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Abstract

This paper presents a combined field-monitoring and numerical-analysis study. The aim is to examine the influence of reservoir level fluctuations on the settlements of embankment dams by analysing the long-term monitored response of a well-instrumented dam. The field data from three independent monitoring schemes over 25 years are analysed and show a consistent settlement trend with the time. A relevant statistical analysis identifies a strong correlation between the frequency of fluctuations of the settlements and the reservoir level changes which suggests climate-dependent settlements. Finally, hydro-mechanical finite-element analysis of the structural performance of the dam examines the relative effects of soil consolidation and reservoir level changes. It is found that the majority of the dam settlements are due to long-term soil consolidation and that some small fluctuations are due to reservoir level changes.

Keywords	embankment dams; reservoir; monitoring; finite-element analysis; statistical analysis; consolidation
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Dear Editor

Please find attached a manuscript by L. Pelecanos, D. Skarlatos and G. Pantazis entitled "**Influence of reservoir level fluctuations on the long-term settlements of embankment dams".** This work considers the long-term settlement behaviour of embankment dams and focuses in particular on the effects of reservoir level changes. It is a combined field monitoring and numerical analysis study.

A well-instrumented dam, the Kouris dam, which is the largest dam in Cyprus is considered. New previously-unpublished monitoring data (of more than 25 years) are presented, analysed and further processed. A relevant statistical analysis is performed to identify any relation between the frequencies of reservoir level changes and dam settlements. Finally, a nonlinear coupled hydro-mechanical elasto-plastic finite element analysis is performed that simulates the entire stress history of the dams, including layered construction, reservoir impoundment, consolidation, operation and reservoir level changes. It is found that reservoir level changes do have an effect on the dam settlements which also exhibit fluctuations that follow the seasonal variations of the reservoir level. However, it is highlighted that the majority of the total absolute settlements is due to soil consolidation and therefore reservoir level changes contribute to a comparatively lesser extent.

Thank you for your interest in our work.

Best regards

L. Pelecanos

D. Skarlatos

G. Pantazis

Influence of reservoir level fluctuations on the long-term settlements of embankment dams

L. Pelecanos, D. Skarlatos, G. Pantazis

Highlights:

1. The effect of seasonal reservoir level changes on the settlements of embankment dams is investigated
2. Long-term (25 years) monitoring data from 3 independent instrumentation sets are analysed and published for the first time
3. Statistical analysis in the frequency domain reveals an identical dominant frequency of reservoir level changes and dam settlement fluctuations suggesting a strong correlation
4. Nonlinear hydro-mechanical FEA shows that soil consolidation has the largest contribution to the total absolute displacements.

Influence of reservoir level fluctuations on the long-term settlements of embankment dams

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Abstract

This paper presents a combined field-monitoring and numerical-analysis study. The aim is to examine the influence of reservoir level fluctuations on the settlements of embankment dams by analysing the long-term monitored response of a well-instrumented dam. The field data from three independent monitoring schemes over 25 years are analysed and show a consistent settlement trend with the time. A relevant statistical analysis identifies a strong correlation between the frequency of fluctuations of the settlements and the reservoir level changes which suggests climate-dependent settlements. Finally, hydro-mechanical finite-element analysis of the structural performance of the dam examines the relative effects of soil consolidation and reservoir level changes. It is found that the majority of the dam settlements are due to long-term soil consolidation and that some small fluctuations are due to reservoir level changes.

Keywords: embankment dams, reservoir, monitoring, finite-element analysis, statistical analysis, consolidation

1. Introduction

Large water-retaining structures such as dams, levees and reservoirs are types of infrastructure which consist an important part of modern society's needs for water supply and public health. These massive structures are designed to operate over a long lifetime and therefore careful initial designs and subsequent monitoring schemes are essential for their stability and people's safety. The long-term behaviour of earth dams is of particular importance for their stability and structural integrity. Moreover, these are associated with several complicated geotechnical processes such as consolidation, creep, reservoir level fluctuations, seismic wave propagation and internal erosion and therefore careful monitoring is required along with relevant subsequent re-assessment of the structure's stress state.

This paper presents a combined study of field monitoring data and finite element analysis of the long-term behaviour of Kouris earth-rockfill embankment dam in Cyprus. This dam is used as a well-documented case study to investigate the long-term operational behaviour of embankment dams primarily due to reservoir level changes, and secondly to soil consolidation and seismic activity. Monitoring data from three independent instrumentation techniques over a period of more than 25 years post-construction are analysed, exhibiting the real behaviour of the dam during its initial operational period. A relevant statistical analysis is performed, which identifies a correlation between the frequency of fluctuations of the dam settlements and the reservoir level and therefore suggests a seasonal dependence of settlements on reservoir level changes. This is complemented by a nonlinear elasto-plastic coupled hydro-mechanical finite element analysis, which quantifies the relative effects of soil consolidation and reservoir level fluctuations on the settlements of the dam.

