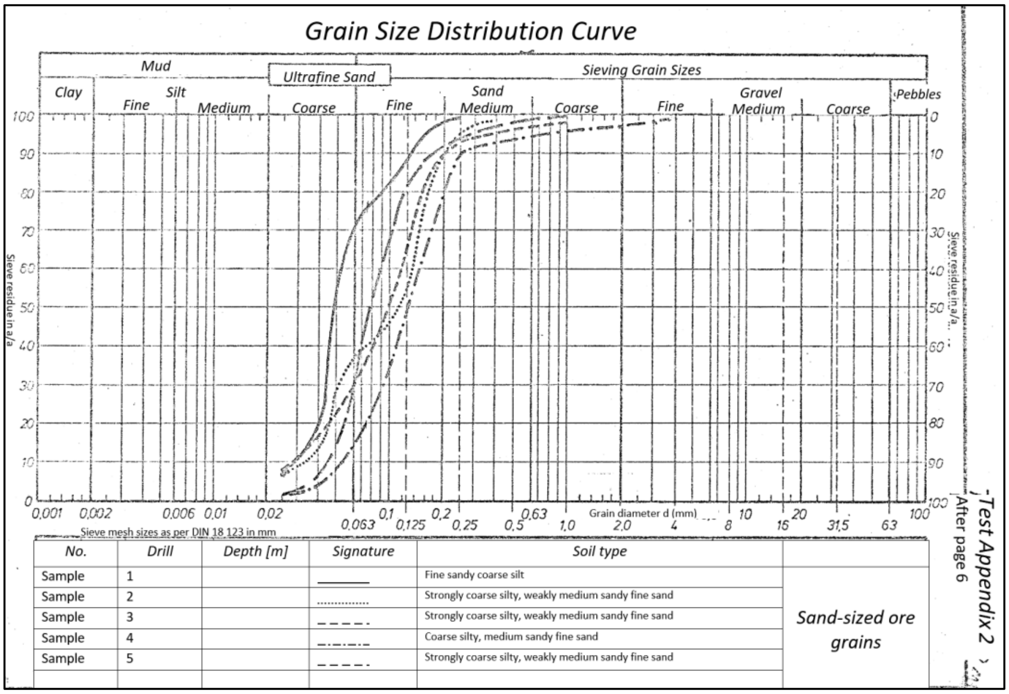

Figure 9b: Tailings Gradation Curves[38]

5.3.2 Design Approach

The initial Second Raising design described a series of five stages, each having a height of 3 m, to raise the dam to a crest elevation of 889 m msl using the upstream construction method.[39] The design cross-section is shown in Figure 8. The design includes stages constructed of compacted coarse tailings, covering of the downstream slope with a 1.5-m thick layer of compacted laterite, and inclusion of horizontal drainage blankets in two of the stages (see internal drainage discussion in Section 5.3.2.2). The design approach addressed the surface water drainage system and decants, the dam itself, and the use of tailings to construct the raising. The design discusses the importance of the formation of the beach to the stability of the dam and the function of the beach in the formation of materials suitable for use in the future stages of dam construction.[40]

5.3.2.1 Geometry

The initial Second Raising design is shown above in Figure 8. The design called for dam upstream and downstream slopes of 2H:1V. Downstream slopes were designed having a 1.5 m-

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[38] Erb Design Documents. Note that the label "Ultrafine Sand" is from the original document and is not a term used in the particle size classification system referenced in the underlying figure.
[39] Tecnosan Second Raising Design Documents.
[40] Tecnosan Second Raising Design Documents.

thick compacted laterite facing. The outer slopes of the stages were to be tied into the slope of the Starter Dam, with channels at the toe of each stage. The base of the berm stages was to be constructed directly on the tailings beach formed by tailings deposited behind the previous stage. The design report indicates that the beach was intended to form the foundation for the overlying stage.[41]

5.3.2.2 Internal Drainage

Internal drainage for the Second Raising initial design was to consist of sinterfeed or crushed stone horizontal filters ("drainage blankets") at the base of the second and fourth of the five planned Second Raising stages.[42] A cross-section of the Second Raising, second step (also referred to in Table 1 as the "Stage 3 berm"), and the drainage blanket are shown in Figure 10. The filter was designed to reduce the hydrostatic pressure behind Dam I. In Figure 10, within the 50-cm (i.e., about 18-inch) thick sinterfeed layer, a longitudinal pipe that slopes towards the abutments was specified. This pipe appears to discharge at intervals to the surface channels on the dam, but this is not clear in the design documents that are available. These discharge pipes appear to correlate to flow meter devices, which were used to measure flows from the drains (see Section 7 of Appendix C for discussion of drain flow instruments and measurements), although the available documents do not provide a clear correlation between these pipes and the location of any specific drain flow measurement device.

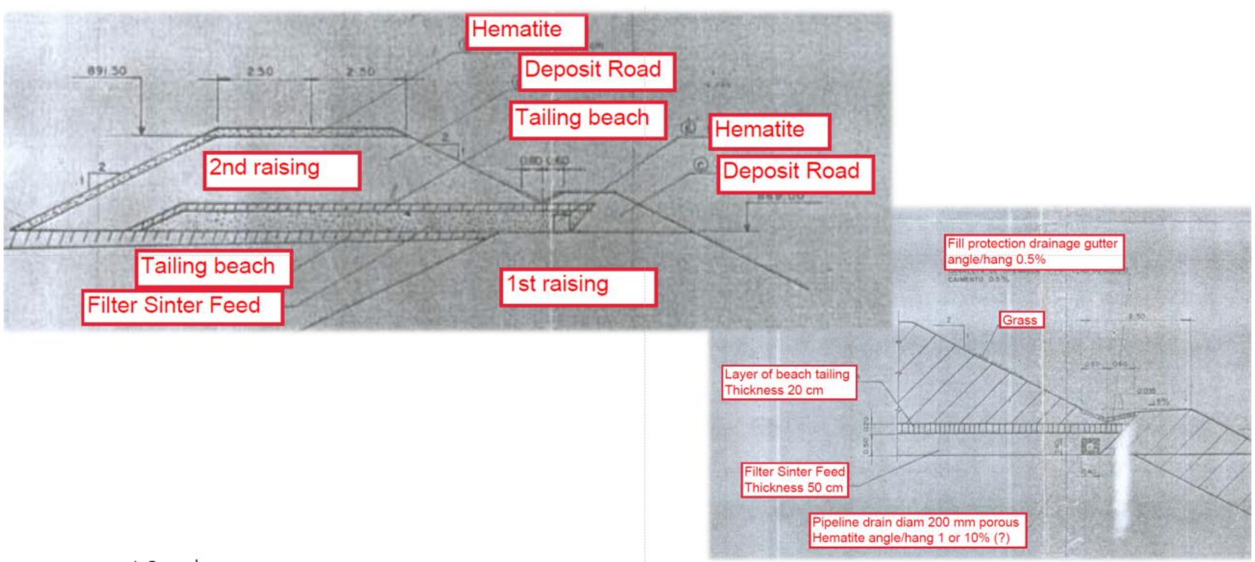

Figure 10: Second Raising Internal Drainage Detail (Stage 2)[43]

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41 Tecnosan Second Raising Design Documents.
42 Tecnosan Second Raising Construction Specs.
43 Tecnosan Second Raising Design Plan Image. Text added by authors.

The design included an assessment of the ability of the sinterfeed drain to filter the tailings material that would be placed over it and a calculation of whether the drainage pipe would filter the rock bedding material (i.e., hematite) around the pipe. The design included a calculation of whether the sinterfeed meets filter criteria ($\mathrm{D}_{15,\mathrm{Sinterfeed}} < 5D_{85,\mathrm{Tailings}}$) for Tailings Sample "B." The calculation results showed that the sample met the filter criteria. However, the filter calculation was not performed for Tailings Sample "A," which appears to fail the filter criteria. Other filter criteria (e.g., uniformity, permeability, segregation, gravel correction for > #4 sieve portion) do not appear to have been checked. The sinterfeed gradation in Figure 9a shows a band of acceptable sinterfeed gradations, but no conformance test results for compliance with this filtration gradation band were found in the documents provided, and no representative study of tailings gradation was found from this period. For the pipe filter calculation (i.e., $\mathrm{D}_{\mathrm{Pipe\ Hole}} < 2D_{85,\mathrm{Itabirite}}$), it is apparent that the result was a recommendation that the stone should have a diameter of 4.8 to 12.5 millimeters (mm), but the intent of this recommendation is unclear.[44]

5.3.2.3 Surface Water Management

Construction of a channel to route decant water out of the impoundment area was discussed in the Second Raising initial design documents, but no formal designs were provided. Along with the previously mentioned laterite designed to protect the downstream slope and crest of the dam, a layer of grass was specified for the downstream slope to prevent erosion.[45]

5.3.2.4 Geotechnical Stability

The geotechnical stability of the Second Raising was evaluated using Bishop's method.[46] Only drained conditions were evaluated, and only a typical cross-section was analyzed. The material parameters used in the design are described in Section 5.3.1. A flow net was constructed to estimate the level of the phreatic surface behind the stages, assuming that the tailings were saturated except where they were drained by the drainage blankets discussed in Section 5.3.2.2. It appears that the analysis assumed that there was no phreatic surface or hydrostatic pressure within the Starter Dam. A FS of 1.7 was calculated for the analyses presented.[47]

5.3.2.5 Construction Specifications

The Second Raising construction specifications called for a minimum relative compaction of 60% for the compacted sand and a minimum degree of compaction of 97% for the compacted laterite with a moisture content within -2/+1% of optimum.[48] No specifications were found for

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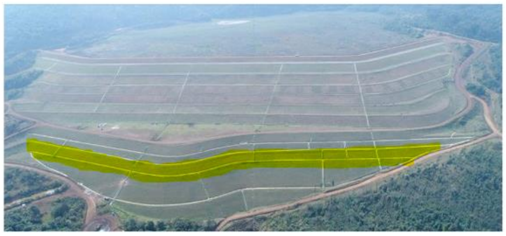
[44] Tecnosan Second Raising Design Documents.
[45] Tecnosan Second Raising Design Documents.
[46] Tecnosan Second Raising Design Documents.
[47] Tecnosan Second Raising Design Documents.
[48] Tecnosan Second Raising Design Documents.

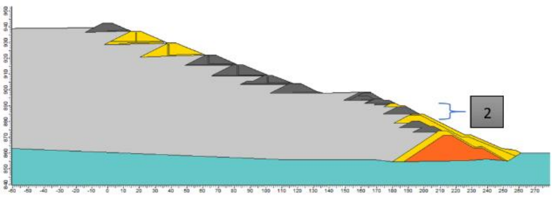
the sinterfeed, hematite, or itabirite materials. Formal specifications for the beach tailings were not provided in the design, although the beach was described as a means to create a coarse fraction of the tailings closest to the dam and the beach was a construction expedient.[49]

5.3.3 Complications and Variances

Actual construction of the Second Raising differed substantially from the original design. The first two Second Raising stages, Stages 2 and 3, were built as planned, using the upstream method as specified in the design.[50] Stage 2 raised the Dam I crest from El. 874 m msl to 877 m msl, and Stage 3 elevated the crest to El. 879 m msl. A number of construction inspection reports were produced that document certain construction aspects of the Stage 2 and 3 berms (primarily the field density tests performed).[51] No information was located regarding material quality control (i.e., gradation of source materials); specifically, no construction-phase tests were provided for the actual sinterfeed, hematite, or itabirite materials used during construction to verify that they met the gradations assumed in the design calculations. In addition, no verification was found that the drainage blanket specified for Stage 3 was constructed as specified.

5.4 <u>Second Raising: Revised Design, 1984 through 1990</u>





The Second Raising design was revised after the construction of Stage 3. The revised Second Raising design stages (location shown on the index figure to the right) were constructed between 1984 and 1990. The goal of the revised Second Raising design was to raise Dam I by 29 m from El. 880 m msl to a crest elevation of 909 m msl. In actuality, the dam was raised only to El. 891.5 m msl in connection with this design. Various documents describe the design and construction of the revised Second Raising, including (i) the revised design, slope stability analyses, and construction technical specifications;[52] and (ii) drilling and laboratory testing reports performed during the Second Raising stages.[53]

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49 Tecnosan Second Raising Design Documents.
50 Tecnosan Second Raising Design Documents.
51 Second Raising Construction Inspection Documents.
52 Revised Design Report, Second Raising (Tecnosan 1983) (“Tecnosan Second Raising Revised Design”); Tecnosan Second Raising Construction Specs.
53 Pavisolos & Sondag Geotechnical Investigation Reports 1 and 2.

The documents reviewed do not contain a specific discussion of the reason(s) for the design change. However, the documents do indicate a few factors that could have been the basis for the revised design:

- Seepage was detected at the base of Stage 2 after the construction of Stage 3, which is reported to have been completed in 1983.[54]
- The stability of the dam apparently was considered a concern before Stage 4 was constructed, as we located a document titled "Verification of Stability."[55]
- No other contemporaneous reports mentioned seepage or seepage problems in the Starter Dam or Second Raising stages during the timeframe of 1980 to 1983. Later reports suggest "reports of seepage" in the Starter Dam base, but do not contain details as to when those reports occurred.[56]

5.4.1 Geotechnical Investigation

In 1983, a geotechnical investigation was performed which consisted of 13 percussion boreholes, numbered F-1 through F-11, including F-8a and F-9a, with standard penetration tests ("SPTs") and laboratory testing of samples collected from the boreholes.[57] The field investigation locations are presented in Figure 11. Figure 11 does not clearly indicate where the boreholes were located, and the text of the geotechnical report does not clarify this. However, the borehole logs suggest that boreholes F-1 through F-7 were performed in the tailings impoundment, and F-11 appears to have been performed on the dam.[58] The locations of the remaining boreholes are unclear.

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[54] Supplementary Technical Review – Stability Analysis Under Undrained Loading Conditions (Geoconsultoria).
[55] Stability Verification, Second Raising (Tecnosan 1983) ("Tecnosan Verification of Stability").
[56] Design Documents, Fourth Raising (Tecnosolo) ("Fourth Raising Design Report").
[57] Pavisolos & Sondag Geotechnical Investigation Reports 1 and 2.
[58] Pavisolos & Sondag Geotechnical Investigation Borehole Logs, Report 173/83 (Pavisolos & Sondag 1983).

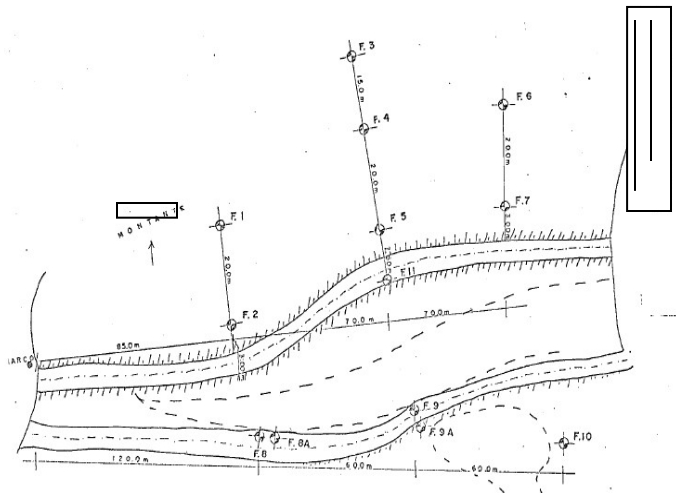

Figure 11: Second Raising Site Exploration (Revised Design)[59]

Disturbed tailings samples were tested for unit weights, moisture content, grain size distribution, and relative density in the laboratory. The results are shown in Table 4 below.

Table 4: Laboratory Test Results on Tailings Samples (Second Raising, Revised Design)[60]

Bore Hole	Dry Field Density (g/m³)	Minimum Void Ratios	Void Ratio in the Field	Natural Moisture Content (%)	Specific Gravity (g/m³)	Minimum Dry Specific Weight (g/m³)	Maximum Dry Density (g/m³)
1	2.361	0.79	1.025	16.1	4.780	2.15	2.675
2	2.329	-	1.020	14.7	4.704	-	-
3	2.481	0.77	0.844	14.4	4.574	2.15	2.579
4	2.388	-	1.096	15.5	4.006	-	-
5	2.359	-	0.901	10.9	4.484	-	-
6	2.402	0.77	0.932	16.6	4.640	2.15	2.614
7	2.380	-	1.011	7.5	4.786	-	-

5.4.2 Design Approach

The Second Raising revised design called for the construction of one centerline stage to a top elevation of 884 m msl followed by five subsequent stages, each 5 m in height, which would

59 Pavisolos & Sondag Geotechnical Investigation Report 1.
60 Modified from Tecnosan Second Raising Revised Design (translated from original Portuguese).

raise the dam crest to El. 909 m msl using the upstream method.[61] The approach is illustrated in Figure 12. The design called for a drainage blanket at the base of each of the five stages above the centerline stage (i.e., above Stage 4). Although the concept and importance of the beach was discussed in the initial design of the Second Raising,[62] it was not discussed in any of the revised Second Raising design documents.

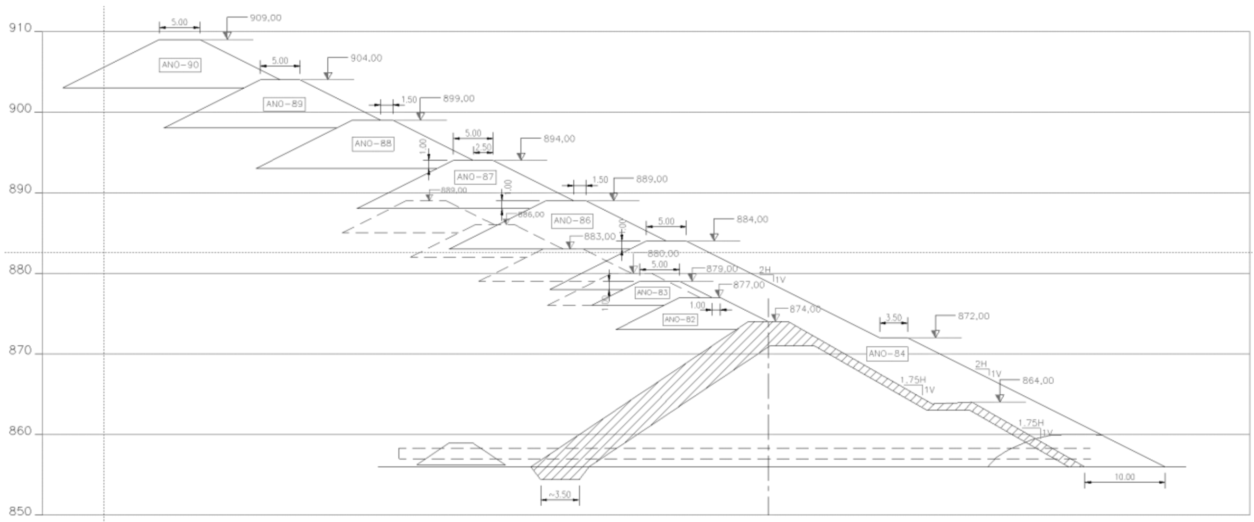

Figure 12: Revised As-Designed Second Raising Stages[63]

5.4.2.1 Geometry

The layout of the as-designed revised Second Raising is shown in Figure 12 above. The geometry of the stages called for a compacted soil slope having a grade of 2H:1V on the downstream side and, apparently, a similar slope on the upstream side. A channel was apparently intended between stage berms to manage surface water. The geometry of the centerline stage is shown as extending over the downstream side of all lower stages and having a top elevation of 884 m msl. The subsequent five stages were planned to be approximately 5 m high to raise the dam to El. 909 m msl using tailings from the beach. A 1.5-m thick compacted laterite facing was planned on the downstream slopes of the revised design Second Raising berms with slopes of 2H:1V to protect the dam from erosion.