2. Current state-of-the-knowledge

It has been for long recognised that earth dams experience displacements for a long period after their construction has been completed [1] [2] [3] [4] [5]. Tedd et al. [6] assessed the monitored performance of five instrumented embankment dams in Yorkshire, United Kingdom and reported that the main mechanisms causing dam deformations are: slope instability, internal erosion, fill and foundation consolidation, clay volume change and rapid drawdown.

Kyrou et al. [7] discussed the performance of the Lefkara dam in Cyprus over the first 30 years of its operational life. It was observed that vertical displacements occur at the dam surfaces (mainly at the top of the upstream slope) which tend to stabilise with time. It was also found that the reservoir level fluctuations although they may affect the values of internal pore pressures, they do not seem to have a significant effect on the dam settlements. More recently, Dounias et al. [8] presented field monitoring data from the operation of seven Greek dams and discussed their performance. They showed that vertical displacements (settlements) of the crest and other dam surfaces develop with time and the maximum values are found on the dam mid-crest and away from the abutments. Moreover, they showed that horizontal displacements of the embankment body also develop mainly due to the impoundment of the reservoir and their rate tends to decrease with time. However, their study showed that, in general, the influence of the reservoir level fluctuations did not seem to be very significant on the overall magnitude of crest settlements.

The effects of the upstream reservoir on the behaviour of earth dams during both static [9] [10] and dynamic [11] [12] [13] [14] conditions have been studied widely [15]. Regarding static conditions, it is well accepted nowadays that changes in the reservoir level induce additional displacements in the dam. Rapid-drawdown results in excessive settlements due to the increased pore water pressures in the upstream shell and the absence of the upstream reservoir resisting stress [9]. Tedd et al. [6] and Charles [16] discussed the settlements of Ramsden dam in the U.K. and they showed that rapid-drawdown resulted in excessive downwards displacements, whereas re-filling induced significant upward displacements due to swelling of the upstream rockfill. Of particular interest are dam deformations due to wetting of the upstream dam rockfill. Partially saturated soils may expand (swell) or collapse due to wetting and drying depending on their type and confining stress [17] [18] because of changes in their volume and strength. Therefore, systematic field measurements of the effect of reservoir fluctuations on the settlements dams have been performed [4] [19] [20] [21] [22] [23] [24] [25] in order to explore the relationship between raising reservoir level and dam settlements. It was shown by Pytharouli & Stiros [26] that the frequency of fluctuations is related to the frequency of reservoir level changes which suggests a strong relation between the two.

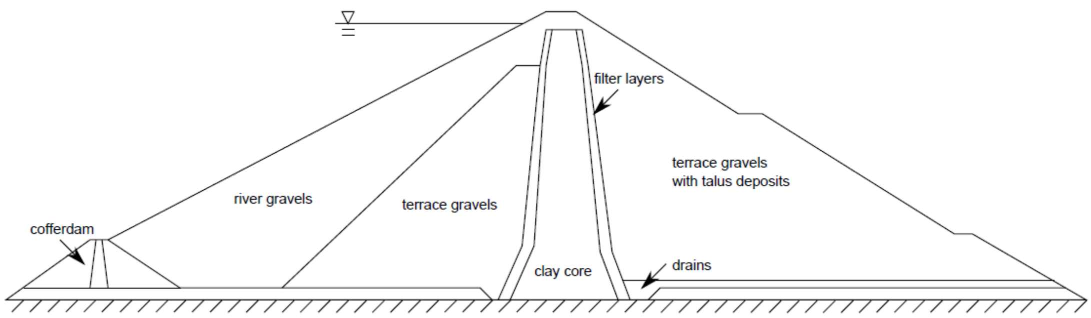
Several attempts were made in previous years in order to numerically analyse the observed behaviour of earth and rockfill dams and further investigate the mechanics of their deformations [27] [28] [29] [30] [31] [32] [33] [34] [35] using numerical and constitutive models of a wide range of sophistication. In particular, different studies taking into account consolidation, creep and collapse reservoir-induced deformations are reported in the literature. As far as long-term consolidation settlements are concerned, a systematic study by Lollino et al. [36], examined the monitored behaviour of Pappadai dam in Italy and numerically analysed its response during construction, reservoir impoundment and operation. They used coupled hydro-mechanical elasto-plastic analysis and it was found that soil consolidation induces long-term settlements. Gikas & Sakellariou [37] studied the long-term settlement behaviour of Mornos dam in Greece by using monitored data and relevant finite element analyses. Their analyses adopted simple elasto-plastic Mohr-Coulomb soil constitutive models and were able to reproduce the field measured data to a decent extent. Finite element analyses with more advanced creep constitutive models were performed by Karstunen & Yin [38] to investigate the time-dependent behaviour of Murro test embankment in Finland using an elasto-viscoplastic model with consolidation, anisotropy and destructuration. Their numerical results were compared with field measurements of settlements and pore water pressures and they exhibited a reasonably well agreement, in particular regarding the delayed dissipation of excess pore pressures. The long-term response of Shuibuya concrete-face rockfill dam (CFRD), which is the tallest dam of its kind in the world, was investigated by Zhou et al. [39]. Time-dependent deformation, dam construction and reservoir impoundment were taken into account using the Duncan & Chang [40] E-B model and creep effects. It was found that the deformation modulus from the back-analysis was smaller than that from the laboratory tests. Finally, Chai et al. [41] analysed numerically the behaviour of an embankment on soft Ariaka clay in Saga, Japan using laboratory and field test data performing Class-A and Class-C predictions. However, although they used critical state models in their simulations they concluded that their Class-A predictions which lacked adequate field data did not provide a good agreement with the measured settlements due to the overestimation of subsoil yield stress and underestimation of the soil compressibility.

The case of reservoir-induced deformations is another interesting issue in the static deformations of earth dams [42]. As mentioned earlier, partially saturated soils are subject to swelling or collapse upon wetting and drying, mainly depending on their type and confining pressure. More specifically, in the case of earth dams, reservoir impounding may result in collapse settlements due to the inundation-collapse behaviour of rockfill. Early attempts to numerically address this issue were performed by Nobari & Duncan [43] who analysed a dam under dry and saturated conditions and simply applied the difference as nodal forces on the dry analysis. Similar approaches were subsequently followed by a number of researchers [44] who tried to reproduce laboratory experiments of wetting samples by reducing the stresses upon wetting through the use of a collapse coefficient. Recently, Mahinroosta et al. [20] attempted to advance this approach by relating the value of collapse coefficient to soil saturation. The case of Beliche dam in Portugal is a representative example of the effects of collapse settlements. Naylor et al. [45] performed finite element analyses of Beliche dam in Portugal to predict its long-term behaviour after construction. Field data were not available then, but they carried out nonlinear (K-G model) total stress numerical analyses with model parameters derived from back-analysis of triaxial and oedometer laboratory tests in order to provide an insight into the (then) future settlements of the dam. Further analyses were later performed by Naylor et al. [44] when field data of the dam performance became available in order to back-analyse its observed behaviour. Reservoir impounding and associated collapse settlements were approximately taken into account by using the approach of Nobari & Duncan [43] and critical-state elasto-plastic models were adopted in their simulations. That work was able to reproduce the observed settlements with a fair agreement but it still under-predicted the downstream dam slope settlements. Subsequently, Alonso et al. [46] elegantly extended the work of Naylor et al. [45] [44] on Beliche dam using appropriate critical state constitutive models [18] which could properly take into account partially saturated behaviour and collapse settlements due to wetting [17] and rainfall infiltration. They managed to reproduce satisfactorily the observed dam response and their main argument was that both reservoir level fluctuations and rainfall have contributed to dam settlements.

3. Kouris dam

Kouris dam is a zoned earth-rockfill dam in Cyprus operated by the Cyprus Water Development Department (WDD). It is the tallest and largest in capacity dam in Cyprus and serves as the main water storage facility in the country. It was built during 1984-1988 as part of the Cyprus Southern Conveyor Project for water supply and irrigation for the arid parts of the country [47].

Its embankment consists of a central clay core of low permeability, followed by thin layers of fine and coarse filters. The upstream shell consists of terrace gravels, which are adjacent to the upstream filters and then river gravels covered by rip rap on the upstream dam slope with a small cofferdam at the upstream dam toe. The downstream shell consists in its entirety by terrace gravels with talus deposits, which rest on a thin drainage gallery. Its crest is 570m long and the embankment is 112m high with the highest level of its reservoir at 102m, allowing for a clear freeboard of about 10m and its total reservoir capacity is 115 million m^3. Figure 1 (a) and (b) illustrate schematically the cross-sectional and plan views of the dam respectively.