61 Tecnosan Second Raising Revised Design.
62 Tecnosan Second Raising Design Documents.
63 Tecnosolo Second Raising Design File.

5.4.2.2 Internal Drainage

The original Second Raising design called for a drainage blanket in its second stage, but there is no evidence it was installed. The revised Second Raising design report, which was prepared after the construction of Stage 2 of the Second Raising, indicates that each remaining stage of the raising was to be constructed with a horizontal filter at the base of the stage, including a longitudinal pipe to conduct percolation to the abutments. The report notes that the purpose of the drainage system was to "…reduce the under-pressures acting on the berms of the slope, thus increasing the safety factor against its breach…."[64] There is no other discussion of internal drainage, and no illustration of the drainage filter or longitudinal pipes was found for these berms.[65]

5.4.2.3 Surface Water Management

The revised Second Raising design included a channel to drain water from the impoundment area.[66] The channel was specified to be made of concrete, which routed water to two concrete culverts that would presumably be constructed through a "cofferdam" (not the dam itself) between the impoundment and the channel leading to the original creek downstream of the toe of the dam. No plan view was found of the location of the cofferdam and decant tower.

5.4.2.4 Geotechnical Stability

Geotechnical stability for the Second Raising revised design was evaluated by Tecnosan in the "Verification of Stability" report.[67] The report provided tailings shear strength parameters of 0 kilograms per square centimeter (kg/cm^2) cohesion and a 30° angle of internal friction but provides no citation or test result for these values. Laterite and fine ore were assumed to have the same parameters as those used for the design of the starter dam and were not reevaluated. The analyses were performed by hand using both Bishop's method of slices for circular failure surfaces and a simple force balance for block surfaces, apparently assuming that no pore water pressure would be present within the dam or tailings.[68] The minimum reported FS was 1.44, which was considered adequate by the designer, given the 1.5- to 2-year anticipated construction period.

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64 Tecnosan Second Raising Revised Design (translated from original Portuguese).

65 The results of the Panel's seepage modeling strongly suggest that internal drainage was in fact installed within the Second Raising. See Appendix G.

66 Tecnosan Second Raising Revised Design; Tecnosan Second Raising Design Plan Image.

67 Tecnosan Verification of Stability.

68 Both calculations show a location for pore pressure to be considered in the calculation, and in both these locations are left blank. Tecnosan Verification of Stability.

5.4.2.5 Construction Specifications

The specifications for the construction of the Second Raising revised design stages are included in one paragraph of the design report.[69] The specification called for a minimum of 60% relative compaction for the sand and a minimum degree of compaction of 97% Proctor for the compacted laterite, with a moisture content within -2/+1% of optimum. No additional discussion or specifications were found for the sinterfeed, hematite, or itabirite materials in the revised Second Raising design documents.

5.4.3 Complications and Variances

Actual construction of the Second Raising revised design was, as was the case for the initial design, substantially different from that intended by the design. The as-constructed configuration of the revised Second Raising is illustrated in Figure 13; the centerline raising (Stage 4) was constructed in 1984, generally as planned in the design. The next stage, Stage 5, was constructed in 1986 with a planned height of 5 m but included an apparent "buttress" over the entire downstream face of the dam; this feature is not discussed in any of the available documents, and so its purpose, construction detail, and drainage considerations (if any) are not known. The third and final stage of the Second Raising revised design (Stage 6) was constructed in 1990 and was only 2.5 m high instead of 5 m high.[70]

No construction inspection reports were found that document the construction of Stage 4, and accordingly, it cannot be confirmed whether the materials used met the required specifications or whether the drainage layers were constructed as planned in the design. Several documents provide construction inspection documentation on Stages 5 and 6. The remaining three stages were not constructed as shown in the plans. Instead, those subsequent three stages were included within the design planned for the Third Raising, as discussed in Section 5.5.[71]

69 Tecnosan Second Raising Revised Design.
70 Engineering Plan, Third Raising (Chammas Engenharia 1991) ("Third Dam Raising Engineering Plan").
71 1986 Ferteco Scans (Exacta 1986) ("1986 Ferteco Scans 1-5").

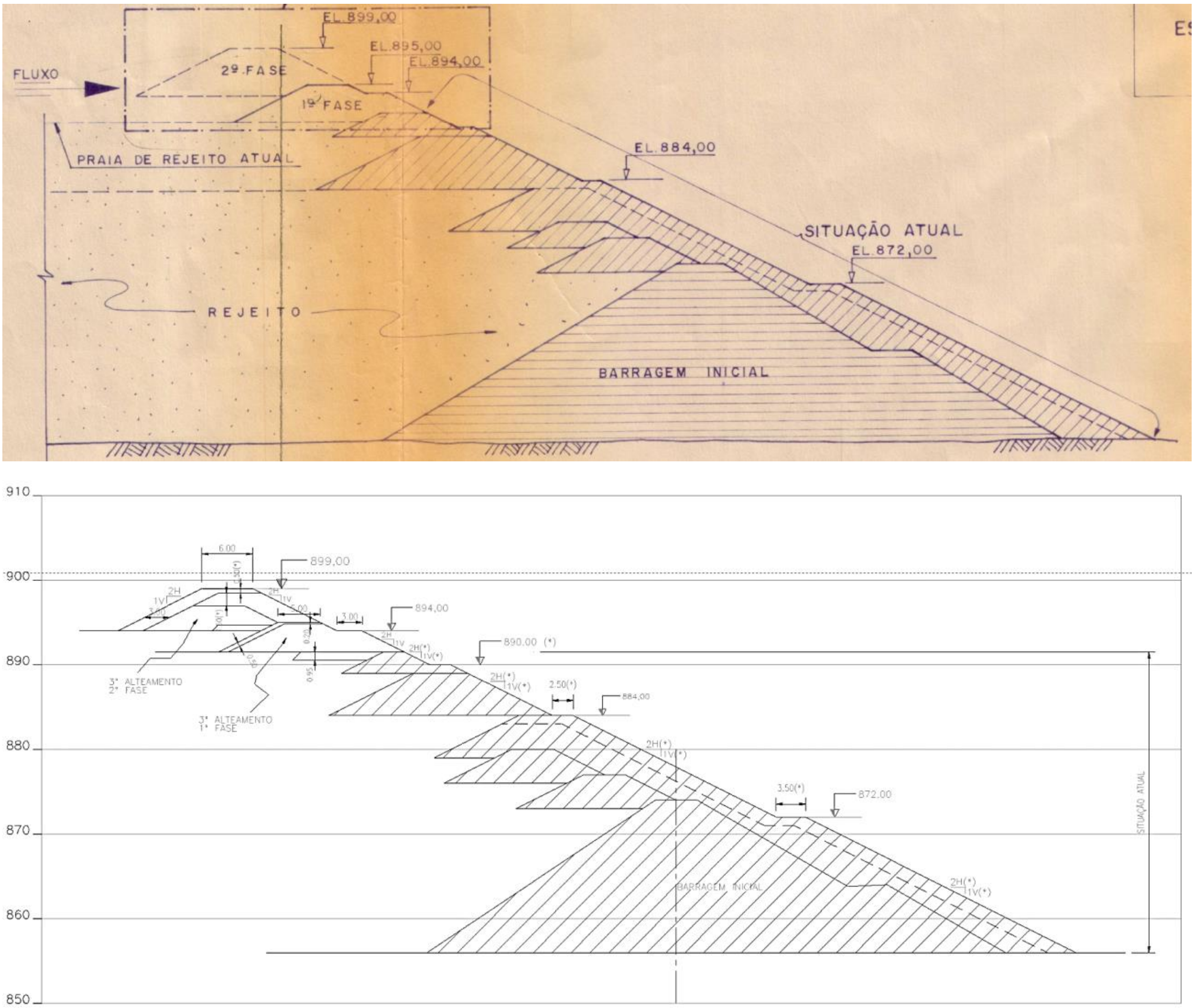
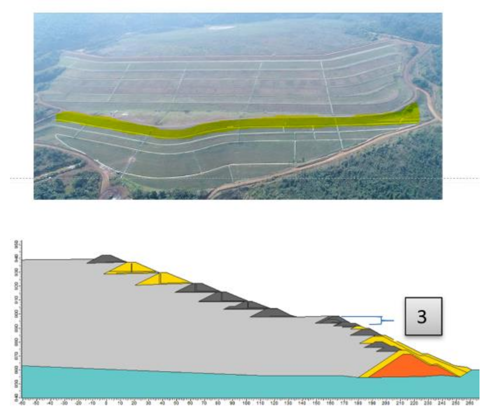


Figure 13: As-Constructed Second Raising Stages[72]

5.5 Third Raising: 1991 Through 1993



The Third Raising (location shown on the index figure to the right) was constructed between 1991 and 1993. The goal of the Third Raising design was to raise Dam I by 7.5 m from El. 891.5 m msl to a crest elevation of 899 m msl. The information available regarding the Third Raising includes a design report and design drawings by

[72] Third Dam Raising Engineering Plan; Tecnosolo Second Raising Design Files.

Chammas Engenharia for Stage 8 of the design;[73] no design documents are available for Stage 7, and no site exploration or construction-phase inspection reports are available.

5.5.1 Geotechnical Investigations

The design report includes the results of boreholes that were performed for the Third Raising. Figure 14 shows that the boreholes were performed before the first stage of the Third Raising (Stage 7). SPT blow counts indicate variable penetration resistance with depth which could signal variable material types or variable levels of compaction of the material. On the left and right abutments (i.e., boreholes SP-05 and SP-03), only tailings were encountered above the depth of exiting ground. Borehole SP-04, which was advanced through the crest of the Stage 6 berm (i.e., the fifth berm of the Second Raising) near the center of the dam, reportedly encountered layers of tailings and "fill" material used to construct the underlying berm stages. The report also indicates that the borehole encountered layers of drainage blanket material (i.e., "filtro de areia" and "filtro de minerio fina") and that water loss occurred at the depth of the drainage blanket during drilling, suggesting that the drainage blankets shown in the design drawings for the Second Raising were installed. No further information is available regarding the material encountered in the boreholes.

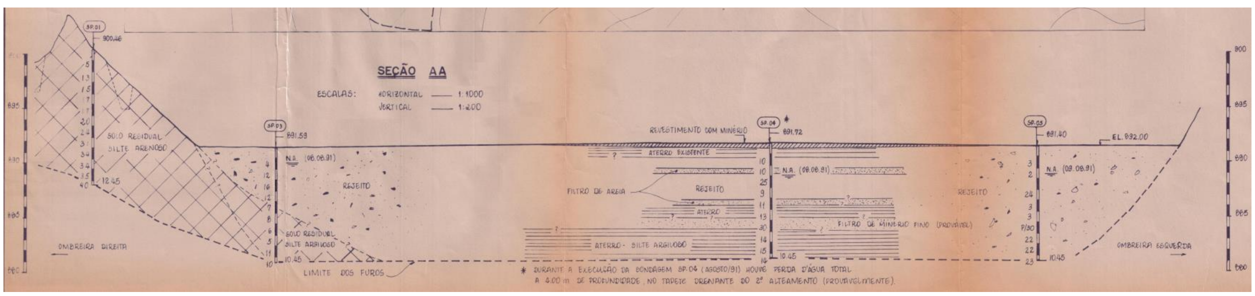


Figure 14: Third Raising Geotechnical Investigation[74]

5.5.2 Design Approach

The design concept for the Third Raising is illustrated in Figure 15. The Third Raising was developed in two phases. Phase 1 involved the construction of Stage 7, and Phase 2 involved the construction of Stage 8. For Phase 1, no design documents are available. For Phase 2, a detailed design document and construction drawings were located. The design document notes that the design of Phase 2 was based on a design for Phase 1 that had been performed in August 1991

[73] Design Documents, Third Raising Phase 2 (Chammas Engenharia 1993) ("Third Raising Phase 2 Design Report").

[74] Third Dam Raising Engineering Plan.

and construction information collected during Phase 1 construction from September to December 1991.[75] The goals of the Phase 2 design were reported to be:

- improving the existing spillway design and performance, including survey;
- improving access to the crest of the dam;
- utilizing on-site soils for borrow;
- replacing the gravel drainage layer materials with crushed sand; and
- identifying the installation of instrumentation to monitor the performance of the dam.[76]

The drawings show that Phase 1 consisted of raising the dam crest from El. 891.5 m msl to 895 m msl, and Phase 2 consisted of raising the dam crest an additional 4 m to El. 899 m msl. Both phases were designed using the upstream technique. The upstream and downstream slopes were designed having slopes of 2H:1V with all (for Phase 1) or part (for Phase 2) of the outer portion constructed using laterite clay. Clay-sized tailings sourced from the impoundment near the right abutment were specified for use where tailings were called for, and soils sourced from a borrow pit excavated as part of Phase 1 were specified to be used elsewhere. Sod was specified to protect the downstream slopes from erosion.[77]

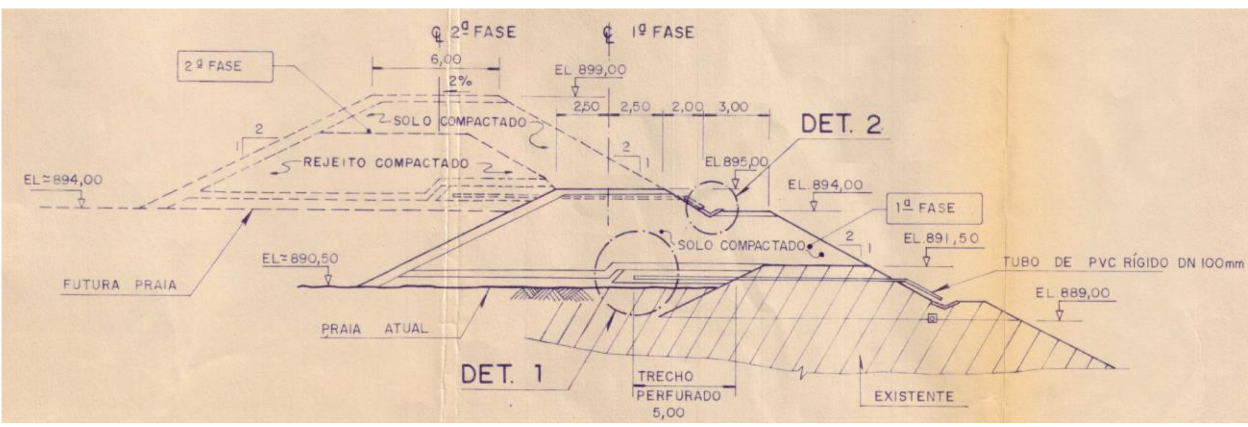


Figure 15: Third Raising Design Cross-section[78]

5.5.2.1 Geometry

In Figure 15, both Phases 1 and 2 were designed having upstream and downstream slopes with grades of 2H:1V, with all (Phase 1) or part (Phase 2) of the upstream portion constructed using clay. A 3-m berm was planned at the bottom of the downstream slope where a drainage channel

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[75] Third Raising Phase 2 Design Report.
[76] Third Raising Phase 2 Design Report.
[77] Third Raising Phase 2 Design Report.
[78] Third Dam Raising Engineering Plan.

was constructed during Phase 1. The crest of the Phase 2 construction was designed with a 2% upstream slope to direct precipitation into the decant pond.[79]

5.5.2.2 *Internal Drainage*

The Third Raising stages were designed with internal drainage features to control seepage. The design detail is shown in Figure 16 and included a 1.5-m thick drainage layer at the base of the stages constructed using a pellet feed transition layer underlain by a continuous gravel toe drain of sinterfeed wrapped in a geotextile ("Bidim OP-20") that discharged water directly to the drainage channel on the downstream slope of the dam. Drainage from the drain layer was to be routed from the toe drainage to the discharge channel using rigid 100-mm diameter polyvinyl chloride ("PVC") pipes installed every 50 m along the length of the beach. These PVC pipes were planned to be perforated along the section embedded in the drainage layer.[80]

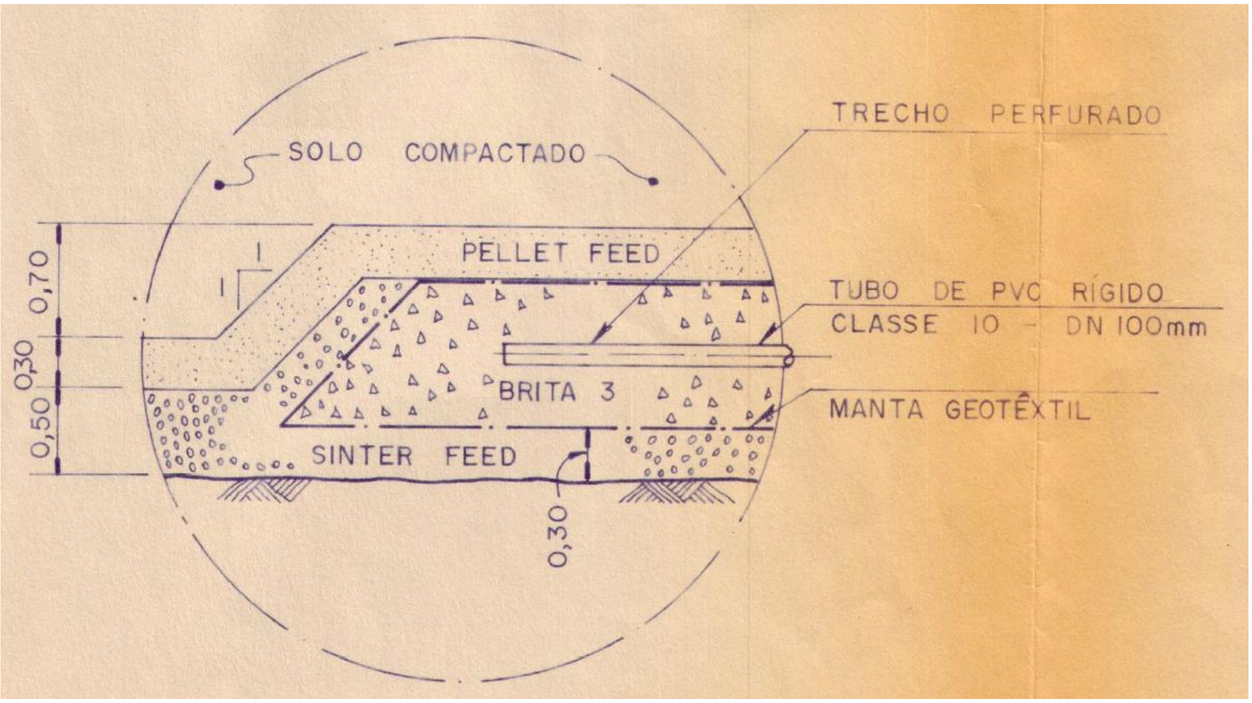


Figure 16: Third Raising Design Drainage Layer[81]

79 Third Raising Phase 2 Design Report.
80 Third Raising Phase 2 Design Report.
81 Third Dam Raising Engineering Plan.

5.5.2.3 Surface Water Management

The Third Raising was designed to manage surface water by routing ponded water above the tailings to the Feijão creek around the dam's right abutment. No details are available for the Phase 1 surface water design, but the Phase 2 design included significant detail on the drainage control structure within the tailings and the discharge channel around the right abutment. The design called for a 35-m long reinforced concrete rectangular spillway at El. 897.5 m msl to route flow into the previously constructed Phase 1 spillway. A channel was designed along the crest to protect the Phase 2 spillway excavation slopes and to prevent the erosion of the right abutment.[82]

5.5.2.4 Geotechnical Stability

No geotechnical stability analyses for the Third Raising berms are available. The stability of the Third Raising stages is mentioned, however, in the design of the Fourth Raising.[83] The stability analyses provided in the summary tables of that report show the results of analyses performed for both Phase 1 and 2 of the Third Raising. The analyses were performed and resulted in a minimum FS of 1.33 for Phase 1 and a minimum FS of 1.23 for Phase 2 (Table 5).