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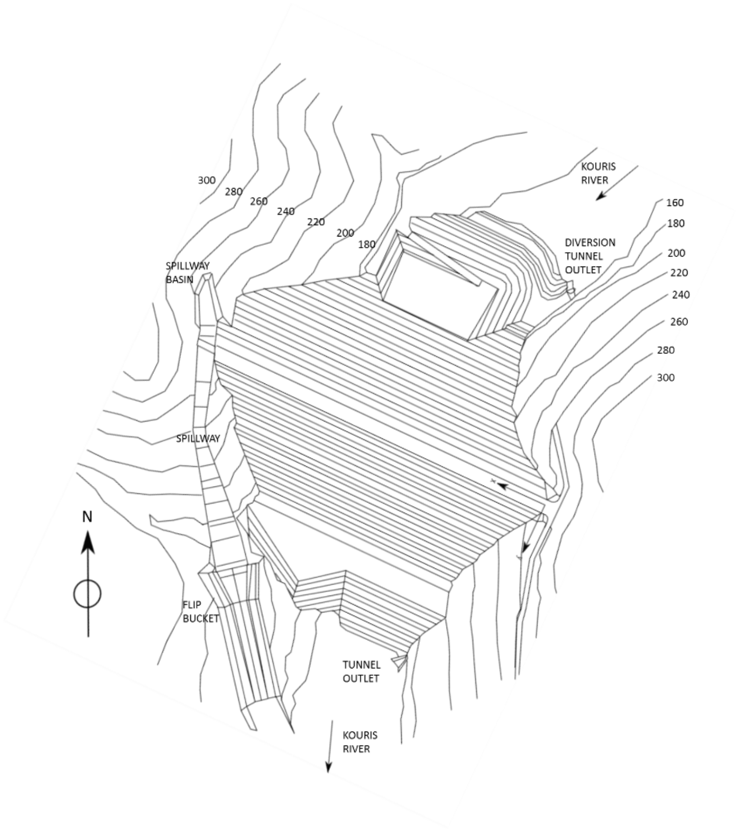

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Figure 1. Geometry of the dam: (a) cross-sectional view and (b) plan view of the dam and surrounding valley (After WDD (38)).

4. Instrumentation

Three independent types of instrumentation were installed on the dam structure and surrounding canyon to provide monitoring data regarding the deformations of the dam: (a) Embankment Crest Movement Indicators (ECMI), (b) a vertical geodetic network and (c) a three-dimensional (3D) geodetic network.

The first instrumentation network, installed in 1991, consists of an observation pillar, which serves as a fixed point and six embankment crest movement indicators (ECMI) which were installed at the time of construction. The measurements of these points are being carried out (irregularly, monthly or bi-monthly) since 1990 by the Cyprus WDD [48]. The horizontal distance and the height difference of each ECMI from the pillar was determined using a Leica TC1101 total station, which provides accuracy of ±2mm ±2ppm for the distance measurements and ±1" for the angle measurements. Figure 2 (a) shows the position of the ECMIs on the dam and the pillar.

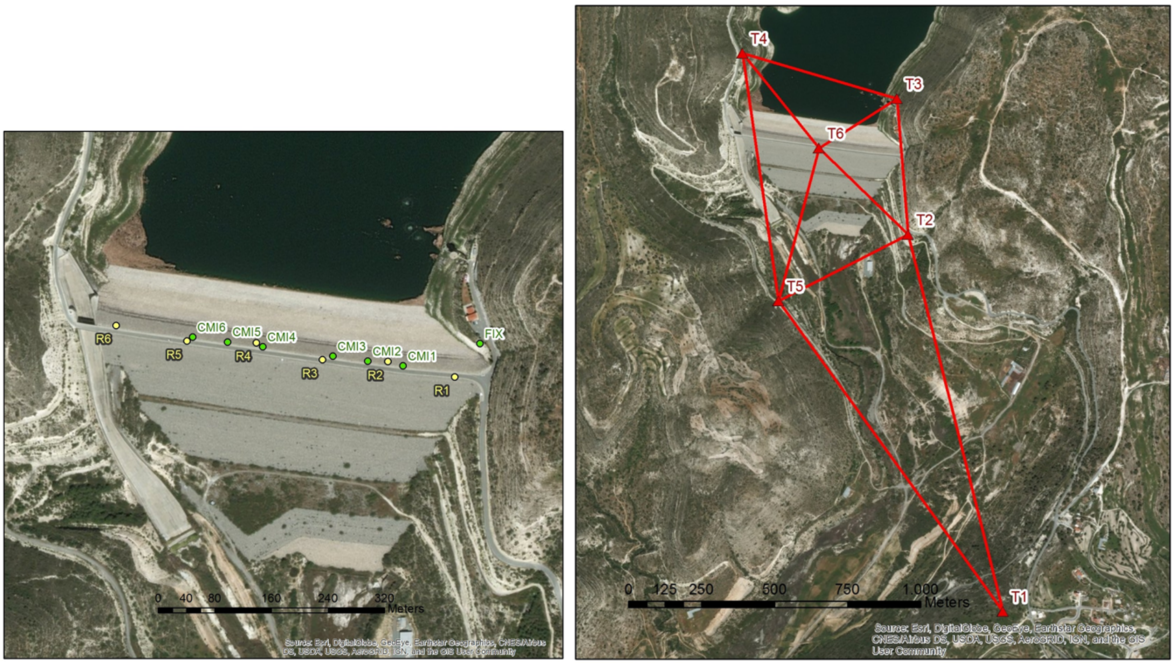
Further two modern geodetic networks were installed in 2006. A vertical (1D) and a three-dimensional (3D) control network. Generally, deformation monitoring is applied by the establishment, the measurement and the adjustment of such network. In this way the obtained results are more reliable as the least square adjustment method provides the variance – covariance matrix namely the uncertainty of the points position determination and their covariances. Thus the evaluation of the displacements is carried out by a robust statistical analysis for each measurement phase. Also as all the check points are referred to a common coordinate system not only the magnitude but also the direction of the displacement can be calculated. Additionally, after an adequate number of measurement series the above procedure can provide clearly the trend of the dam's displacements [49] [50].