82 Third Raising Phase 2 Design Report.
83 Fourth Raising Design Report.

Table 5: Third Raising Design Stability Analyses Results[84]

Elevation of Crest of the Dam (m)	**Elevation of Tailings (m) Nr**	**Elevation of the Water Level in the Reservoir (m) Na**	**Factor of Safety – Modified Bishop**	**Factor of Safety- Fellenius**	**Condition**	**Notes**	**Date of the Studies**
909	ND	ND	1.44	ND	1	2	August / 1983
891.5	891	891	1.521	1.353	Prior to the Third Raising		
895	891	894	1.509	1.344	After Construction of First Phase of Third Raising, Before Tailings Placement	2	ND
895	894	894	1.487	1.330	After Construction of First Phase of Third Raising and Tailings Placement		
899	898	894	1.429	1.235	After Construction of Second Phase of Third Raising, Before Tailings Placement		
899	898	898	1.408	1.225	After Construction of Second Phase of Third Raising and Tailings Placement		

ND: Not Available.
1: Maximum elevation of the dam considered in 1983 (elevation 909 m).
2: Research done with rupture surfaces tangent to the foundation line.

5.5.2.5 Construction Specifications

The Third Raising Phase 2 Design Report provides detailed construction specifications for Phase 2. Phase 2 was specified to be constructed of compacted silt-clay soils from the nearby borrow pit area as well as tailings located on the right abutment area of the impoundment (i.e., fine tailings). The construction specification indicated the need for a field laboratory which would oversee compaction as well as survey and grade control and construction quality assurance oversight. The specification also indicated the need to verify that the drainage materials were constructed having specified characteristics.[85]

84 Modified from Fourth Raising Design Report (translated from original Portuguese).
85 Third Raising Phase 2 Design Report.

Compacted lifts were specified to have a maximum thickness of 25 cm for areas to be compacted using heavy equipment and 15 cm for areas where manual compaction was required. Compaction requirements specified optimum moisture content within 2% of the optimum moisture content (i.e., +/-2%) and a minimum of 98% of the standard Proctor density. A pellet feed transition layer material was specified to be compacted to a minimum of 65% of relative compaction density and was required to meet the following filter gradation requirements:

- $D_{15,\text{transition}} > 5D_{15,\text{base material}}$
- $D_{15,\text{transition}} < 5D_{85,\text{base material}}$
- $D_{5,\text{filter}} > 0.74\text{mm}$ (i.e., 200 sieve)[86]

Clean rock free from contamination available from a regional supplier was specified for the drainage gravel. Alternatively, onsite crushed ore could be used, with approval from the engineer.

5.5.3 Instrumentation

As part of Phase 2, instrumentation was installed to monitor Dam I. The Third Raising Phase 2 Design Report identifies[87] the following instruments to be installed during development of the Third Raising, Phase 2, which are illustrated in Figure 17:

- piezometers[88] in the foundation soils and in the dam;
- sub-horizontal drains in the Phase 2 drainage layer; and
- concrete surface monitoring points (MS-1 through MS-22)[89] on the berms located at El. 872 m msl, 884 m msl, 890 m msl, 894 m msl, and 899 m msl.

86 Third Raising Phase 2 Design Report.

87 Third Raising Phase 2 Design Report.

88 Instrumentation Plan, Third Raising (Chammas Enhenharia 1993) ("Third Raising Instrumentation Plan"). The Third Raising Instrumentation Plan indicates 18 piezometers would be installed, not 21 as described in the text of the design report or 22 as implied by the numbering of the piezometers.

89 There is a discrepancy between the reference to 18 surface monitoring points in the text of the report and the 22 shown on the Third Raising Instrumentation Plan. See Third Raising Instrumentation Plan.

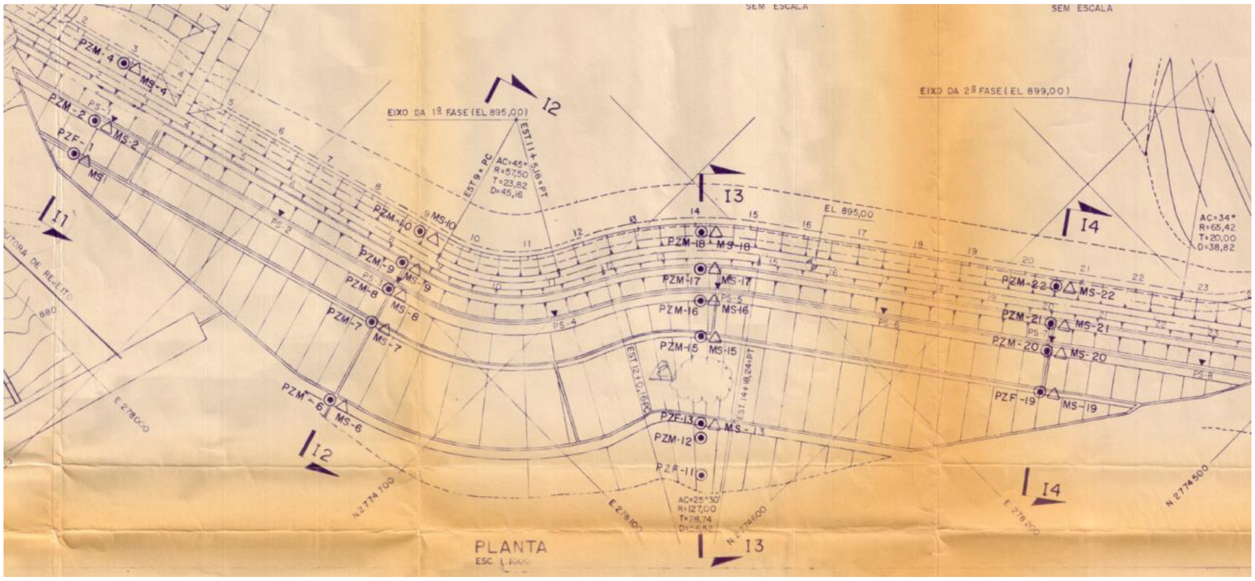


Figure 17: Third Raising Instrumentation Plan[90]

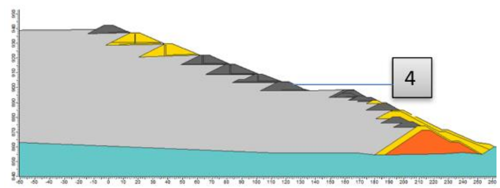
The instrumentation system was designed to monitor excess pore water pressures, dam movement, and internal drainage efficiency. The design document mentions, in particular, that the purpose of the piezometers is "…to monitor the development of interstitial pressures and to establish the current phreatic surface and its evolution as a function of future raisings…" and that the sub-horizontal drains "…together with data from the adjacent piezometers will allow an assessment to be made of the behavior and performance of the internal drainage system of the dam."[91] A stability and hydrology report for the Fifth Raising references their installation.[92]

90 Third Raising Instrumentation Plan.

91 Third Raising Phase 2 Design Report (translated from original Portuguese).

92 Design Report Volume 2, Fifth Raising (Tecnosolo 1998) ("Tecnosolo Fifth Raising Design Report Vol. 2").

5.6 Fourth Raising: 1995



The Fourth Raising (location shown on the index figure to the right) was constructed as one stage (as was the case for the remaining six raisings). It was developed after the Third Raising had been constructed according to its original plan. However, the Fourth Raising represented a departure from the original earlier design[93] by incorporating a setback in the face of the dam, as discussed in Section 5.6.2.1. A significant amount of information is available for the Fourth Raising, including survey information, design reports, construction drawings, and technical specifications.[94] The attachments to those reports, including detailed calculations, are not available. The results of the design analyses, however, are available and summarized below. No construction phase documents are available that describe how the Fourth Raising actually was constructed.

5.6.1 Geotechnical Investigations

The design report[95] mentions that a geotechnical investigation was performed for the Fourth Raising but the appendix containing the geotechnical report is not available. The design report also indicates that piezometric water level readings were collected from piezometers between the dates June 20, 1994 and June 6, 1995,[96] but no data are available regarding the installation or measurement of these piezometers from this period.[97]

5.6.2 Design Approach

5.6.2.1 Geometry

The geometry of the dam was significantly changed during the Fourth Raising. As discussed in Section 5.2.2.5, the alignment of the dam was offset approximately 4 m upstream of the existing Third Raising slope due to stability concerns. A typical cross-section is shown in Figure 18, which shows that the slopes were to be graded at 2H:1V both upstream and downstream[98] and a drainage blanket (referred to as "tapete filtrante de sinterfeed" on the figure) was to be constructed under the downstream side of the centerline of the dam.

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93 Tecnosan Second Raising Revised Design.
94 Tecnosolo Fourth Raising Design Files.
95 Fourth Raising Design Report.
96 Fourth Raising Design Report.
97 As detailed in Appendix C, dates of available piezometer data begin in April 1996.
98 Tecnosolo Fourth Raising Design Files.

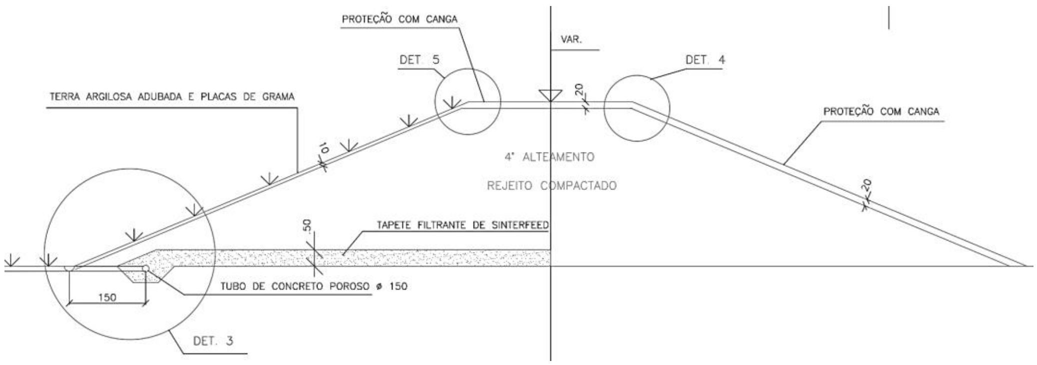

Figure 18: Typical Cross-section of the Fourth Raising Berm[99]

The technical specifications mention that this filter layer should be constructed using the tailings from the beach formed by the Third Raising.[100]

5.6.2.2 Selected Material Parameters

Materials for construction are discussed in the engineering report and the technical specifications,[101] and also on the drawings (as illustrated above in Figure 18).

5.6.2.3 Internal Drainage

The internal drainage system of the Fourth Raising consisted of a horizontal sinterfeed filter layer at the downstream half of the berm's base. The filter layer fed into a longitudinal 150-mm PVC pipe wrapped in hematite at the downstream edge of the raising. The longitudinal pipe diverted water to a perimeter canal, as shown in Figure 19.[102] No filter analysis calculation was provided in the reports available for review of these materials, and the locations of the outlet pipes are not specified.

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99 Tecnosolo Fourth Raising Design Files.
100 Technical Specifications, Fourth Raising ("Fourth Raising Technical Specs").
101 Fourth Raising Design Report; Fourth Raising Technical Specs.
102 Tecnosolo Fourth Raising Design Files.

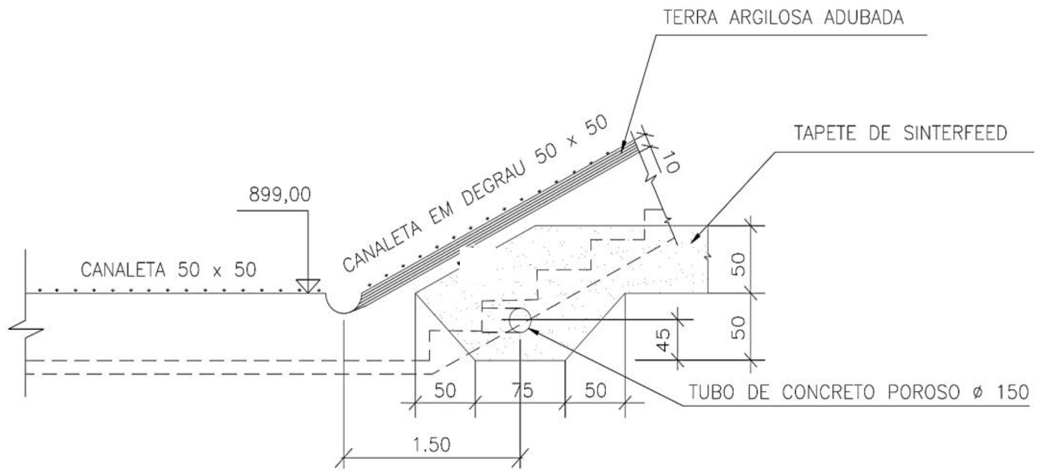

Figure 19: Fourth Raising Carpet Drain Edge Detail[103]

5.6.2.4 Surface Water Management

The Fourth Raising design included an analysis of the water flows that would need to be conveyed by the channel using the U.S. Soil Conservation Service hydrograph method and an assumed precipitation event. This was used as input to the hydraulic design of the perimeter concrete and outlet channels that conveyed water to the creek downstream of the dam. A concrete perimeter canal was constructed as part of the Fourth Raising. Water collected in the perimeter canal was routed along the right abutment to Dam VI.[104] Like the Third Raising, the design specified that upstream slope of the Fourth Raising was to be protected by a layer of compacted laterite and that all constructed slopes above the water level were to be protected by grass.[105]

5.6.2.5 Geotechnical Stability

As discussed in Section 5.5.2.1, the design report included a discussion on geotechnical stability in the body of the report, but the detailed stability analysis report that was included as Annex 6.2 is not available for review. The design report notes that the results of the piezometer readings

103 Tecnosolo Fourth Raising Design Files.
104 Tecnosolo Fourth Raising Design Files.
105 Tecnosolo Fourth Raising Design Files.

taken before the report was prepared[106] indicated that seeps were visible on the face of the dam:[107]

> "The readings of water level in the piezometers, in very high quotas, show very unfavorable conditions, which we consider as not recommended for the safety of the dam.
>
> "During the site visit, it was possible to observe the occurrence of percolation (seepage) of water in some points of the dam massif, at levels below the quota 875 m msl (region corresponding to the initial dam)."[108]

The report suggested that the design be revised to address these issues and that piezometric monitoring be continued to assess water conditions in the dam:

> "The indicated factors [of safety] and the determined reduced factors [of safety], recommend that urgent measures be taken with the goal of identifying the causes and better defining adequate solutions to solve the problems.
>
> "Regardless of the measures above, it is recommended to continue the monitoring of the preferential path (percolation) of the water through the dam massif, with the support of the existing piezometers and those planned to be installed (see diagram in the Annex 6.6)".[109]

Both the Fellenius and modified Bishop methods were used to assess the stability of the Fourth Raising. The designer calculated FS of approximately 1.1 for conditions at the time of the Third Raising. The Fourth Raising design report states that the "analysis of the obtained results leads us to conclude that the safety factors are below the values that would be considered ideal"[110] (Table 6).

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106 The drawings and specifications for the Fourth Raising are not dated, so it is not clear when they were prepared. The stability analyses studies are noted as being performed in June 1995, so it is assumed that the reports, drawings, and specifications were issued in that timeframe.
107 Fourth Raising Design Report.
108 Fourth Raising Design Report (translated from original Portuguese).
109 Fourth Raising Design Report (translated from original Portuguese).
110 Fourth Raising Design Report (translated from original Portuguese).

Table 6: Stability Analysis Summary for Fourth Raising Design[111]

Elevation of the Crest of the Dam (m)	Elevation of Tailings (m) Nr	Elevation of the Water Level in the Reservoir (m) Na	Factor of Safety – Modified Bishop	Factor of Safety-Fellenius	Condition	Notes	Date of the Studies
899	898	898	1.13	1.07	After Construction of Second Phase – Third Raising and Tailings Placement	1	June 1995
905 average quota (axis displaced)	903	903	1.13	1.10	After Construction of Fourth Raising and Tailings Placement		
905 average quota (axis displaced)	903	903	1.33	1.23	After Construction of Fourth Raising and Tailings Placement	1 2	
945 average quota (axis displaced)	943	943	1.33	1.22	Dam Completed		
945 average quota (axis displaced)	943	943	1.93	1.87	Dam Completed	3	

1: Research carried out considering water level readings in the piezometers as provided by Ferteco and also considering rupture through the foundation.
2: Assuming that the instability problems observed in the current massif were solved.
3: Surface of the rupture passing through the baseline created by the displacement of the axis of the Fourth Raising (898 m quota).

Future raisings to El. 945 m msl also were analyzed. As shown in Table 6, these analyses estimated higher FS of between 1.22 and 1.93.[112] No information is available on the assumed geometry, material strengths, or piezometric conditions for the cases analyzed.

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[111] Modified from Fourth Raising Design Report.

5.6.2.6 Construction Specifications

Construction specifications are available for the Fourth Raising.[113] The specifications are descriptive in nature and quantitative only in a few specific instances. Construction materials were to be obtained from the tailings of Dam I and Dam VI.[114] Compaction lifts were specified with a maximum thickness of 20 cm and a minimum density of 100% of the standard Proctor within 2% of optimum moisture content,[115] except in lifts directly above the planned filter which specified the following minimum densities (suggesting an understanding and attention to the importance of this material to the performance of the dam):

- first lift above the drainage layer: 90% of standard Proctor;
- second lift above the drainage layer: 93% of standard Proctor; and
- third lift above the drainage layer: 96% of standard Proctor.[116]

Sinterfeed material required for the internal drainage system was specified[117] to have a similar gradation to the grain size distribution curve provided in Annex 6.1 (which is not available for review). The report does not specifically require quality control testing for grain size distribution or quantify what constitutes "similar" gradation.

5.6.3 Instrumentation

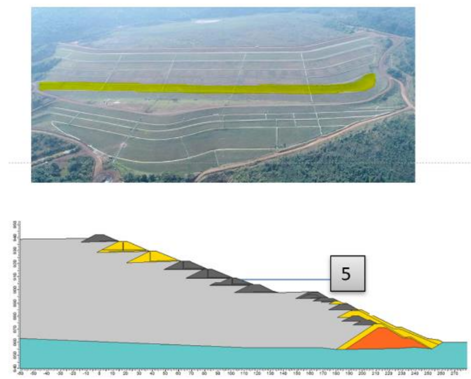
The design document indicates that, as part of the Fourth Raising, instrumentation would be installed to monitor Dam I.[118] However, the instruments are not identified in the engineering report, on the drawings, or in the technical specifications.

5.6.4 Complications and Variances

The available reports do not provide any construction-phase information for the Fourth Raising berm. However, the design documents for subsequent raisings do not describe any variances to the design or specifications during the implementation of the Fifth Raising. As-built information was not available.

112 Fourth Raising Design Report.
113 Fourth Raising Technical Specs.
114 Fourth Raising Technical Specs.
115 Fourth Raising Technical Specs.
116 Fourth Raising Technical Specs.
117 Fourth Raising Design Report.
118 Fourth Raising Design Report.

5.7 <u>Fifth Raising: 1998</u>



The Fifth Raising (location shown on the index figure to the right) was designed by Tecnosolo using the upstream method to raise the crest of Dam I by 5 m to El. 910 m msl in a single stage. A significant amount of information is available for the Fifth Raising, including a design report, construction drawings, slope stability and hydraulic analyses results, technical specifications, and a piezometer installation plan. The design documents are dated February and March 1998,[119] and the raising was constructed in 1998. No attachments are available for any of these reports, and so no detailed calculations are available for review, but the results of the design analyses are summarized below. No construction phase documents are available that describe how the Fifth Raising actually was constructed.