The vertical control network was established in 2006 [51]. It consists of 7 control points. Six of them (R1-R6) are bronze benchmarks, located along both sides of the wall on the road at the crest of the dam and in a distance of about 100m between each other, as shown in Figure 2 (a) and (b). The seventh point is the pillar T2 about 1 km far away, which serves as the fixed reference point of the network.

Three periods of measurements (July and December 2006 [52] and June 2012 [53]) were carried out for the determination of the height differences between the points by using either:

(a) The spirit leveling method, using the Leica DNA 010 digital level and the corresponding bar code staffs.
(b) The accurate trigonometric heighting method [54] [55] using the reflectorless total station Leica TCR303, which provides accuracy of ±3" in the angle measurements and ±2mm ±2ppm in the distance measurement.

Both methods provide accuracy of ±1 - 2mm for the height difference determination. The network was adjusted by the least square method. Thus the orthometric heights of the points as well as their uncertainties, which fluctuate from ±1mm to ±2mm, were determined. More details about these geodetic methods can be found by Lambrou and Pantazis [54] [55] [56].

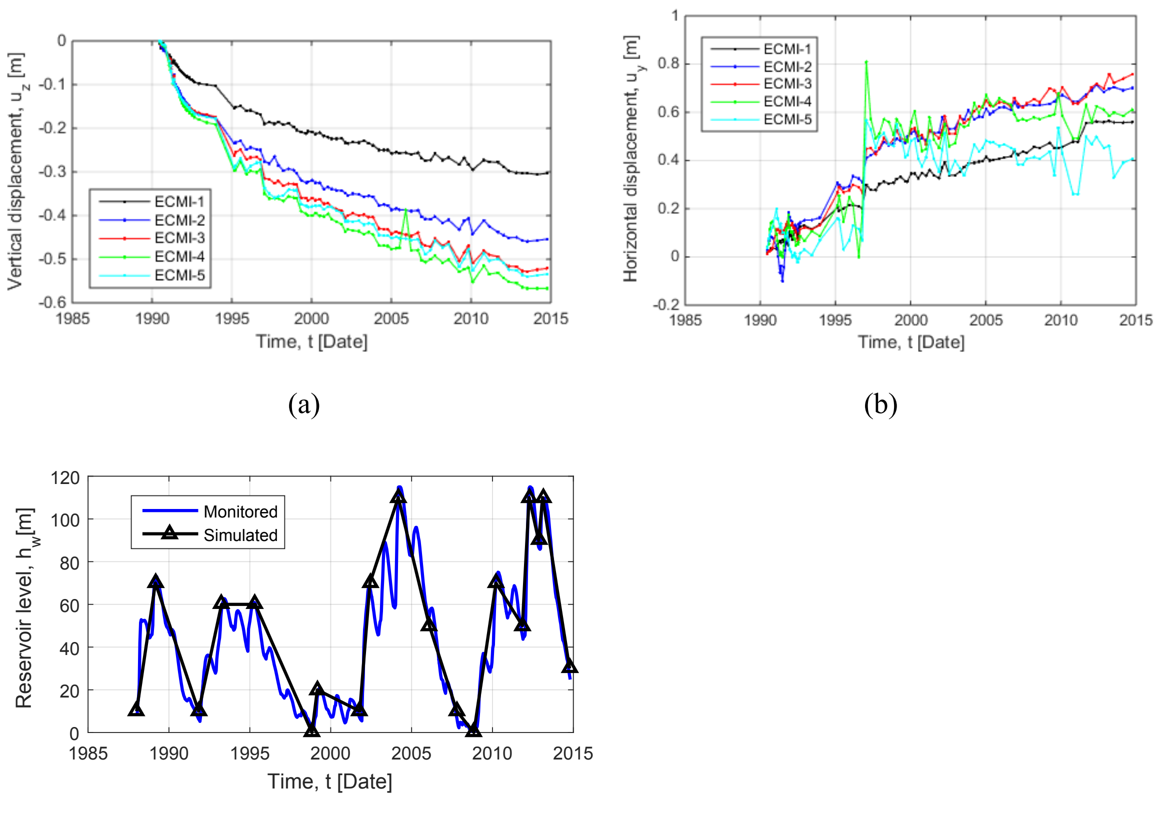


(a) (b)

Figure 2. Monitoring instrumentation: (a) instruments on the dam and (b) global 3D network.