5.7.1 Geotechnical Investigations

The stability and hydrology analysis report indicates that the design was developed after a review of geotechnical investigations performed in 1995 and 1997 by Tecnosolo,[120] but these reports are not available for review. The report also discusses the groundwater conditions considered for the design.[121] The design report describes the foundation conditions as varying in the area where the Fifth Raising berm was to be constructed, with soft foundation conditions near the right abutment and more firm foundation conditions near the left abutment.

> "On the section between the right shoulder and approximately stake 10 + 10m, the seat will be placed on top of the soft, spongy waste, saturated or with a high degree of saturation. On this section, the plan is to slowly and carefully build the initial platform that will have 2.0 m of height, with the utilization of light transportation and earthmoving equipment, without excessive vibration."[122]

The report also indicates that in the remainder of the Fifth Raising foundation (i.e., stake 10 + 10 m up to stake 29 + 10 m) "…the existing tailings beach shows more favorable conditions, and the compacted landfill can be placed there beginning at the levelling course,"[123] suggesting that the tailings surface near the right abutment did not have a properly formed beach.

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[119] Design Report Volume 1, Fifth Raising (Tecnosolo 1998) ("Tecnosolo Fifth Raising Design Report Vol. 1"); Design Report Volume 2, Fifth Raising (Tecnosolo 1998) ("Tecnosolo Fifth Raising Design Report Vol. 2").

[120] Tecnosolo Fifth Raising Design Report Vol. 2.

[121] Tecnosolo Fifth Raising Design Report Vol. 2.

[122] Tecnosolo Fifth Raising Design Report Vol. 1 (translated from original Portuguese).

[123] Tecnosolo Fifth Raising Design Report Vol. 1 (translated from original Portuguese).

5.7.2 Design Approach

5.7.2.1 Geometry

The Fifth Raising design consisted of one stage that was to be constructed directly adjacent to the Fourth Raising and parallel to its alignment. An initial 4-m wide by 2-m high berm (Figure 20) was designed on a 210 m section starting at the right abutment of the Fourth Raising to mitigate the soft saturated conditions described in Section 5.6.1. This berm raised the right section of the Fifth Raising 2 m higher than the left.[124]

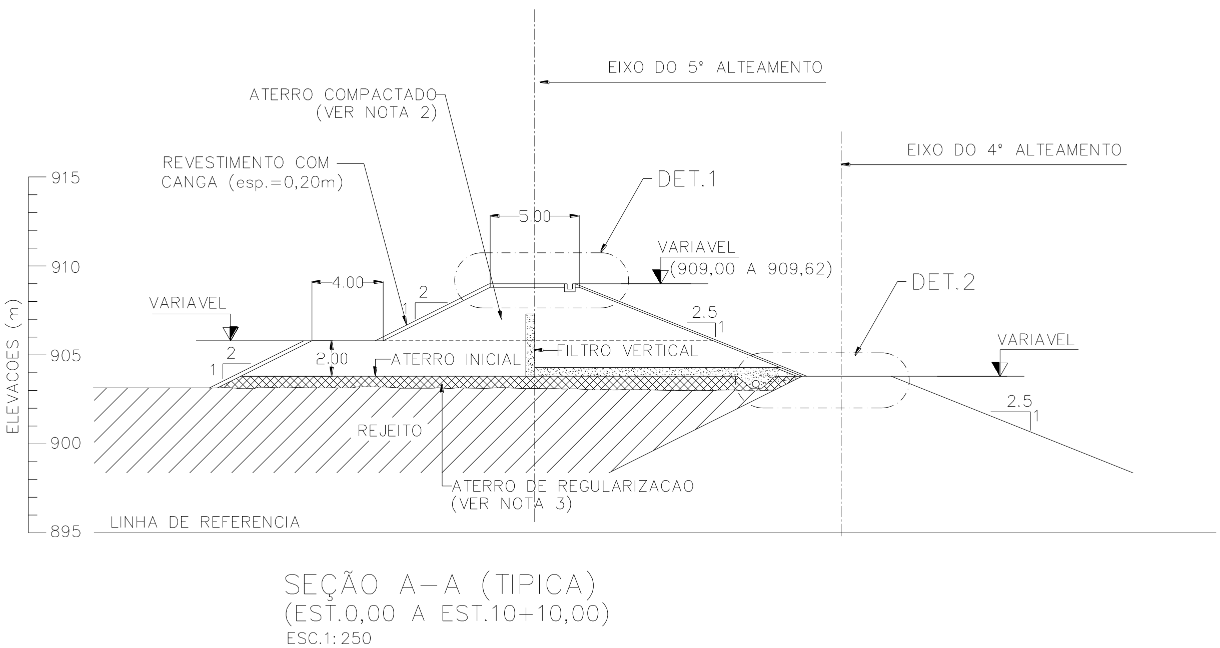

Figure 20: Typical Cross-section from Station 0,00 – 10+10,00[125]

The Fifth Raising was planned to be 5-m high with a 5-m wide crest. A crest elevation of 911 m msl near the left abutment and 909 m msl near the right abutment was specified. The crest was designed with a 1% slope to divert runoff into the decant pond. Upstream and downstream slopes were to be constructed at 2H:1V and 2.5H:1V, respectively.[126]

124 Tecnosolo Fifth Raising Design Report Vol. 1.
125 Tecnosolo Fifth Raising Design Files.
126 Tecnosolo Fifth Raising Design Report Vol. 1.

5.7.2.2 Selected Material Parameters

Selected geotechnical parameters for the Fifth Raising were provided in the design report and are presented in Table 7.

Table 7: Fifth Raising Material Parameters[127]

Material	Total Unit Weight (kN/m^3)	Friction Angle (ϕ)	Cohesion (kPa)	Undrained shear strength (kPa)
Sluiced/loose tailings	25	33º	0	20 to 40
Compacted tailings	27.5	40º	0	-
Clayey soils	19	30º	8.5	-
Foundation – residual soils	19	30º	8.5	-
Foundation – colluvial soils	17	25º	15	-

The undrained shear strength range for saturated tailings was determined by the designers through back-calculations, using the assumption that the Fourth Raising was stable at an assumed FS. Using this approach, the undrained shear strength was calculated as 20 kilopascals (kPa) for an assumed FS of 1.10 and 25 kPa for an assumed FS of 1.30. An undrained strength ratio (s_u/σ'_v) of 0.22 was then selected.[128] No basis is provided for the selection of undrained strength ratio; however, a shear strength of 40 kPa was used for long-term conditions in analyses.[129]

5.7.2.3 Internal Drainage

Design drawings indicate an internal drainage system consisting of a vertical filter and horizontal filter constructed in an L shape and PVC pipes. The vertical filter (0.8-m thick) was planned within the core, starting 1 m below the Fifth Raising crest to the tailings where it would tie into a 0.5-m-thick drainage blanket (Figure 21). The vertical filter and drainage blanket were designed to be constructed using sinterfeed material. Unlike earlier design documents, no discussion of gradation or filtering of the different drainage layer materials is provided in the report.

[127] Modified from Tecnosolo Fifth Raising Design Report Vol. 2 (translated from original Portuguese).
[128] Tecnosolo Fifth Raising Design Report Vol. 2.
[129] Tecnosolo Fifth Raising Design Report Vol. 2.

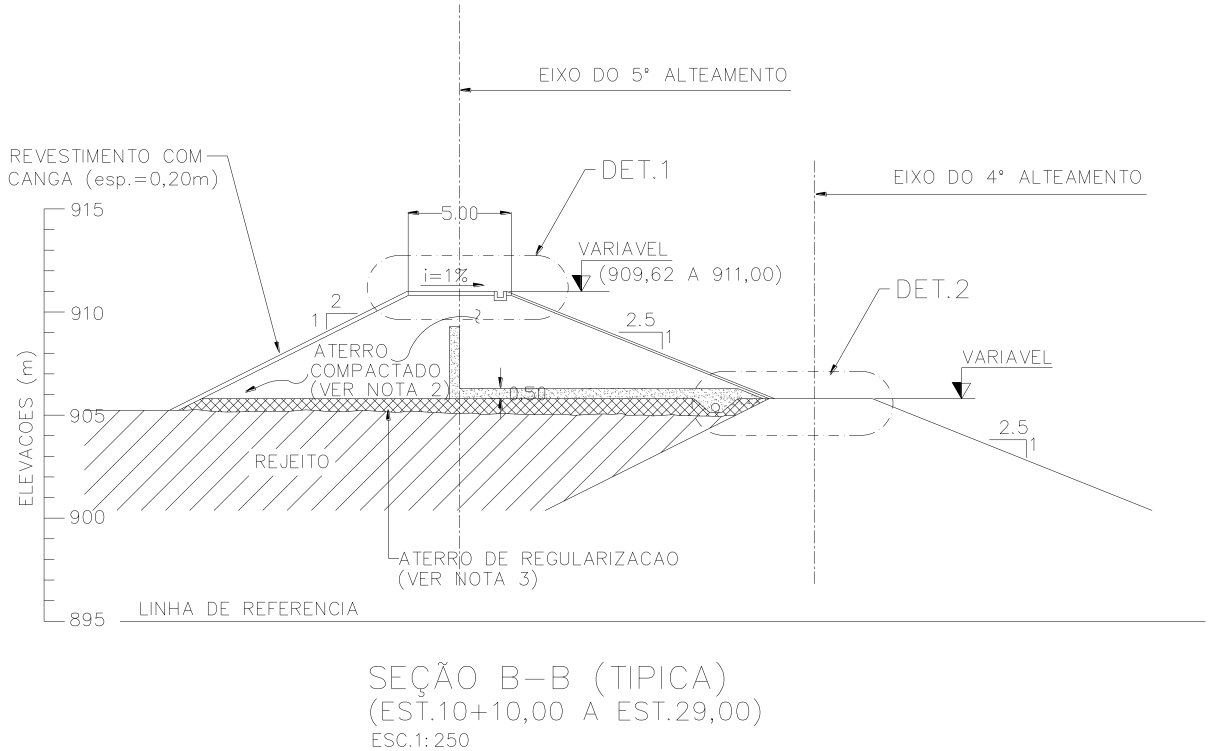

Figure 21: Typical Cross-section from Station 10+10,00 – 29,00[130]

Horizontal perforated PVC pipes 150 mm in diameter were planned every 50 m along the Fifth Raising alignment, placed perpendicular to the alignment. The drainage system was designed to outflow into a reinforced concrete channel at the toe of the raising (Figure 22).

[130] Tecnosolo Fifth Raising Design Files.

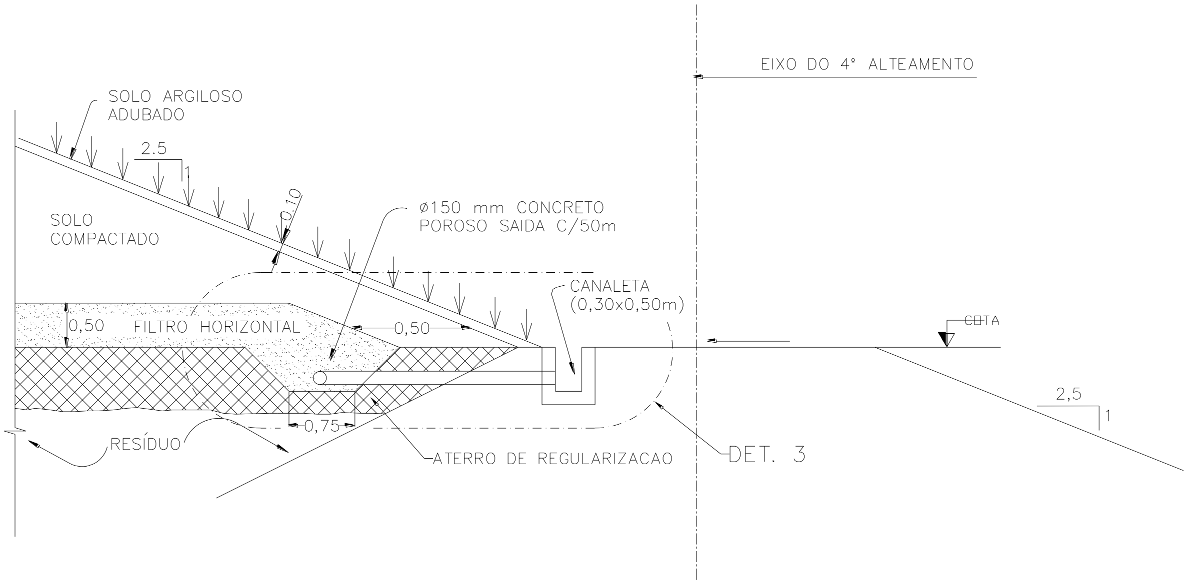

Figure 22: Typical Detail of Drainage Outfall[131]

5.7.2.4 Surface Water Management

The surface drainage system was planned to consist of open concrete channels at the toe of the Fifth Raising on the downstream side. The channels were designed to feed into the existing Fourth Raising drainage infrastructure through a transverse gutter system. As described previously for the Fourth Raising, the drainage system was routed to an overflow that discharged into Dam VI.[132]

A 0.2-m-thick layer of compacted material (e.g., laterite or hematite) was specified for use in the construction of the upstream slope and crest, to act as a protective layer.[133] Grass was specified for use on the downstream slope, similar to previous raisings, to prevent erosion. A spillway system was planned on the right shoulder starting at El. 909 m msl and consisting of the following:

- a trapezoidal channel with its bottom at El. 907.75 m msl, a width of 1.5 m, and a slope of 1H:1V including a concrete weir structure approximately 200 m from the Fifth Raising alignment;
- a rectangular reinforced concrete spillway that passes over the crest of the dam; and

[131] Tecnosolo Fifth Raising Design Files.
[132] Tecnosolo Fifth Raising Design Report Vol. 1.
[133] Tecnosolo Fifth Raising Design Report Vol. 1.

- the existing Fourth Raising channel.[134]

5.7.2.5 Geotechnical Stability

Geotechnical stability was evaluated using piezometric data to estimate water levels in the dam and elevated piezometric conditions, assuming the water surface reached the internal drainage layers of each raising.[135] The report describes an "unfavorable piezometric line" used in the long-term analysis as that resulting from a full upstream reservoir flowing through the vertical filters and horizontal drainage blankets;[136] however, the appendices which apparently provide the elevations and data for this condition are not available. Both total and effective stress Bishop's method analyses were implemented using the computer software UTEXAS-3 to estimate FS. The report indicates that there was a possibility of liquefaction that could lead to an undrained shear strength condition,[137] and therefore, undrained and drained conditions were considered in the analyses. A minimum FS of 1.32 was calculated for the pre-construction conditions (i.e., Fourth Raising). The minimum FS of the Fifth Raising calculated under the "most unfavorable" piezometric conditions was 1.22, considered valid for the most saturated region near the right abutment area.[138] Finally, the stability of the fully constructed dam to El. 945 m msl under normal operation was evaluated assuming "very unfavorable" piezometric conditions and had a minimum FS of 1.40.[139]

5.7.2.6 Site Preparation

Fifth Raising design documents indicate the following items were required to be inspected during site preparation:

- vegetation and organic material removed;
- shoulder areas of the dam with slopes greater than 1H:1V be regraded to a maximum slope of 1H:1V;
- the final surface of the tailings deposit be level with the crest of the Fourth Raising; and
- soft material near the right abutment must be removed.[140]

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[134] Tecnosolo Fifth Raising Design Report Vol. 1.
[135] Tecnosolo Fifth Raising Design Report Vol. 2.
[136] Tecnosolo Fifth Raising Design Report Vol. 2.
[137] Tecnosolo Fifth Raising Design Report Vol. 2.
[138] Tecnosolo Fifth Raising Design Report Vol. 2.
[139] Tecnosolo Fifth Raising Design Report Vol. 2.
[140] Tecnosolo Fifth Raising Design Report Vol. 1.

5.7.2.7 Construction Specifications

Construction specifications for the Fifth Raising call for the raising to be constructed of silty-clayish soils free of organics and debris obtained from excavations created during drainage channel and access road construction.[141] The excavated material was planned to be supplemented by tailings from the left abutment beach if needed. Design documents call for silty-clayey soils to be used for the initial layers along the full length of the dam with tailings for layers placed thereafter.[142]

Compaction lifts were specified with a maximum thickness of 20 cm, a minimum density of 95% of the standard Proctor, and average density of 98% of the standard Proctor within +/- 2% of the optimum moisture content.[143]

In the soft saturated section between the right abutment and Station 10+10,00, the initial soil platform design called for horizontal layers no more than 50-cm thick to be compacted via spreading equipment until a height of 2 m is achieved, noting: "On this section, the construction of the initial platform should be done slowly and carefully, with the use of light transportation and earthmoving equipment, without excessive vibration."[144]

5.7.3 Instrumentation

The design called for survey benchmarks and piezometers to be installed[145] to monitor the foundation and raisings of Dam I. The design drawings show that 10 piezometers were specified having casing lengths between 8 m and 20 m, five located at the top of the dam and five located at the downstream toe of the Fifth Raising.[146] Also, a piezometer monitoring plan was issued with the Fifth Raising design that identified certain piezometric levels that corresponded to a calculated FS against slope failure of 1.30 (which were referred to as "attention" levels) and also higher piezometric levels that corresponded to a calculated FS against slope failure of 1.15 (which were referred to as "alert" levels). The plan also mentions the new piezometers to be added and describes the importance of maintaining and monitoring the piezometers for the stability of the dam and understanding water movement through the dam.[147]

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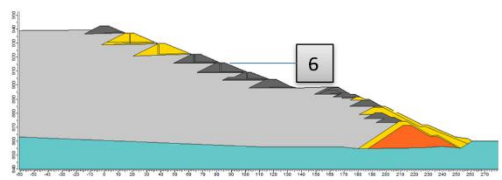
141 Tecnosolo Fifth Raising Design Report Vol. 1.
142 Tecnosolo Fifth Raising Design Report Vol. 1.
143 Tecnosolo Fifth Raising Design Report Vol. 1.
144 Tecnosolo Fifth Raising Design Report Vol. 1 (translated from original Portuguese).
145 Tecnosolo Fifth Raising Design Report Vol. 1.
146 Tecnosolo Fifth Raising Design Files.
147 Piezometric Levels Monitoring Plan, Fifth Raising (Tecnosolo 1998) ("Tecnosolo Fifth Raising Piezometer Monitoring Plan").

5.7.4 Complications and Variances

As described earlier, the area approximately 210 m from the right abutment was built more slowly due to the presence of soft and saturated tailings. This required a 2-m-thick lift to bridge the soft soils as well as a wider crest by approximately 4 m, which led to the variable elevation of the Fifth Raising crest. As-built information for the Fifth Raising is not available. Therefore, there is no information available about whether the subgrade was prepared or fill placement monitored with any particular emphasis on the potential for subgrade instability. Also, no records are available that document the materials actually used to construct the drainage blanket layer and outlet.

5.8 <u>Sixth Raising: 2000</u>



The Sixth Raising (location shown on the index figure to the right) was designed by Tecnosolo and consisted of one stage to raise the crest of Dam I by 6.5 m to El. 916.5 m msl along the alignment of the Fifth Raising. The available design documents indicate an approach that closely resembles the Fifth Raising. Available information includes design drawings, stability and hydrology analysis reports, stability analysis figures, and a laboratory report of beach tailings test results. The stability calculation report text[148] and figures showing the results of the stability analyses were provided and are summarized below.

5.8.1 Geotechnical Investigations

Four boreholes (SP-01, SP-02, SP-03, and SP-04) were completed and samples collected for laboratory testing.[149] Samples were characterized, and the following tests were performed: compaction, direct shear, permeability, unconsolidated undrained (UU) triaxial compression. Water level measurements from the Fifth Raising and from SP-01, SP-02, SP-03, and SP-04 were used to estimate the water levels applied during the slope stability assessments.[150]

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[148] Design Report, Sixth Raising (Tecnosolo 2000) ("Tecnosolo Sixth Raising Design Report").
[149] Table of Test Results, Sixth Raising.
[150] Tecnosolo Sixth Raising Design Report.

5.8.2 Design Approach

5.8.2.1 Geometry

The Sixth Raising was planned to be 6.5 m high with a 5-m-wide crest. A crest elevation of 916.5 m msl was specified. Upstream and downstream slopes were constructed at 2H:1V and 2.5H:1V, respectively, as shown in Figure 23.

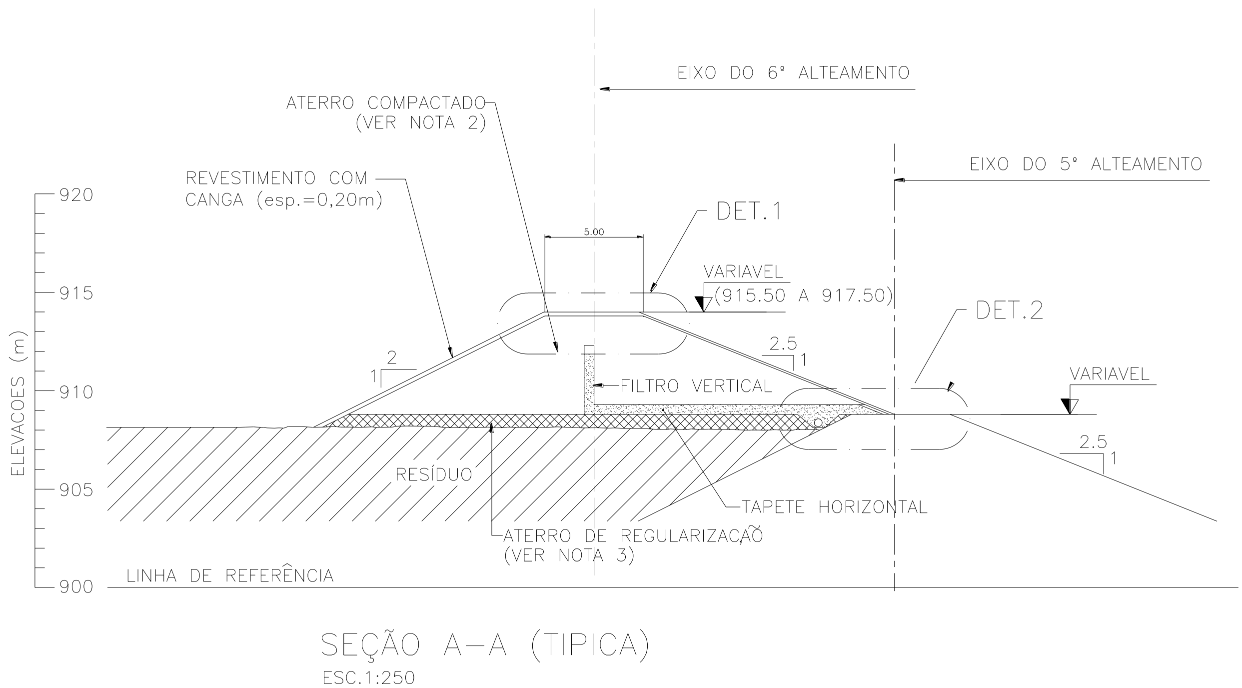

Figure 23: Typical Cross-section of the Sixth Raising[151]

5.8.2.2 Selected Material Parameters

Laboratory tests performed for the Sixth Raising provided unit weight and cohesion results that were considered to be different compared to results from previous raisings and previously performed tests, and unreliable; therefore, they were not used in the analyses for the Sixth Raising.[152] Instead, Tecnosolo elected to use previously selected parameters, summarized in Table 8.

[151] Tecnosolo Sixth Raising Design Files.
[152] Tecnosolo Sixth Raising Design Report.

The undrained shear strength for saturated tailings was calculated as 25 kPa and 30 kPa, respectively, through back-calculation, assuming the FS for the Fifth Raising were between 1.10 and 1.30. For long-term analyses, an undrained strength ratio (s_u/σ'_v) of 0.22 was selected. A shear strength of 50 kPa was used for long-term analyses described as the mid-layer strength. No other basis was provided for the selection of undrained strength ratios.[153]

Table 8: Sixth Raising Material Parameters[154]

Material	Total Unit Weight (kN/m³)	Friction Angle (ϕ)	Cohesion (kPa)	Undrained Shear Strength (kPa)
Silty-clayey fill	19	30º	8.5	-
Tailings – compacted	27.5	40º	0	-
Tailings – medium compact	27	39º	0	-
Tailings – loose to low compact	26	37º	0	50
Tailings – low compaction	26	38º	0	70
Tailings – loose	25	36º	0	30
Residual soils	19	30º	8.5	-
Colluvial soils	17	25º	15	-

5.8.2.3 Internal Drainage

Design drawings indicate the use of an internal drainage system similar to the system for the Fifth Raising. It consisted of a vertical filter, drainage blanket, and PVC pipes. The vertical filter (0.8-m thick) was planned within the core starting 1 m below the Fifth Raising crest to the tailings, where it would tie into a 0.5-m-thick drainage blanket, creating an L-shape internal drainage filter. The vertical filter and drainage blanket were designed to be constructed using sinterfeed.

Horizontal perforated PVC pipes with a 150-mm diameter were planned every 50 m, which would be placed perpendicular to the Fifth Raising and direct water into a reinforced concrete trough at the toe of the raising (Figure 24).[155]

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153 Tecnosolo Sixth Raising Design Report.
154 Modified from Tecnosolo Sixth Raising Design Report (translated from original Portuguese).
155 Tecnosolo Sixth Raising Design Files.

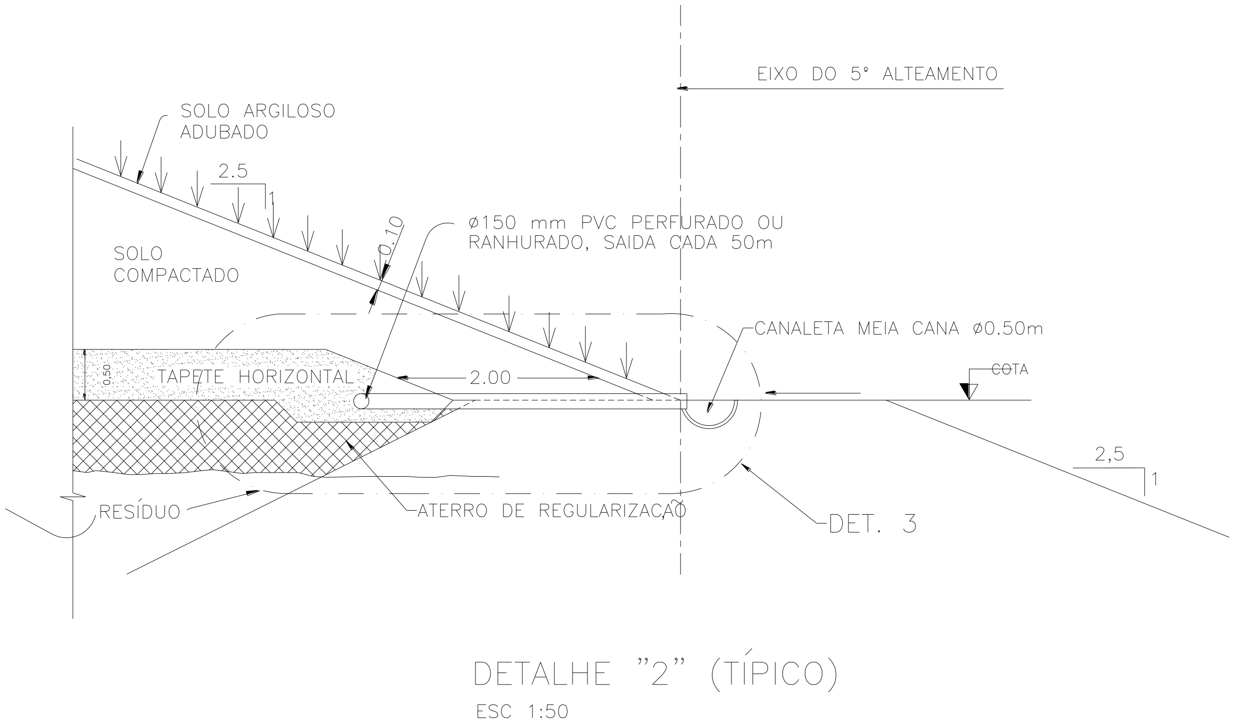


Figure 24: Internal Drainage and Surface Water Management Detail [156]

5.8.2.4 Surface Water Management

The perimeter surface water drainage system was designed to route overflow to Dam VI, similar to the Fourth and Fifth Raisings (Figure 24). The design report describes in detail the hydrology studies performed to assess the conditions that the surface water system was designed to manage. The report uses the 1,000-year return frequency storm (i.e., peak flow, $Q_1 = 1.458$ m³/s) as the basis of design, as well as an assumed flowrate from the pumping of tailings into the reservoir (i.e., $Q_2 = 0.222$ m³/s) resulting in a total peak flow (Q) of 1.680 m³/s.[157]

5.8.2.5 Geotechnical Stability

Geotechnical stability analyses for the Sixth Raising were performed using the Bishop and Janbu methods implemented in the program XSTABL for both short- and long-term conditions (i.e., undrained and drained). The approach is similar to that used for the Fifth Raising in that the undrained shear strength values were estimated by back-calculation assuming a certain level of stability for that existing condition. For the Sixth Raising, however, the shear strengths were

156 Tecnosolo Sixth Raising Design Files.
157 Tecnosolo Sixth Raising Design Report.

increased by approximately 15% based on observations of the existing conditions of the Fifth Raising and the expectation that the shear strengths would likely be higher than assumed in the Fifth Raising design document.[158]

Stability analyses provided for the Sixth Raising included FS calculations for the existing Fifth Raising, Fifth Raising after construction of the Sixth Raising, and Sixth Raising. The existing Fifth Raising had computed FS of 1.46 and 1.26 for typical and elevated piezometric surfaces, respectively. The FS for the Fifth Raising, assuming the Sixth Raising was constructed and full, were calculated as 1.38 and 1.27 for typical and elevated piezometric surfaces, respectively. The Sixth Raising FS were calculated as 1.46 and 1.39 for typical and elevated piezometric surfaces, respectively. The stability of the final height of the dam (i.e., to El. 945 m msl) was also evaluated under a number of conditions, with a resulting calculated FS of 1.46 for favorable groundwater conditions and 1.39 for unfavorable groundwater conditions.[159]

The Sixth Raising stability analyses acknowledge the presence of high piezometric conditions, the impact of high piezometric conditions on the calculated FS, the critical effect of having a well-formed zone of drainage material (e.g., beach) behind the dam, and the sensitivity of the undrained stability analysis results to the value of undrained shear strength ratio. The analyses used assumptions of the piezometric levels, the nature and extent of the beach, and the undrained shear strengths of the tailings and berm materials. The report acknowledged that FS were lower than required (i.e., $FS_{required} \geq 1.5$)[160] and notes that a higher FS could be achieved from lower piezometric conditions.[161]

5.8.2.6 Construction Specifications

Technical specifications are not available for the Sixth Raising construction.

5.8.3 Instrumentation

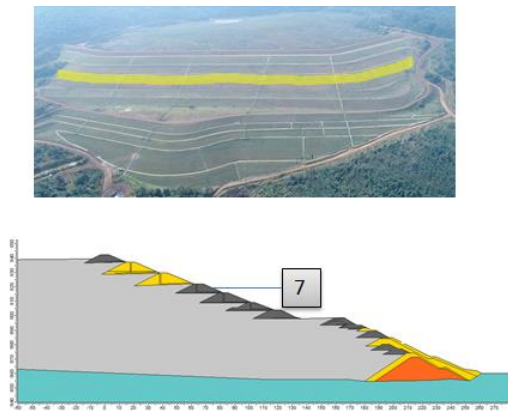
The design called for the installation of 10 piezometers during the Sixth Raising to monitor Dam I.[162] The Sixth Raising design report also discusses monitoring and the water levels in the existing wells for which the calculated FS against slope failure would be 1.30 or 1.15.[163]

158 Tecnosolo Sixth Raising Design Report.
159 Tecnosolo Sixth Raising Design Report.
160 Tecnosolo Sixth Raising Design Report.
161 Tecnosolo Sixth Raising Design Report.
162 Tecnosolo Sixth Raising Design Report.
163 Tecnosolo Sixth Raising Design Report.

5.8.4 Complications and Variances

No indications of any complications were noted in the reports issued after the Sixth Raising design.

5.9 <u>Seventh Raising: 2003</u>



The Seventh Raising (location shown on the index figure to the right) was designed by Tecnosolo and consisted of one stage to raise the crest of Dam I 6 m to an average elevation of 922.5 m msl (El. 921.5 m msl and 923.5 m msl, on the right and left abutments, respectively).[164] The design documents indicate similar assessments as those performed for the Sixth Raising. Available information includes stability and hydrology analysis reports, stability analysis figures, minutes of construction-phase meetings, and as-built drawings of the berm. A stability calculation report was provided,[165] along with the figures showing the results of the stability analyses.

5.9.1 Geotechnical Investigations

Three SPT boreholes (SP1, SP2, and SP3) and geotechnical tests were performed to estimate soil parameters of the Seventh Raising.[166] The upstream subsurface in the beach area was characterized by 5 m of fine tailings (silty sand-sized) of medium density overlain by a protective layer of clayey silts underlain by a 10- to 20-m-thick sand-sized tailings layer with loose to medium compaction.[167]

5.9.2 Design Approach

5.9.2.1 Geometry

The Seventh Raising was designed as a single 6-m stage from El. 916.5 m msl to 922.5 m msl (Figure 25). The top elevation of the right abutment (i.e., El. 921.5 m msl) of the raising was specified to be 2 m lower than the left abutment (i.e., El. 923.5 m msl).[168]

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[164] Stability Report, Seventh Raising (Tecnosolo 2001) ("Tecnosolo Seventh Raising Stability Report").
[165] Tecnosolo Seventh Raising Stability Report.
[166] Tecnosolo Seventh Raising Stability Report.
[167] Tecnosolo Seventh Raising Stability Report.
[168] Tecnosolo Seventh Raising Stability Report.

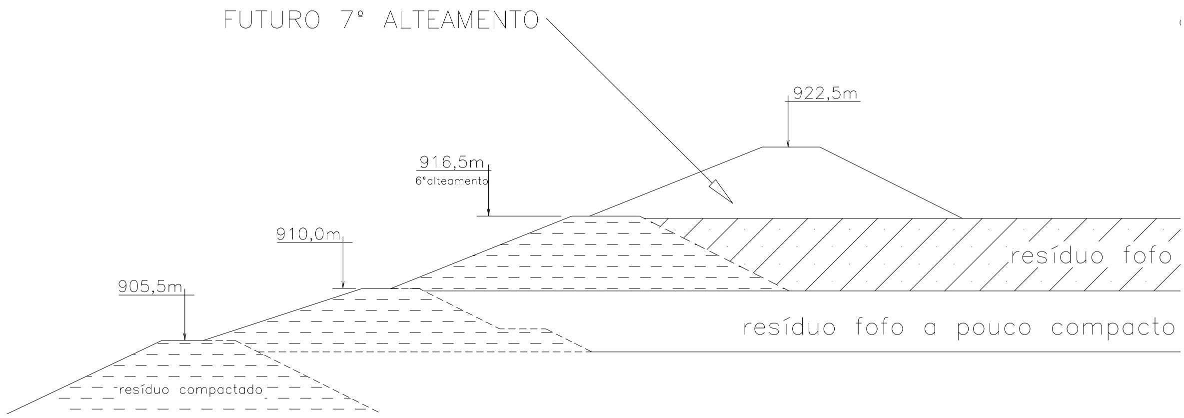


Figure 25: Typical Cross-section of the Seventh Raising [169]

5.9.2.2 Selected Material Parameters

Selected material parameters for the Seventh Raising are presented in Table 9. Geotechnical parameters were established based on prior investigations and studies, tested remolded samples, and boreholes specific to the Seventh Raising.

Unit weight and cohesion results from the Seventh Raising investigations were not used due to "…the discrepancy with values adopted in the past (…) [which] does not justify the use of these values."[170] Therefore, previously selected parameters were used (Table 9). The design also stated that undrained shear strength was difficult to determine, and therefore, data from the Sixth Raising investigation were used. Strength gains due to placement of the Seventh Raising were accounted for in underlying layers by assuming an undrained shear strength ratio of 0.22. Using this assumption, at intermediate depths, a S_u of 50 kPa was calculated and used for the stability analyses.[171]

The stability analysis report states that the "most critical condition in terms of stability can occur" along the right abutment due to the soft and saturated conditions of the foundation tailings.[172] The tailings near the right abutment were characterized as potentially liquefiable (e.g., by dynamic loading) due to the loose and saturated state.[173]

[169] Tecnosolo Seventh Raising Design Files.
[170] Tecnosolo Seventh Raising Stability Report (translated from original Portuguese).
[171] Tecnosolo Seventh Raising Stability Report.
[172] Tecnosolo Seventh Raising Stability Report (translated from original Portuguese).
[173] Tecnosolo Seventh Raising Stability Report.

Table 9: Seventh Raising Material Parameters[174]

Material	Total Unit Weight (kN/m³)	Friction Angle (ϕ)	Cohesion (kPa)	Undrained Shear Strength (kPa)
Silty-clayey fill	19	30	8.5	-
Tailings – compacted	27.5	40	0	-
Tailings – medium compaction	27	39	0	-
Tailings – loose to low compaction	26	37	0	50
Tailings – low compaction	26	38	0	-
Tailings – loose	25	36	0	30
Residual soils	19	30	8.5	-
Coluvial soils	17	25	15	-

5.9.2.3 Internal Drainage

Design drawings indicate a vertical filter (0.8-m thick) and horizontal filter (0.5-m thick) made of sinterfeed constructed in an L shape (Figure 26). The outflow system connects to perforated PVC pipes, which run to perimeter drainage channels similar to the Sixth Raising. The design drawings and report do not describe the required parameters of the drainage layer materials. No filtration calculations are provided in the report to assess the ability of the compacted soil layer to be filtered by the sinterfeed.

174 Modified from Tecnosolo Seventh Raising Stability Report (translated from original Portuguese).

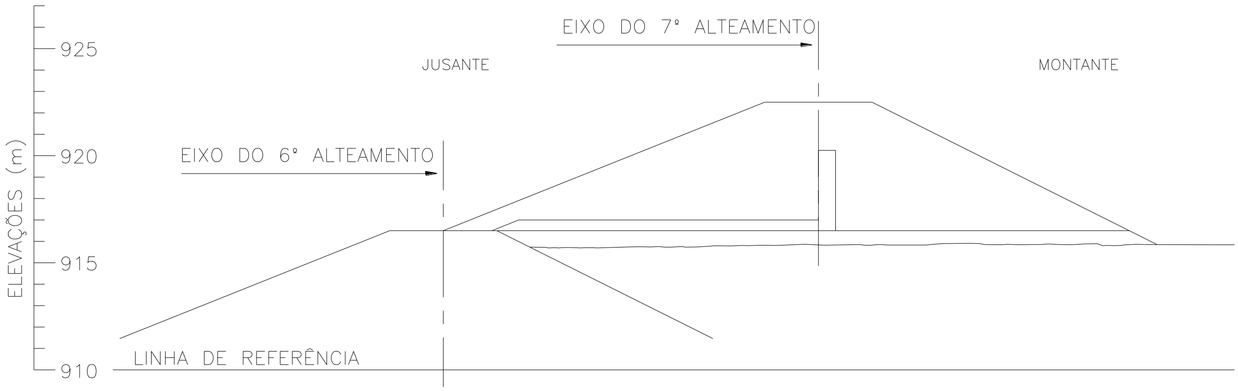


Figure 26: Seventh Raising Drainage Features[175]

5.9.2.4 *Surface Water Management*

A water redirect system was constructed along the right abutment of the dam.[176] The design was based on the same methodologies and input parameters that were used for the Fourth through Sixth Raisings. The features of the system were developed to address the need to route water from an elevation higher than previous stages to Dam VI, which required more robust structures as the raisings increased the pond elevation.