5. Monitoring data evaluation

Figure 3 (a) and (b) show the time-histories of the vertical, u_z, and horizontal, u_y, displacements of several points at the dam crest respectively, as these were obtained from the ECMIs. It should be noted that information from ECMI-6 is not shown because this instrument experienced malfunction and therefore its readings cannot be trusted. The observed trend is that horizontal displacements are mostly positive, i.e. in the downstream direction and the vertical displacements are negative, i.e. in the vertical downward direction (i.e. settlements). The values of both horizontal and vertical displacements tend to increase with time, but the rate of increase seems to get smaller with time and the displacement values tend towards reaching an ultimate value exhibiting a plateau after some time. Figure 3 (c) shows the monitored seasonal changes in the reservoir level which show that the dam experienced both periods of full and empty reservoir. On the same figure, the simulated levels of reservoir in the subsequent FE analysis (Section 7) are also included for comparison. Furthermore, Figure 4 (a) and (b) show snapshots of vertical and horizontal crest displacement profiles respectively. As far as vertical displacements are concerned, it is shown that larger monitored values seem to occur at the middle of the dam's crest and this data show a smooth curved dam crest. These observations regarding the trend of the evolution of dam displacements are generally in good agreement with earlier observations of Kyrou et al. [7], Dounias et al. [8] and Gikas & Sakellariou [37] for similar dams (see Section 2).



(a) (b)

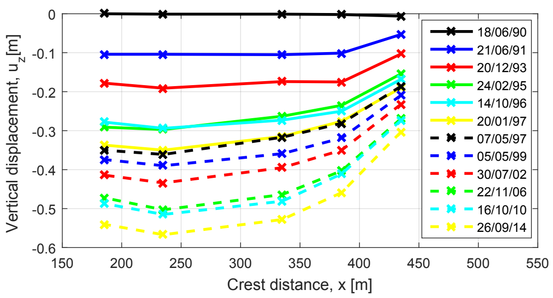
(c)

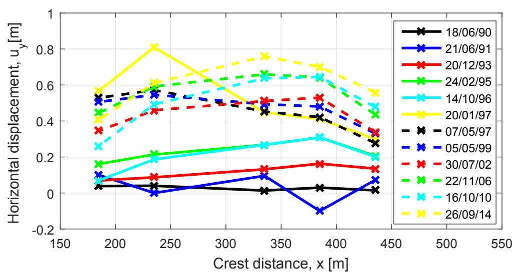
Figure 3. Time-histories of monitoring data: (a) vertical displacements, (b) horizontal displacements and (c) reservoir level fluctuations.

Additionally, two time periods are of particular interest, where a significant step in both the vertical (Figure 4(a)) and horizontal (Figure 4 (b)) displacement time-histories is observed. The two time periods, as judged from the available discrete monitoring data are (a) between December 1993 and February 1995 and (b) October 1996 and January 1997. It should be noted that two major earthquakes were reported to have occurred in Cyprus during these two periods with epicentres very close to the dam location. According to the Cyprus Geological Survey Department [57], an earthquake of Ms 5.7

happened on the 23rd of February 1995 (referred to here as EQ1) in the west of Cyprus, 71km away from Kouris dam. Another earthquake of Ms 6.5 struck on the 9th of October of 1996 (referred to here as EQ2) in the southwest of Cyprus, 78km away from the dam. It is anticipated that those two severe seismic events resulted in dynamic accelerations which may have affected the stability of the dam and introduced further displacements in both the vertical and horizontal directions due to the transient seismic shaking. The vectors of displacement profiles along the crest during those two seismic events are shown graphically in Figure 4 (a) and (b) with black and grey arrows for EQ1 and EQ2 respectively. Specific values of the vertical and horizontal displacements from these two seismic events are shown in Figure 4 (c) and (d) respectively. Larger vertical displacements occur during EQ1, whereas larger horizontal displacements are found for EQ2. The profiles of vertical displacements are similar to such profiles observed at other dams (e.g. La Villita [58] and Infiernillo dams [59] in Mexico). Seismic permanent displacements have been for long recognised [60] [61] and their magnitude depends primarily on the induced seismic accelerations and soil strength properties [62] [63] [64] [65] [66]. The profiles of the inferred seismic displacements in this case are found to be similar to other monitored cases [58] [13]. This dynamic dam response is of high research interest [67] [68] [13] and can of course be of further investigation, but because of its complexity and lack of relevant input data (e.g. seismic records) this is beyond the scope of the current study.