5.9.2.5 *Geotechnical Stability*

Geotechnical stability analyses were performed using Spencer's method, implemented using the computer program UTEXAS3. Stability was evaluated for the final condition of the Sixth Raising during the Seventh Raising design. A FS of 1.57 was calculated under normal piezometric conditions, while a FS of 1.18 was calculated for a less favorable piezometric condition defined by a full reservoir and piezometric levels reaching vertical drains and horizontal drainage blankets of each raising).[177] Based on these analyses, it was recommended that piezometric monitoring activities should continue throughout the Seventh Raising.

The stability of the Seventh Raising was evaluated for drained and undrained conditions. The design report explained that undrained conditions were analyzed because construction methods could create undrained conditions within the raising, and a minimum allowable FS of 1.20 was established.[178] A FS of 1.57 was calculated in the undrained analysis, assuming that normal

[175] Tecnosolo Seventh Raising Design Files.
[176] Hydrological and Hydraulic Studies Report, Seventh Raising (Tecnosolo 2001) ("Tecnosolo Seventh Raising Hydrology Report").
[177] Tecnosolo Seventh Raising Stability Report.
[178] Tecnosolo Seventh Raising Stability Report.

piezometric elevations were maintained.[179] However, during elevated piezometric conditions (i.e., phreatic surface at the internal drainage features), the calculated FS was 1.30. Additionally, undrained analyses adopting the most favorable value of S_u = 30 kPa resulted in an FS of 1.25 for the upstream slope and FS < 1 for the downstream slope near the right abutment.[180] This condition was addressed in the report recommendations stating that design features for the Seventh Raising, when constructed, should prevent such conditions (e.g., construction should be done slowly and carefully to avoid an increase in pore pressures).[181] A FS greater than 1.5 was considered achievable if construction methods maintained drained conditions throughout.[182]

5.9.2.6 Construction Specifications

No technical specifications are available for the Seventh Raising berm construction.

5.9.3 Instrumentation

The design called for the installation of piezometers and water level indicators to monitor Dam I. The design report and drawings[183] show that 10 piezometers[184] were proposed along five cross-sections through the dam. A piezometer monitoring plan was issued with the Seventh Raising design to establish the water levels that would trigger action levels for calculated FS against a slope failure of 1.30 or 1.15.[185]

5.9.4 Complications and Variances

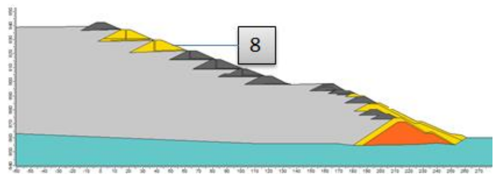
A construction complication anticipated in the design relates to the impact of soft materials near the right abutment during construction of the raising in that area, but no mention of problems associated with this issue during construction was noted in the available documents.

Meeting minutes during the construction period, dated July 15, 2002, state that Ferteco (which by this time had been acquired by Vale) requested that a third-party contractor (Construtora Impar) clean the drains due to rapid water rise adjacent to the dam to prevent saturation of the drainage system which could negatively impact stability.[186] Furthermore, meeting minutes from

179 Tecnosolo Seventh Raising Stability Report.
180 Tecnosolo Seventh Raising Stability Report.
181 Tecnosolo Seventh Raising Stability Report.
182 Tecnosolo Seventh Raising Stability Report.
183 Tecnosolo Seventh Raising Design Files.
184 Tecnosolo Seventh Raising Stability Report.
185 Tecnosolo Seventh Raising Stability Report.
186 Minutes of July 15 Meeting with Construtora Impar, July 15, 2002, Seventh Raising.

July 25, 2002, state that Ferteco again requested that Construtora Impar maintain the drain outfalls to prevent structural issues.[187]

5.10 Eighth Raising: 2004



The Eighth Raising (location shown on the index figure to the right) was designed by Tecnosolo and consisted of one stage to raise the crest of Dam I to an average elevation of 929.5 m msl. The available design documents indicate the designer's intent to follow the approach of the Seventh Raising. Available information includes a geotechnical report with figures, a hydrology analysis report, and as-built drawings of this raising. The design documents are dated December 2003, and the berm was constructed in 2004.

5.10.1 Geotechnical Investigations

The Eighth Raising design considered results from all available previous investigations at the site, in addition to two geotechnical investigations.[188] The first additional investigation dated August 2003 included four SPT boreholes located on the tailings beach and the collection of four samples that were analyzed for grain size distribution and Atterberg limits.[189] The second investigation was performed in 2003, when eight boreholes (SPDAM 02 through SPDAM 05 and SPDAM 07 through SPDAM 10) were advanced 30.45 m.[190] The borehole locations are not indicated in the available documentation.

The subsurface stratigraphy identified during the investigation for the Eighth Raising consisted of the following from the beach surface:

- Reject tailings, classified as silty sand and sandy silts that generally were present in alternating layers of low resistance and high resistance likely resulted from the tailings placement procedures. Tecnosolo noted that the tailings resistance, as measured by standard penetration tests (SPTs), did not generally increase with depth as expected.
- Compacted soils from previous raisings.

187 Minutes of July 25 Meeting with Construtora Impar, July 25, 2002, Seventh Raising.
188 Geotechnical Studies Report, Eighth Raising (Tecnosolo 2003) ("Tecnosolo Eighth Raising Geotechnical Studies").
189 Table of Test Results, Eighth Raising (Tecnosolo).
190 Tecnosolo Eighth Raising Geotechnical Studies.

- Foundation soils, generally described as alluvium or residual deposits of silty clay, clayey silt, sandy silt, and silty sand.[191]

5.10.2 Design Approach

5.10.2.1 Geometry

The Eighth Raising was designed as one 7-m stage from an average elevation of 922.5 m msl to 929.5 m msl. The upstream and downstream slopes were consistent with previous designs.

5.10.2.2 Selected Material Parameters

There were numerous material test results reported and boreholes summarized in the Eighth Raising documents, including a discussion of a "most realistic maximum possible section of the dam"[192] with reference to the design cross section included in the report;[193] however, there was no discussion on the selection of the material parameters for use in the design analyses.

5.10.2.3 Internal Drainage

The internal drainage system of the Eighth Raising consisted of a 0.8-m-thick vertical filter and 0.5-m-thick drainage blanket constructed in an L shape.[194]

5.10.2.4 Stability

Stability analyses for the Eighth Raising are not available.

5.10.2.5 Construction Specifications

Construction specifications for the Eighth Raising are not available.

5.10.3 Instrumentation

A discussion on instrumentation is not found in the documents available for review.

5.10.4 Complications and Variances

Documentation discussing complications and variances for the Eighth Raising are not available.

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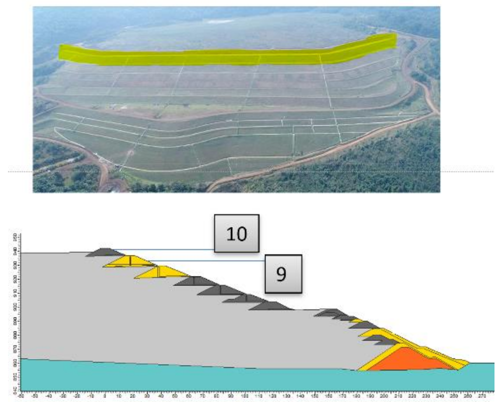
[191] Tecnosolo Eighth Raising Geotechnical Studies.
[192] Tecnosolo Eighth Raising Geotechnical Studies (translated from original Portuguese).
[193] Tecnosolo Eighth Raising Design Files.
[194] Tecnosolo Eighth Raising Design Files.

5.11 Ninth and Tenth Raising: 2008 and 2013



The Ninth and Tenth Raisings (location shown on the index figure to the right) were designed by Geoconsultoria and consisted of one stage each to raise the crest of Dam I by 7 m and 5 m to El. 937 m msl and 942 m msl, respectively. The Ninth and Tenth Raisings are discussed together because of the similar nature and timing of the development of their design investigations, analyses, and design documents. Comprehensive design documents are available for both raisings, including design drawings, stability and hydrology analysis reports, laboratory testing program details and results, field investigation plans and results, stability analysis figures, and a laboratory report of beach tailings test results. In addition, the record included an operation manual, technical specifications for the construction for of each raising, and a substantial photographic record of the conditions near the time of construction. No construction inspection reports are available. The design documents are dated August 2006. The Ninth Raising was constructed in 2008, and the Tenth Raising was constructed in 2013.

5.11.1 Geotechnical Investigations

The geotechnical investigations for the Ninth and Tenth Raisings were carried out in 2005 and 2006.[195] The test results are included in the Field and Laboratory Test Summary Report in Appendix B. The field work consisted of the following:

- boreholes with SPT tests on the beach and the crest of the dam, including the collection of undisturbed Shelby tube and Osterberg samples;
- Cone Penetration Tests with pore pressure dissipation (CPTu) and vane shear strength tests;
- in situ density tests of the tailings;
- collection of disturbed and intact block samples from the tailings beach area;
- auger boreholes in the Dam I and borrow areas;
- installation of piezometers and water level gauges (“INAs”) on the beach and in Dam I; and
- installation of two inclinometers (INC-01 and INC-02).[196]

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[195] Design Report, Ninth and Tenth Raising (Geoconsultoria 2006) (“Geoconsultoria Ninth and Tenth Raising Design Report”).

[196] Geoconsultoria Ninth and Tenth Raising Design Report.

The maximum depth of the CPTu tests was 26 m, which did not penetrate the full depth of the tailings. Laboratory tests performed on the samples and the test results are also provided in Appendix B; tests performed during the investigation included grain size distribution, natural moisture content, density and relative density, Atterberg limits, permeability, standard Proctor, triaxial compression, and consolidation.

The test results provided significant additional information on the nature and variability of the tailings that were deposited before the design of the Ninth Raising. The observations on the geotechnical investigations provided in the design report are summarized below:

- The sedimentary process intended in the beach area (i.e., to produce segregation of the materials with coarser materials close to the dam and finer materials further from the dam) was not occurring, particularly on the right abutment of the dam.
- Raisings were constructed using compacted tailings, with the first three raisings covered with clay (as discussed earlier).
- Tailings were placed hydraulically either on the beach or submerged in the impoundment, resulting in layers of different densities and strengths.
- At a few locations, high piezometric pressure conditions were identified at depths greater than 25 m below the surface by CPTu tests. The cause of these conditions was unknown but interpreted by the designer to be due to a coarse layer underlying fine tailings, which results in a confinement condition in the bottom of the valley.
- The CPTu tests indicate the presence of layers where undrained behavior likely controls shear strength, but the report concludes that these layers are few and of limited extent (i.e., beneath the Fourth Raising and a few meters below the level of tailings at the time of design, which would have been approximately at El. 928 m msl).
- Vane shear tests confirmed undrained behavior within Dam I in distinct areas (i.e., not across the entire tailings deposit).
- Infiltration test results indicate that tailings permeability ranges from 1×10^{-7} m/s to 1×10^{-6} m/s. In deeper portions of Dam I, where the dam was mixed with soils, permeability ranges from 1×10^{-8} m/s to 1×10^{-7} m/s.
- Density generally decreases and fines content increases, as distance from the beach increases.
- The occurrence of plastic materials is limited (note: this comment was not specific to the tailings or the dam).
- The tailings range from fine sand to silt-sized with specific gravity ranging from 4 to 5. Dry unit weights of samples collected in the beach area ranged from 1.85 tons per cubic

meter (t/m^3)to 2.29 t/m^3. Relative compaction densities generally ranged from 50% to 60%.[197]

5.11.2 Design Approach

The design approach for the Ninth and Tenth Raisings is illustrated on Figure 27 and called for upstream construction. The raisings were designed to be constructed of tailings, overlain with a layer of lateritic gravel along the crest of the dam and upstream slope, and vegetation on the downstream slope.

During the initial stages of the design, concerns were expressed by the designer regarding stability due to the water levels interpreted to be artesian in some of the piezometers, and due to uncertainties about the construction methods and materials used for the initial stages of the dam, leading to the installation of two inclinometers.[198] The Ninth Raising design was developed with the expectation that it would be the last raising unless future evaluations of stability, tailings re-processing, environmental permitting, and the schedule for the development of an additional tailings storage facility allowed future raisings of Dam I. During the conceptual design of the Ninth and Tenth Raisings, an alternative was considered of constructing a new dam (Barragem 3 – Santana) to replace Dam I and operated while Dam I was mined, after which time Dam I would be put back into operation.[199] There is no indication that this alternative was pursued.

5.11.2.1 Geometry

The Ninth and Tenth Raisings design report indicates that the upstream and downstream slopes were designed having 2H:1V and 2.4H:1V, respectively, similar to previous raisings.[200] The crests of the raisings were to be 5-m wide at El. 937 m msl and 942 m msl, respectively.

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197 Geoconsultoria Ninth and Tenth Raising Design Report.
198 Geoconsultoria Ninth and Tenth Raising Design Report.
199 See Conceptual Study on Disposal of Tailings, Ninth and Tenth Raising (Geoconsultoria 2006) (“Geoconsultoria Ninth and Tenth Raising Disposal Study”); Preliminary Design, Ninth and Tenth Raising (Geoconsultoria 2005).
200 Geoconsultoria Ninth and Tenth Raising Design Report.

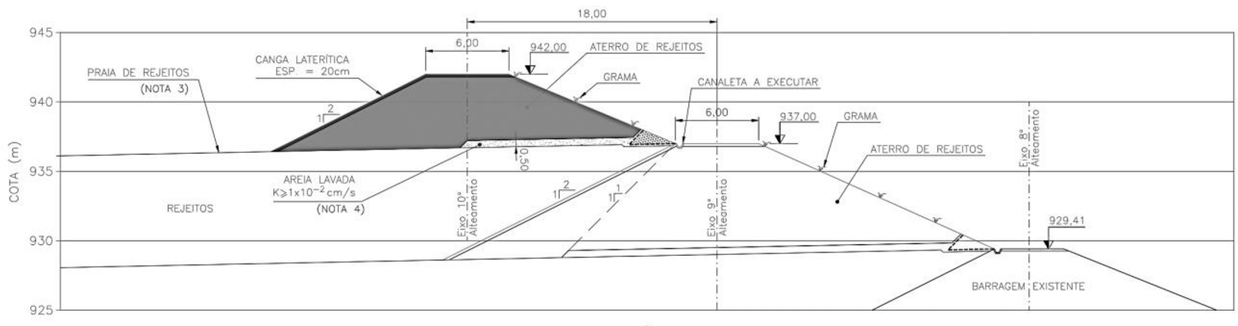

Figure 27: Ninth and Tenth Raisings As-Designed[201]

5.11.2.2 Selected Material Parameters

Material parameters used in the analysis of slope stability under drained conditions were determined to be the same as those used in the design of previous raisings.[202] The designers determined that the field and laboratory test results from the investigation performed as part of the Ninth and Tenth Raisings confirmed the results of previous investigations. Undrained shear strength parameters were derived from CPTu and FVT tests.[203]

5.11.2.3 Internal Drainage

The internal drainage design was comprised of a 0.5-m-thick horizontal drainage layer extending along the downstream half of the base of the raising. The drainage layer is described as being constructed of "…washed sand, finished with a gravel drain…"[204] although the characteristics of the sand and gravel materials were not specified and a filter calculation is not provided for the drainage layer materials. The design document states that an L-shaped configuration was not adopted for the design of the Ninth and Tenth Raisings.[205] However, later project documents[206] show the Ninth Raising being constructed having an L-shaped drain configuration. The vertical drain was not included in the original design because the dam material was believed to have moderate permeability, would not impound water along its upstream slope, and would be more efficient at drawing down the phreatic surface.[207]

[201] Geoconsultoria Ninth and Tenth Raising Design Files.
[202] Geoconsultoria Ninth and Tenth Raising Disposal Study.
[203] Geoconsultoria Ninth and Tenth Raising Design Report.
[204] Geoconsultoria Ninth and Tenth Raising Disposal Study (translated from original Portuguese).
[205] Geoconsultoria Ninth and Tenth Raising Design Report.
[206] 2018 TÜV SÜD Periodic Safety Review.
[207] Geoconsultoria Ninth and Tenth Raising Design Report.

5.11.2.4 Surface Water Management

The surface water management system for the Ninth and Tenth Raisings was designed as an extension of the previously existing surface water management system. A decant system was designed to control the water elevation in the reservoir and, in the process, the width of the beach. The designed decant system consisted of three concrete towers connected to a 310-m-long rectangular concrete structure beneath the right abutment to convey water from Dam I to a downstream channel that outflowed to Dam VI. A drainage system along the right abutment was constructed to allow tailings upstream to dewater to improve the formation of the beach. The intent was to stabilize the area to maintain adequate stability of the dam throughout the Ninth and Tenth Raising development.[208]

The storm water system for the downstream face of the dam was designed to convey runoff from the dam face resulting from a 25-year storm event. The runoff system design consisted of cast-in-place concrete channels both parallel to the crest of the existing Eighth Raising berm and down the face of the Fourth through Eighth Raisings. These channels and drop inlets were designed to convey storm water runoff to the existing storm water system that had been constructed during previous raisings.[209]

5.11.2.5 Geotechnical Stability

Geotechnical stability analyses were performed for 10 cross-sections using the computer program SLIDE. As part of the stability analyses, a seepage model was created using SLIDE to estimate the phreatic conditions that would exist in the dam after the dam had been raised and tailings filled behind the dam. The seepage model was calibrated based on existing piezometric data and in situ and laboratory permeability tests.

Sensitivity analyses were performed to evaluate the influence of a number of conditions, including the presence of varying beach conditions, the presence of a layer of undrained materials of limited extent beneath the Fourth Raising, and an elevated phreatic surface within the dam. Analyses were also performed to identify the height of the phreatic surface, under drained conditions, that would result in a FS less than 1.5. Stability analyses also were performed for the excavation of the surface water management system near the right abutment. All analyses produced FS values near or greater than 1.5.[210]

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208 Geoconsultoria Ninth and Tenth Raising Design Report.
209 Geoconsultoria Ninth and Tenth Raising Design Report.
210 Geoconsultoria Ninth and Tenth Raising Design Report.

The potential for static liquefaction was also evaluated for the Ninth and Tenth Raisings.[211] The analyses were performed assuming saturated conditions in the tailings and were based on the results of stress-strain curves from laboratory triaxial shear strength tests on intact samples of tailings. The laboratory tests were interpreted to be predictive of dilative conditions, leading to the report's conclusion that the material had a low liquefaction potential. The design report does not specify which of the laboratory tests were used in the analysis.

5.11.2.6 Site Preparation

The project documents include a discussion of construction methods that describe the procedures to be followed to construct the raisings.[212] The construction specifications describe the clearing of an access road for drainage system construction, preparation of surfaces on which the raisings were to be constructed, and construction of an area near the left abutment that could be used as a borrow pit if tailings were not available.

5.11.2.7 Construction Specifications

The Ninth and Tenth Raisings were designed to be constructed of compacted tailings. A lateritic gravel facing was designed to be constructed on the crest and upstream slopes, while grass was planned on the downstream slopes. Sand for the filter layers was specified as having no more than 5% fines and at least 20% material having less than a 0.42-mm diameter. Construction phase testing of material parameters was specified for tailings (including grain-size, density, standard Proctor, and permeability) and for sand drainage layer material (daily grain size tests). Field relative density tests were specified at a frequency of not less than one per 500 m^3 for placed tailings and one per 200 m^3 for sand drainage material.[213] Flexibility appears to have been provided to the construction and inspection teams regarding raising configuration when implementing the design in the field: "The Inspectors may, with the assistance of the Designers, increase or reduce the width and quotes of the foundations and slopes of the landfill embankments and may make any revisions to the dam's sections that they deem necessary in order to obtain safe and economical structures, within the concept presented in the design documents and drawings."[214]

5.11.3 Instrumentation

The design called for installation of several types of instruments, including:

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211 Liquefaction Evaluation, Ninth and Tenth Raising (Geoconsultoria 2008).
212 Construction Specifications, Ninth and Tenth Raising (Geoconsultoria 2006) ("Geoconsultoria Ninth and Tenth Raising Construction Specs").
213 Geoconsultoria Ninth and Tenth Raising Construction Specs.
214 Geoconsultoria Ninth and Tenth Raising Construction Specs (translated from original Portuguese).

- open Casagrande-type piezometers, to be installed soon after the raising was constructed at depths of about 5 m and within the tailings beneath the downstream toe of the Tenth Raising berm (e.g., PZC-39 in Figure 28);
- water-level indicators, to be installed after the raising was constructed within the drainage blanket material (e.g., INA-37 in Figure 28) to monitor for the presence of water under pressure within this layer;
- surface-control markers, installed at the crest of the berm; and
- water-level gauges within the decant pond.[215]

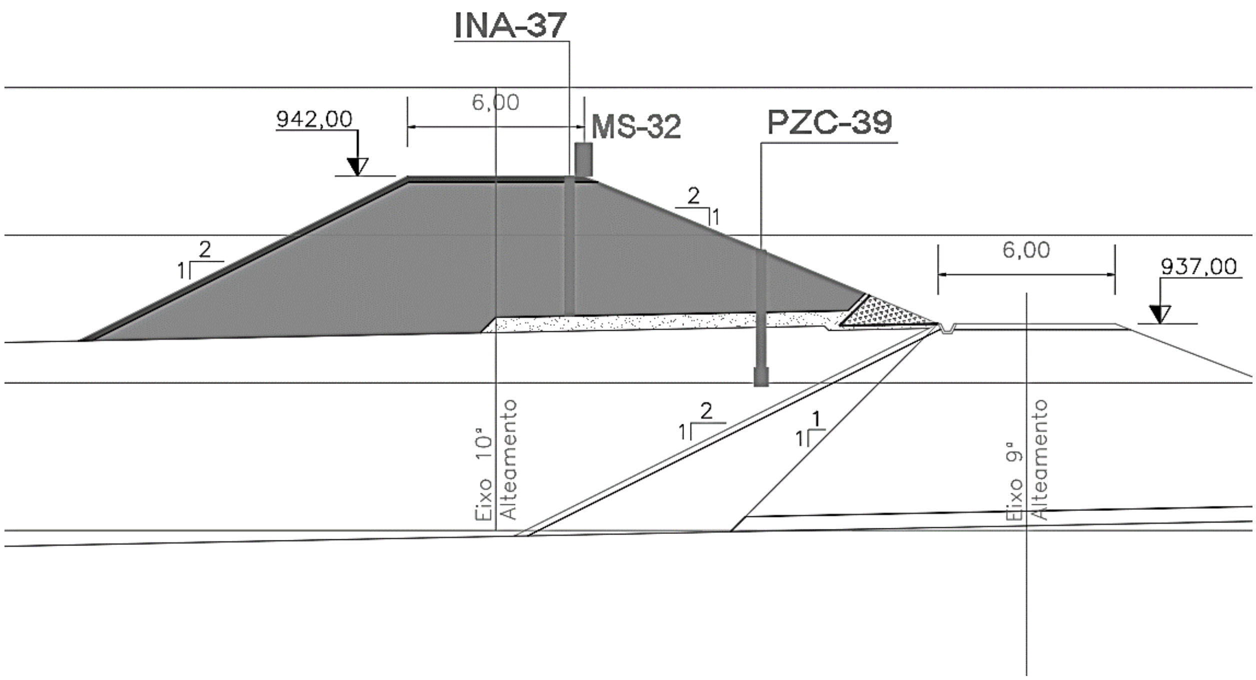

Figure 28: Ninth and Tenth Raising Piezometer and Water-Level Indicator Installation Plan[216]

5.11.4 Complications and Variances

5.11.4.1 As-Built Cross-section and Test Results

The as-built documentation for the Ninth and Tenth Raisings shows that they were built generally consistent with the design and without a vertical internal drainage feature in either the Ninth or Tenth Raising berm (Figure 29). Compliance reports and test results for the materials used during construction to evaluate compliance with the testing requirements of the

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215 Geoconsultoria Ninth and Tenth Raising Design Report.
216 Ninth and Tenth Raising As Built Design Files.

specifications were not available. Also, no reports of materials used for construction were found; however, subsequent reports indicate that the Ninth Raising was constructed using compacted clay instead of the tailings that were called for in the design.[217]

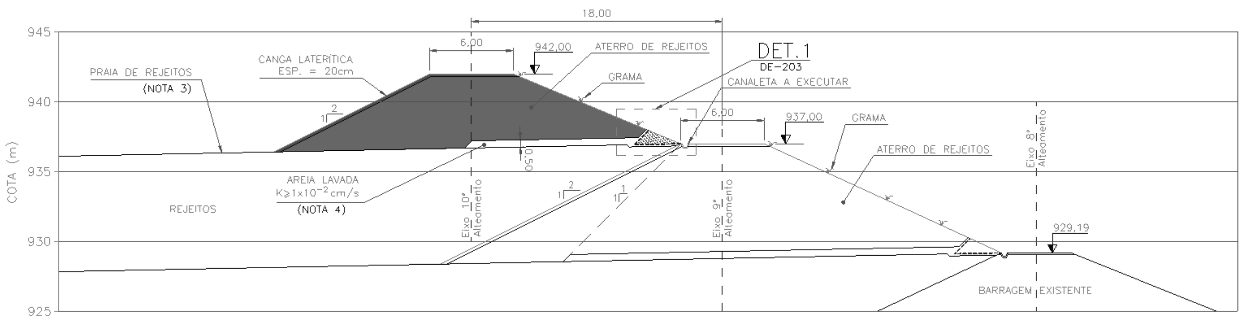

Figure 29: Ninth and Tenth Raising As-Built[218]

5.11.4.2 Tailings Filling Rate

The design report indicated that the anticipated filling rate of the impoundment behind the dam would be approximately 2 m to 3 m per year after the construction of the Ninth Raising.[219] However, the actual filling progressed at 6 m per year (Figure 30).[220]

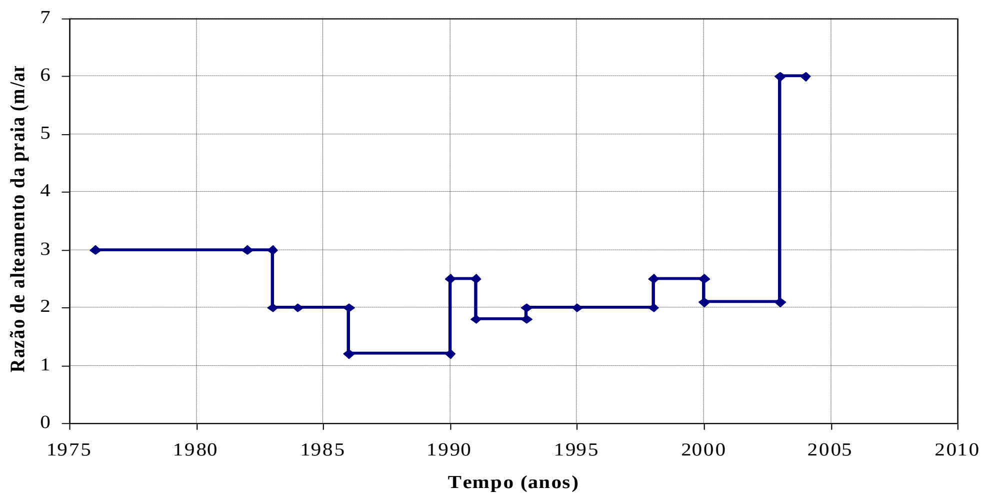

Figure 30: Ninth and Tenth Raising Design Loading[221]

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[217] 2018 TÜV SÜD Periodic Safety Review.
[218] Ninth and Tenth Raising As-Built Design Files.
[219] Geoconsultoria Ninth and Tenth Raising Design Report.
[220] Geoconsultoria Ninth and Tenth Raising Design Report.
[221] Geoconsultoria Ninth and Tenth Raising Design Report.

5.11.4.3 Seeps and High Piezometric Surface

Seeps and high phreatic surface conditions were noted in the design documents for the Ninth and Tenth Raisings. The documents noted that water levels within Dam I were known to fluctuate and that previous studies underestimated phreatic conditions within the dam. The report indicated that the water level was within 10 m of the downstream slope near El. 904 m msl and 898 m msl then dropped to 15 m below the slope, and that at El. 874 m msl the water level increases again, moving closer to the downstream slope.[222] A shallow phreatic surface also was observed between El. 899 m msl and 904 m msl (Third and Fourth Raisings, respectively) and near the crest of the Starter Dam at El. 874 m msl. The design report also indicates that seeps were observed along the downstream slope near the toe of the Fourth Raising near the time that the design was being developed, and that water levels measured within the dam and the size of the tailings beach suggest that water should only be evident in the drainage structures of the Fourth Raising.[223] Details were not provided of the actual locations of the seepage areas. The report suggested that vertical percolation (i.e., vertical gradients) of groundwater flow through Dam I and the tailings had led to higher water levels compared to those that would be anticipated in the piezometers.[224]

5.11.4.4 High Piezometric Head Conditions

The safety evaluation indicated concern due to the high piezometric conditions that developed near the Third and Fourth Raisings (i.e., El. 899 m msl and 904 m msl, respectively) and on the Starter Dam slope[225] due to a confining condition at the base of the tailings deposit at those elevations.[226] The design report concluded that the depth at which these conditions occur did not raise concern related to stability.[227]

6. POST-CONSTRUCTION EVENTS AT DAM I

6.1 <u>Introduction</u>

After the Tenth Raising was completed in 2013, no further construction occurred to raise the elevation of Dam I. As discussed in Section 2.2, Dam I stopped receiving tailings in July 2016. After that time, activities were conducted and events occurred that may provide information

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222 Geoconsultoria Ninth and Tenth Raising Design Report.
223 Inspection Report, Ninth and Tenth Raising (Geoconsultoria 2007) ("Geoconsultoria Ninth and Tenth Raising Inspection Report").
224 Geoconsultoria Ninth and Tenth Raising Design Report.
225 Dam I Safety Evaluation, Ninth and Tenth Raising (Geoconsultoria 2007).
226 Geoconsultoria Ninth and Tenth Raising Design Report; Piezometer Evaluation, Ninth and Tenth Raising (Geoconsultoria 2007).
227 Geoconsultoria Ninth and Tenth Raising Design Report.

related to the condition and performance of Dam I as of the date of the failure. Activities and events that occurred after completion of the Tenth Raising are described in this section related to surface-water management activities (Section 6.2), installation of deep horizontal drains (DHPs) (Section 6.3), the occurrence of seepage at the dam (Section 6.4), and drilling that was performed in the several months prior to the failure (Section 6.5).

6.2 Surface Water Management System

By July 2016, when Dam I ceased receiving tailings, construction of the surface water management system for the dam had been completed to its general configuration on the day of the failure. Between July 2016 and the date of the failure, activities associated with the surface water management system involved managing the surface water in the impoundment and maintaining the surface water management features on the face of the dam so that they would carry water off the dam. These activities are discussed below. Separate activities that were conducted to manage the level of water within the tailings are discussed in Section 6.3.

6.2.1 Management of Surface Water in Impoundment

From the time that the Tenth Raising was completed until Dam I ceased to receive tailings in July 2016, the depth of surface water in the impoundment was managed at the decant tower using stop-logs (i.e., barriers placed in the decant to control the level of water retained in the impoundment), which could be either inserted or removed in the decant tower to raise or lower the elevation of water in the impoundment area, as needed to meet the Dam I operational goals.[228] In or around May 2016, presumably in anticipation of the decision to cease tailings disposal, the volume of water in the impoundment was significantly reduced, leaving a shallow depth of standing water in the impoundment.[229]

After tailings disposal ceased in July 2016, verification of a low surface water depth in the impoundment and maintenance of the surface water drainage system were the subject of regular inspections. Annual technical safety audit reports from 2016, 2017, and 2018, for example, call for maintaining low levels of standing water in the impoundment.[230] This was achieved in various ways. Firstly, on July 25, 2016, a protection berm was extended by 50 m around the decant tower, providing control of the flows of water into the tower and allowing better management of water in the impoundment.[231] Secondly, as of July 2016, the water surface was

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[228] Dam I Operation Manual (Geoconsultoria 2016); Tailings Dam I Dam Operation Manual (Geoconsultoria 2013).

[229] Extraordinary Audit Technical Report Regular Safety Inspection Report (Geoconsultoria 2016) ("Geoconsultoria 2016 Technical Safety Audit").

[230] Geoconsultoria 2016 Technical Safety Audit; Regular Safety Inspection Report: Dam Technical Safety Report (Tractebel 2017) ("Tractebel 2017 Technical Safety Audit"); Regular Safety Inspection Report: Dam Technical Safety Report (ANM) (TÜV SÜD 2018) ("TÜV SÜD 2018 Technical Safety Audit ANM").

[231] Email correspondence, July 25, 2016.

located far from the decant tower, preventing drainage to the towers by gravity flow. Dewatering efforts therefore required pumping water from the standing water area to the decant tower, from which surface water flowed to Dam VI. Therefore, to maintain the low surface water levels, a pump and piping system was designed and installed after the dewatering of the impoundment was completed in mid-2016. The system consisted of a pump system connected to a discharge pipe that routed water to the decant tower (Figure 31).[232] A backup pump was required in case the primary pump failed. The system was only put into operation in late 2018.

Figure 31: Impoundment Dewatering System After Tailings Disposal Ceased[233]

In early 2018, a study was conducted to identify all the sources of water entering the impoundment area.[234] The goal of this effort was to divert all of the water that would otherwise enter the impoundment area. The study identified a spring that was contributing flow to the dam, reportedly at a measured flowrate of 1.17 m^3/hr.[235] In July 2018, the diversion of water from the spring to the decant tower was completed. It appears that construction activities on the diversion system continued and expanded over the course of the next several months, as discussed further in Appendix D. Later in 2018, a sump and channel were planned as a preventive and complementary structure to the spring capture system to divert water from the dam;[236] but it is unclear whether the system was constructed prior to the failure.

Beginning in 2018 and until January 2019, there were reports of maintenance requirements and repairs to the impoundment surface water dewatering system.[237] In the several months leading up to the failure, these included various reports of the pumps at Dam I not working and being

232 Report of Anomalies January 1, 2018-January 27, 2019 (Vale 2019) ("Vale 2018-19 Anomaly Reports"); TÜV SÜD 2018 Technical Safety Audit.
233 TÜV SÜD 2018 Technical Safety Audit (ANM).
234 Descriptive Report of the Water Diversion Work of the Reservoir of Dam I (Vale 2018) ("Vale 2018 Water Diversion Report").
235 Vale 2018 Water Diversion Report.
236 TÜV SÜD 2018 Technical Safety Audit (ANM).
237 TÜV SÜD 2018 Technical Safety Audit (ANM).

repaired, as well as reports of disconnection of the piping used to route water from the pumps to the decant tower and then repair of the piping.[238]

6.2.2 Management of Surface Water on Downstream Face of Dam I

The design of Dam I included surface water drainage features to collect seepage and rainfall runoff from the face of Dam I. As discussed in Section 5, most raising construction included an extension of the surface water drainage system. The resulting drainage system consisted of a set of concrete-lined channels, including channels at the toe of each raising to route water laterally, and channels down the face of the dam to route the drainage to the creek at the base of the dam. The purpose of the system was to route water from the face of the dam in a manner that prevented overtopping of the channels and ponding of water in or near the channels.

Performance issues with the surface drainage system on the dam were observed and reported several times throughout the history of Dam I.[239] Annual technical safety audits of the dam conducted after the completion of the Tenth Raising also identified performance issues with the channels, including lack of proper drainage, damage to the channel concrete lining, and excessive vegetation and sediment from channels.[240] The mitigation efforts included increased monitoring, reduction of clogging, and drainage improvements. In particular, in 2018, several channels were cleared of silt, grading was performed to improve inappropriate drainage in several areas, vegetation was removed where needed to prevent flow from being restricted and a drain was unclogged.[241] Also as part of the efforts associated with enhancing the management of surface water, two channels were reconstructed between September and December of 2018. One of these channels was located near the dam's left abutment; the other channel was located in the vicinity of DHP 15 (see Section 6.3).[242]

6.3 <u>Deep Horizontal Drains</u>

As discussed in Section 5, drainage of the dam and the tailings had been a consideration in the design of the facility since the early stages of planning of the dam. Lateral drainage blankets were incorporated into the berm designs beginning in the Second Raising and continuing to the Tenth Raising. Beginning in 1993, lateral pipes referred to as "sub-horizontal drains" were

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[238] August 2018 Periodic Dam Performance Analysis (Vale 2018); Vale 2018-19 Anomaly Reports.

[239] See, for example, Regular Safety Inspection Report: Dam Safety Technical Report (Pimenta De Avila Consultoria 2014) ("Pimenta De Avila 2014 Technical Safety Audit").

[240] See Regular Safety Inspection Report: Dam Safety Technical Report (Pimenta De Avila Consultoria 2013) ("Pimenta De Avila 2013 Technical Safety Audit"); Pimenta De Avila 2014 Technical Safety Audit; Regular Safety Inspection Report: Dam Safety Technical Report, (Pimenta De Avila Consultoria 2015) ("Pimenta De Avila 2015 Technical Safety Audit"); Geoconsultoria 2016 Technical Safety Audit; Tractebel 2017 Technical Safety Audit; TÜV SÜD 2018 Technical Safety Audit (ANM).

[241] Vale 2018-19 Anomaly Reports.

[242] Photographic Report of Drainage Channel Work (Cogel 2018).

installed in the dam as part of the berms and were monitored for flow.[243] Throughout the design and construction of the Ninth and Tenth Raisings from 2006 to 2013, there are references to field piezometric and seepage conditions that were different than those expected during the design stage, and contemporaneous stability analyses noted high piezometric conditions.

In early 2018, Vale implemented a new drainage concept in the form of deep horizontal drains (referred to as "DHPs" based on their Portuguese acronym). Planning documents for the DHPs predicted that the installation of 11 DHPs at elevation 898 m msl and 21 DHPs at elevation 880 m msl would produce drawdowns of between 8.3 m and 12 m within about two years after their installation and that DHPs would be needed only at El. 880 m msl to achieve the design objective drawdowns.[244]

The plan to install the DHPs included the following:[245]

- using air pressure to advance drilling to the tailings without the use of water;
- installing steel casing and then advancing the hole using a tri-cone bit inside the casing;
- introducing water, since machine torque alone would be insufficient to extend the hole beyond 60 m in depth, with water return directed to a channel downslope of the installation area;
- installing the horizontal drain; the first 5 m of the drain was then grouted with a cement/bentonite mixture.