The validity of the monitored data from the ECMIs of the Cyprus WDD is confirmed by the independent geodetic networks. Figure 4 (e) and (f) show the comparison of the vertical and horizontal displacements at the crest of the dam as these were obtained by the three different instrumentation techniques for the period of 2006-2012. The observed differences for the vertical displacements between them are within their accuracy and uncertainties; hence a very good comparison seems to be obtained between the three techniques especially for the displacements in the vertical direction. More specifically, the data processing of the vertical control network showed that the points R1, R2, R3, R4, R5 have significant subsidence, which varies from 18mm - 67mm between the three measurement periods. The maximum settlement was measured at points R3 and R4, which revealed 65 mm and 67mm respectively, and which are situated at around the middle of the dam's crest. Similar measurements have been reported by the 3D network, of which point T6 presents a vertical settlement equal to 67.5mm.





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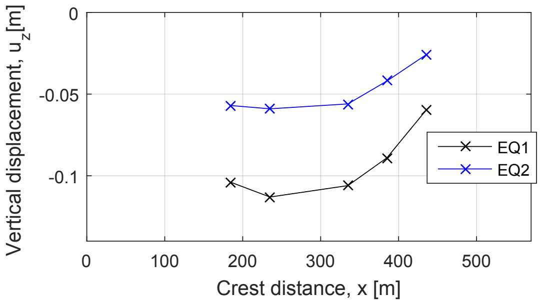

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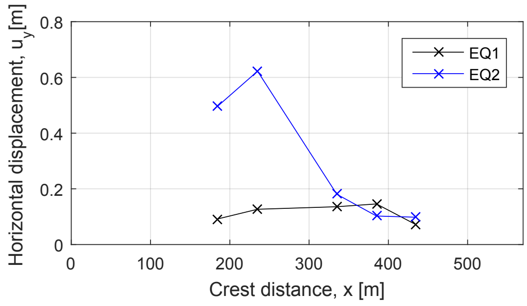

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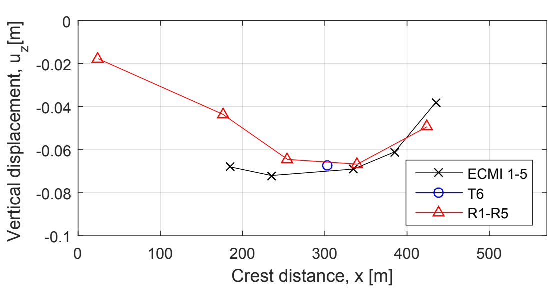

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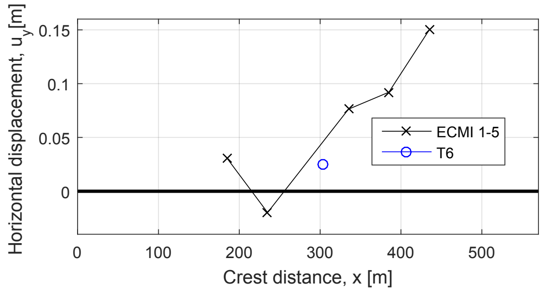


(f)

Figure 4. Profiles of displacements along the crest of the dam: (a) snapshots of vertical displacements (ECMI), (b) snapshots of horizontal displacements (ECMI), (c) Vertical displacements at the crest during the two earthquake events (ECMI), (d) Horizontal displacements at the crest during the two earthquake events (ECMI), (e) Comparison of vertical crest displacements obtained from three different instrumentation techniques for the period of 2006-2012 and (f) Comparison of horizontal crest displacements obtained from two different instrumentation techniques for the period of 2006-2012.

6. Statistical analysis

Further processing of the monitoring data was undertaken using statistical analysis. This was used, in particular, to study any relation between the reservoir level changes and the vertical displacement fluctuations and therefore identify any causative relationship between the two. A direct comparison between the absolute values of the reservoir level (Figure 3 (c)) and vertical displacements (Figure 3 (a)) is not much helpful because of the large consolidation settlements. Therefore, incremental changes from successive readings were examined. Figure 5 (a) plots the incremental changes of reservoir level, ΔR, and incremental vertical displacements, Δu_z, from ECMI-3. It is shown that there is a weak periodicity in the peak values (i.e. large values of upward reservoir level change and downward displacements) of both the reservoir level changes and the crest vertical displacements. It is also shown that the peak values seem to occur at similar times, although some minor differences can be observed. It is also noted that reservoir level changes show clearer peaks than vertical displacements as the latter have less dense (in time) data. This suggests that both quantities exhibit similar periodicity and therefore one could suggest that they have a certain correlation. Moreover, it may be observed that these peak values occur around once a year which may suggest that there are seasonal (climate) variations.