Installation of DHPs began in February 2018. By June 2018, 14 DHPs had been installed (DHP 01 through DHP 14).[246] Figure 32 shows the locations of the DHPs as installed.

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243 Drain monitoring information is provided in the Instrumentation Report in Appendix C.
244 New Simulation of Water Removal From Dam I (MDGeo 2018).
245 DHP Implementation Procedure (AlphaGeos 2018).
246 Report of Installation of First 13 DHPs.

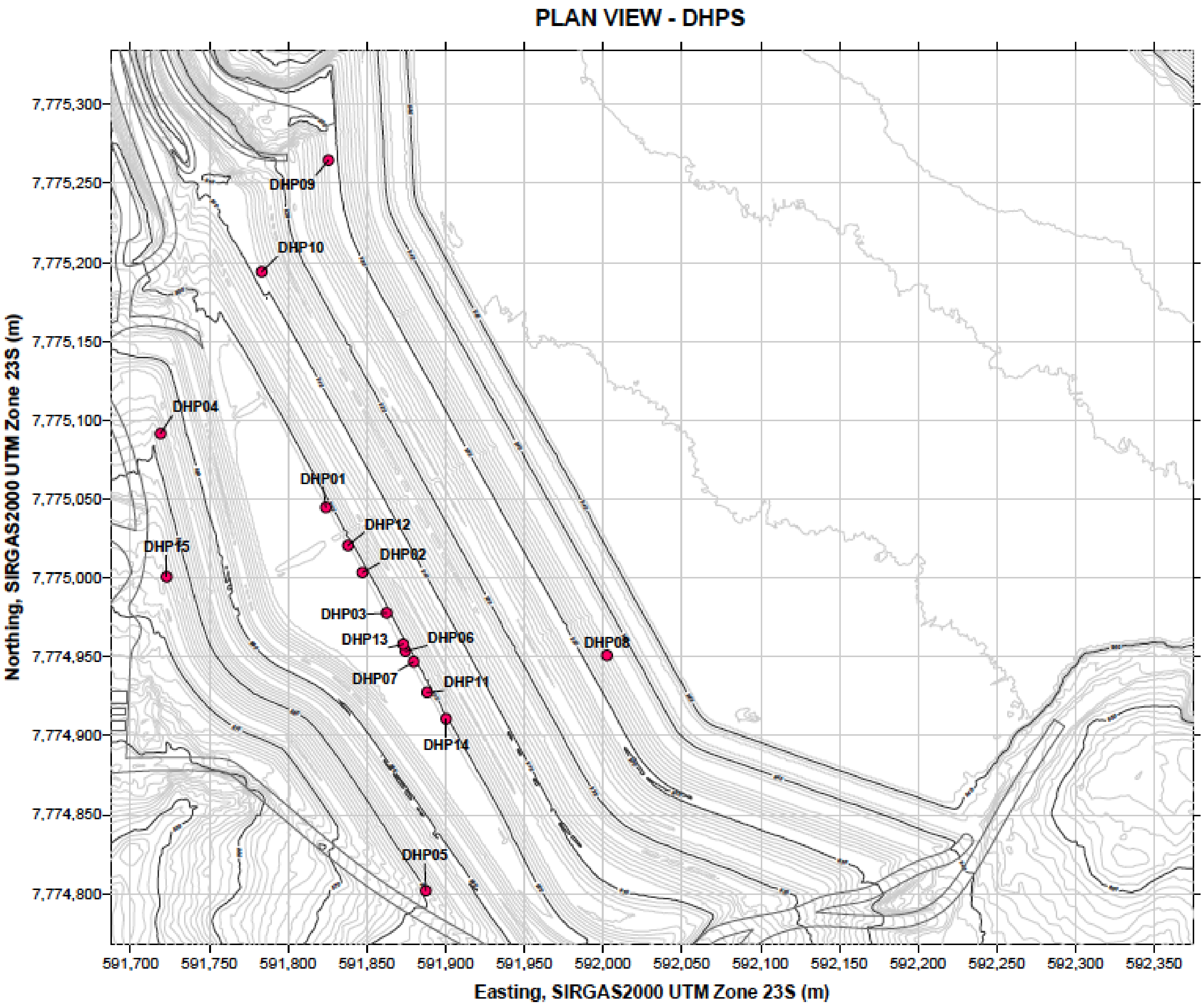

Figure 32: DHP Locations[247]

An installation report for the DHPs describes the installation procedures and provides the logs for the 13 DHPs that were installed by the end of May 2018.[248] The logs indicate that none of the DHPs reached the original design length of 100 m. The typical length of the installed DHPs was about 60 m and the maximum reported length was 81 m. Advancing the drilling through the berm was achieved with an air pressure of up to 600 kPa, with additional water pressure of up to 400 kPa applied in the tailings. The only reported issue with the installation of the first 14 DHPs

247 Figure 9-1 of Appendix C – Historical Instrumentation Data.
248 Report of Installation of First 13 DHPs.

is that DHP 6 was only installed to a depth of about 30 m with no perforated PVC pipe installed.[249]

6.3.1 Installation of DHP 15

The installation of DHP 15 began on the morning of June 11, 2018 by drilling a pre-bored hole through the Starter Dam. Drilling stopped temporarily between 12 and 1 pm for a lunch break.[250] When drilling resumed, problems were encountered that included: (i) loss of pressure in the borehole as it crossed from the berm to the tailings; (ii) loss of water recirculation in the borehole, preventing advancement of the borehole; and (iii) apparent collapse of the hole around the drill rod and loss of the drill rod in the hole. At some time between 2:00 pm and 4:30 pm, localized discharge of water and fines was observed near a surface channel approximately 15 m away and 7 m higher than where DHP 15 was being installed, as shown in Figure 33.[251] The ground based radar detected small but rapid deformation in an area slightly above DHP 15, as described in Appendix D. Subsequent analysis of nearby seismograph records performed by the Panel is described in Appendix I.

Figure 33: Location of Observed Seepage Near DHP 15[252]

[249] June 20, 2018 Technical Memorandum on DHP Installation; DHP Installation Logs.
[250] Daily Work Summaries of DHP Installation (June 2018).
[251] Daily Work Summaries of DHP Installation (June 2018).
[252] Dam I Presentation on DHP15 Installation (Vale 2018). Text added by authors.

When the flow was observed, DHP installation activities ceased and the hole was grouted on the same day.[253] Technicians worked to mitigate the incident, through the removal of seepage water and use of sandbags. The sandbags were stacked up to approximately 80 cm high and four stacks wide. Piezometric levels in nearby piezometers PZM-7 and PZM-9 were monitored approximately every 30 minutes between 3:30 pm and 7:25 pm.[254] During this time period, the levels were observed to rise by approximately 0.6 m and 3.5 m, respectively,[255] but by 7:25 pm that same day, they had returned to normal levels.[256] Small increases of 0.3 m or less were also recorded at PZM-16 and PZC-24.[257]

On June 14, during a field inspection, one blocked pipe was discovered under the soil near the location where DHP 15 was installed. Another blocked pipe was found approximately 20 m from the location of DHP 15. When the pipe 20 m away from DHP 15 was unblocked, the flows at DHP 15 significantly diminished, as did the flow at the pipe found near DHP 15.[258] By June 14, 2018, fines were no longer observed.[259]

By June 15, 2018, corrective measures had been implemented, the seepage was controlled, and work was underway to temporarily repair the nearby damaged drainage channel.[260] For five days following the incident, the team issued a daily safety report. No additional DHPs were installed following the DHP 15 incident. Figure 34 shows an image from one week prior to the failure, highlighting the locations of DHP 15 and the observed seepage on June 11, 2018.

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253 June 20, 2018 Technical Memorandum on DHP Installation (Vale 2018); Daily Work Summaries of DHP Installation (June 2018).
254 June 11, 2018 Dam I Regular Safety Report (Vale 2018).
255 Dam I Presentation on DHP15 Installation (Vale 2018).
256 June 11, 2018 Dam I Regular Safety Report (Vale 2018)
257 Email correspondence, June 14, 2018. This information is not part of the plots of piezometer data provided in Appendix C.
258 June 20, 2018 Technical Memorandum on DHP Installation (Vale 2018).
259 June 14, 2018 Dam I Regular Safety Report (Vale 2018).
260 June 15, 2018 Dam I Regular Safety Report (Vale 2018).

Figure 34: Image from January 18, 2019 Drone Footage

6.4 Seepage

As discussed in Section 5, the design of Dam I included a system of lateral drains and subsurface drains in the raisings, to collect and route water to the surface of the dam. Flows from these drainage features were collected in surface drainage channels and pipes, and were monitored by means of flow meters; the recordings from those flow meters are provided in Appendix C. The drainage features were intended to prevent seepage through the raisings from wetting the downstream face of the dam, which could lead to localized or general instability of the face of the dam.

Seepage was observed and reported several times throughout the history of Dam I. For example, seepage was reported as having been observed in the Second Raising as early as 1983,[261] and seepage was reported as having been observed in the Starter Dam during the design of the Fourth Raising in 1995. Seepage was also reported at the time of the design of the Ninth and Tenth Raisings, which were completed in 2006; the design report indicates that seepage was observed along the downstream slope near the toe of the Fourth Raising during the time that the Ninth and Tenth Raising designs were being developed.[262]

Annual technical safety audits of Dam I conducted after the completion of the Tenth Raising generally do not identify seepage concerns[263] until a 2018 audit, which indicated seepage

261 Geoconsultoria, Supplementary Technical Review – Stability Analysis Under Undrained Loading Conditions (Geoconsultoria); Fourth Raising Design Report.

262 Geoconsultoria Ninth and Tenth Raising Inspection Report.

263 See Pimenta De Avila 2013 Technical Safety Audit, Pimenta De Avila 2014 Technical Safety Audit, and Pimenta De Avila 2015 Technical Safety Audit, which do not mention seepage concerns. See also *cont'd on next page*

transitioning from "good practices" to "moderate noncompliance" beginning in March 2018.[264] However, anecdotal reports indicate that seepage was a regular occurrence on portions of the downstream face of the dam. In July 2018, a periodic performance evaluation report indicated an observance of moisture near cross-section 3 of the dam during construction of the water divergence system,[265] and a review of anomalies from the period of one year prior to the failure identifies numerous reports of seepage and subsequent seepage mitigation efforts.[266] The mitigation efforts included increased monitoring, reduction of clogging, and drainage improvements.

6.5 Drilling Program Ongoing at the Time of Dam Failure

From September 2018, Vale initiated two projects, including drilling activity, at Dam I. One of the projects was the "As-Is" project, which was a subsurface exploration program that was intended to collect information on the material properties of the dam. The second project involved the collection of information for developing plans for decommissioning the dam, including the installation of new instrumentation. Drilling for the two projects was specified to be performed using percussion drilling where possible and then rotary drilling methods where percussion drilling could not advance the borehole.[267] Where drilling could not be advanced using these drilling methods, coring techniques using HQ-sized (i.e., 96 mm outer diameter) core and double barrel (wireline) methods were specified and used.

The scope of work for each of these projects appears to have been as follows:[268]

- The As-Is project included 19 boreholes to be completed using a combination of drilling methods, including percussion, hollow-stem auger, and rotary drilling in rock with water circulation. Of the 19 boreholes, eight were executed prior to the failure; an additional borehole (B1-SM-21) was in progress on the day of the failure. The drilling logs available for review indicate that drilling was performed using a GEO-115 drill rig, but do not describe the equipment used or the drilling method.[269] No final report of drilling was produced for these boreholes. All of the boreholes that were executed prior to the failure were at the base of the dam into natural soil, not on the dam itself. Accordingly, they are not discussed further.

Geoconsultoria 2016 Technical Safety Audit and Tractebel 2017 Technical Safety Audit, which mention that seepage was not observed.

264 TÜV SÜD 2018 Technical Safety Audit (ANM).
265 July 2018 Periodic Dam Performance Analysis (Vale 2018).
266 Vale 2018-19 Anomaly Reports.
267 Technical Specifications for Geotechnical Field Investigations (TÜV SÜD).
268 TÜV SÜD 2018 Complementary Testing Map; Technical Specifications for Geotechnical Field Investigations (TÜV SÜD).
269 Geocontrole Drilling Logs.

- The investigation conducted as part of the decommissioning project included the completion of three CPTu soundings, four seismic DMT tests, installation of four vertical inclinometers, and drilling of eight boreholes in which vibrating wire piezometers were to be installed. The boreholes that had been executed prior to the failure are identified in Table 10, and their locations are shown on Figure 35. All of the boreholes identified in Table 10 were performed on the dam itself. According to the drilling logs available for review, the drilling was completed using a combination of drilling methods, including percussion and rotary drilling in rock with water circulation using a SDH-400 drill rig with 86 mm diameter borehole.[270]

Table 10: Boreholes Executed or Initiated on Dam I as of Date of Failure[271]

Hole ID	Apprx. Ground El. (m msl)	Depth (m)	El. of Bottom of Hole (m msl)	Borehole Started	Borehole Completed	Comments
INC-03	923	53	869.5	Oct 21, 2018	Nov 3, 2018	
INC-04	923.6	40	854.6	Oct 3, 2018	Oct 9, 2018	
INC-05	895.1	47.5	869.5	Oct 17, 2018	Oct 19, 2018	
INC-06	923	45.5	869.5	Nov 7, 2018	Nov 13, 2018	
B1-SM-12	905	50.5	855	Nov. 22, 2018	Dec. 4, 2018	
B1-SM-07	898.5	41.5	857	Nov. 23, 2018	Nov. 29, 2018	
B1-SM-11	899	28.5	870.5	Nov. 30, 2018	Unknown	Drillers' note does not indicate if or when drilling was completed.
B1-SM-08	905	51.4	854	Dec. 7, 2018	Dec. 20, 2018	
B1-SM-09	929.5	76	853.5	Jan. 7, 2019	Jan. 11, 2019	
B1-SM-13	929	65.5	At or below 863.5	Jan. 21, 2019	N/A (In progress on Jan 25, 2019)	Drilling reported here was in progress on the day of the failure. Drillers' notes are available from one day prior to the failure, but no notes are available from the date of the failure.

[270] Detailed Project for Geotechnical Investigation Services and Annexes (Fugro); Fugro Drilling Logs (October 3, 2018 to January 24, 2019).
[271] Detailed Project for Geotechnical Investigation Services and Annexes (Fugro); Vale Instrumentation Map for Dam I; Fugro Drilling Logs (October 3, 2018 to January 24, 2019).

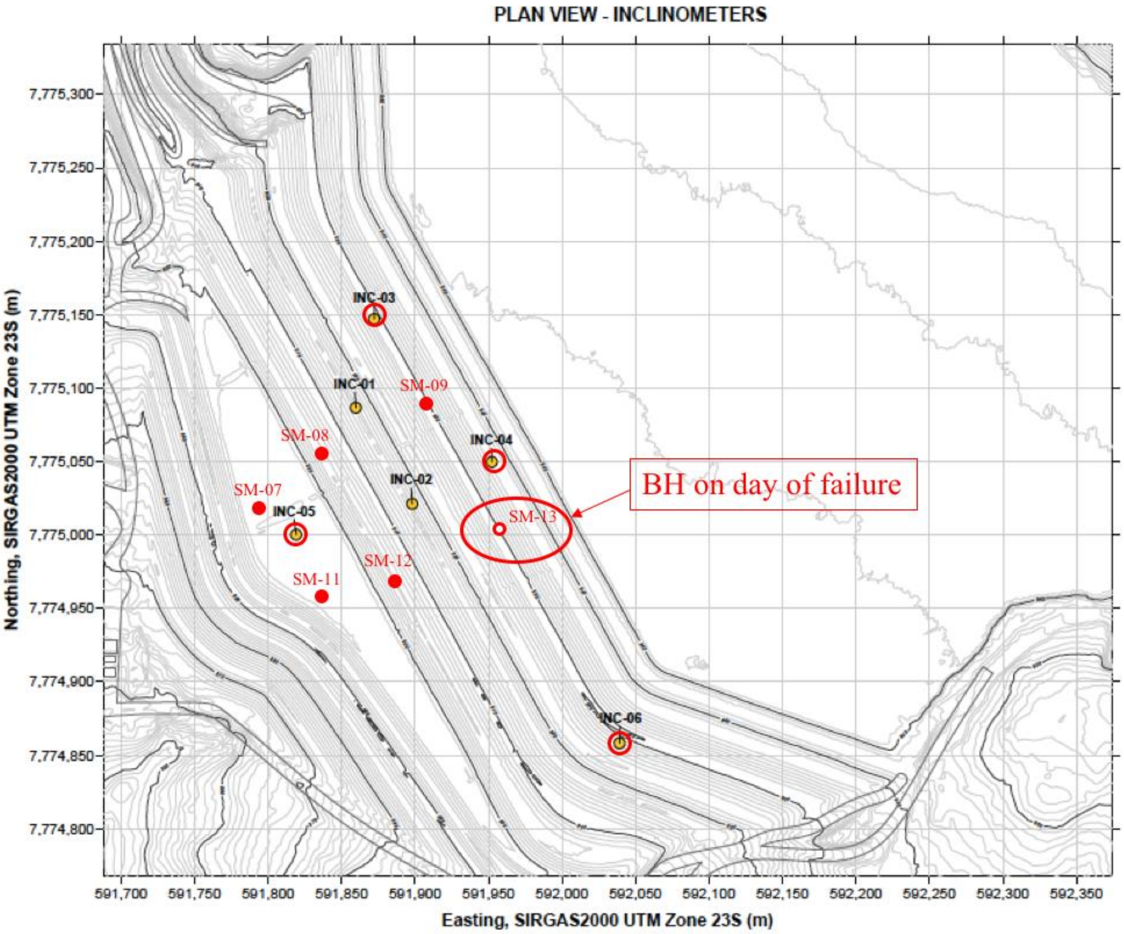

Figure 35: Boreholes on Dam I Between October, 2018 and the Failure[272]

Drilling of B1-SM-13 was in progress on the day of the failure. There is no drill log or other driller's report of the drilling that was performed on the day of the failure. However, drilling logs are available for all prior drilling days, including from the day before the failure. On the day prior to the failure, according to the drilling records,[273] drilling was advanced to a depth of 65.5 m below the ground elevation of approximately 929 m msl to a bottom elevation of approximately 863.5 m msl. This was approximately the elevation of natural soils beneath the tailings. This is the depth and elevation at which the drilling likely started on the day of the failure. Also, records show that the drilling method was changed from percussion at shallow depths to rotary when the water level in the dam was reached; therefore, it is likely that drilling at B1-SM-13 was being performed using rotary methods on the day of the failure. This method involved recirculation of water in the borehole. At the time of the failure, the drillers had been

272 Figure 8-1 of Appendix C – Historical Instrumentation Data, with location of boreholes added.
273 Fugro Drilling Logs (October 3, 2018 to January 24, 2019).

working for most of the morning and had likely advanced the borehole from the previous day's endpoint, possibly by as much as 15 m to a depth of approximately 80 m.

Records of drilling for the other boreholes do not contain any suggestion of problems during drilling. In particular, the nearest borehole to B1-SM-13 was B1-SM-09, which was located at the same elevation on the dam and approximately 50 m towards the right abutment. This borehole was completed about two weeks before borehole B1-SM-13 and to a similar depth. Also, borehole B1-SM-08, which was drilled six weeks before and 50 m away from B1-SM-13, was the only other borehole advanced to a depth that may have encountered soils beneath the tailings similar to those that would likely have been encountered by borehole B1-SM-13. No problems were reported for either B1-SM-09 or B1-SM-08 in the drill logs.[274]

[274] Fugro Drilling Logs (October 3, 2018 to January 24, 2019).