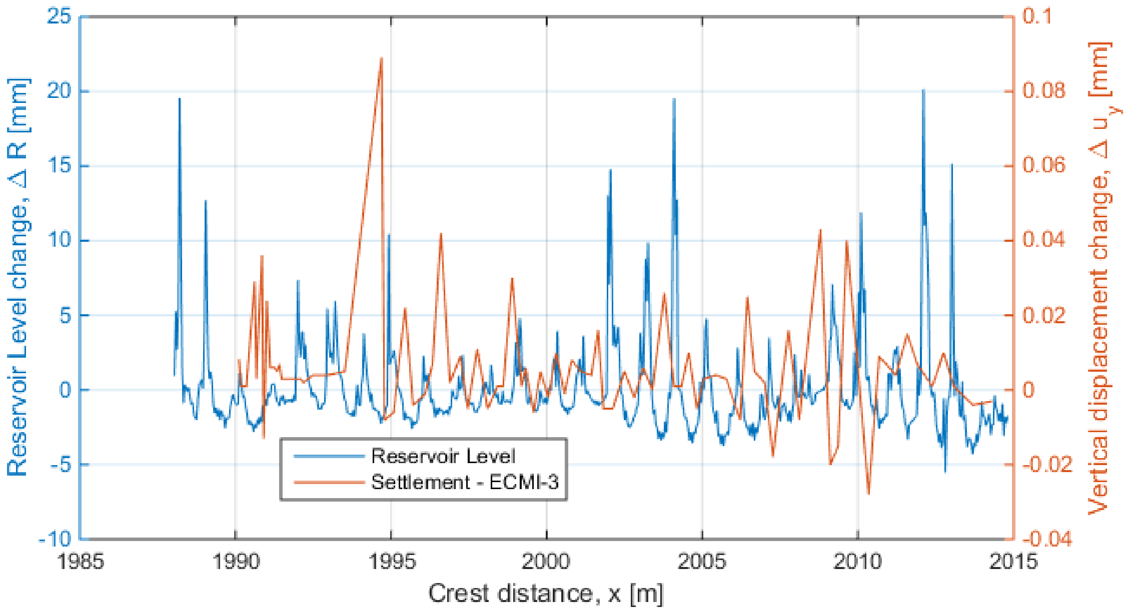
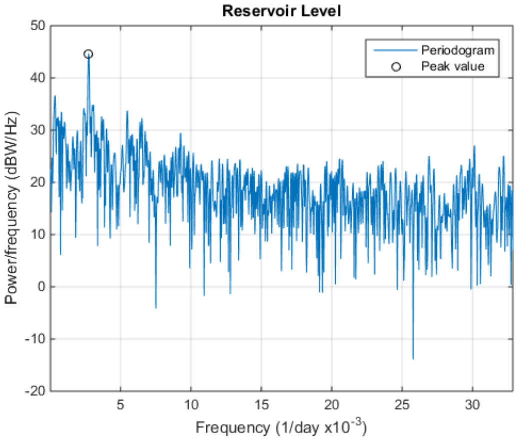


Figure 5. Statistical analysis of monitoring data: differential vertical displacements Vs differential reservoir level.

Since periodicity is observed, a more comprehensive comparison can be made by analysing both time-histories in the frequency domain and identifying their dominant frequencies. Such an exercise is commonly undertaken by Fourier analysis; however, that technique requires equally-spaced sample data, which is not the case here (irregular readings were taken over the study period of 25 years). Interpolation was then performed to generate additional data points to allow a Fourier analysis but this was not successful as it could not identify a single dominant frequency. Another technique was subsequently used instead, which is similar to the popular Fourier analysis, and able to process non-equally-spaced data; that is the Lomb (or Lomb-Scargle) [69] [70] periodogram [26] [19].

Figure 6 (a) and (b) (detail) show the Lomb periodogram for the reservoir level, which shows clearly a single dominant frequency at 2.73 (1/day $\times 10^{-3}$), which means a period of 1000/2.73≈365 days, i.e. a full year. This reinforces the earlier observation that the reservoir level exhibits yearly peaks. Figure 6 (c) and (d) (detail) show the Lomb periodogram for the crest settlements from ECMI-3, which again seems to yield a single dominant frequency at 2.73 (1/day $\times 10^{-3}$), i.e. corresponding to a period of 365 days. This observation is in agreement with earlier suggestions of Pytharouli & Stiros [4] [19] [71] [72] who performed similar statistical analyses and noted a strong correlation between the frequency of reservoir level changes and dam vertical displacements.



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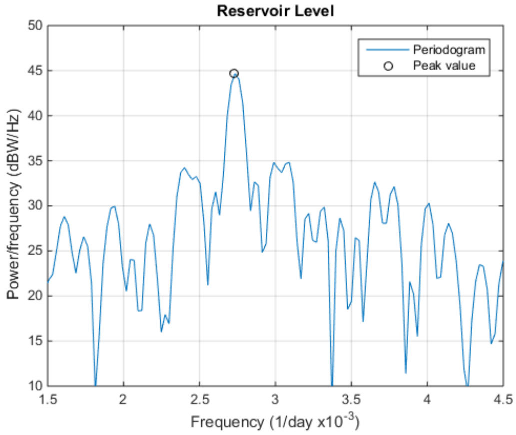

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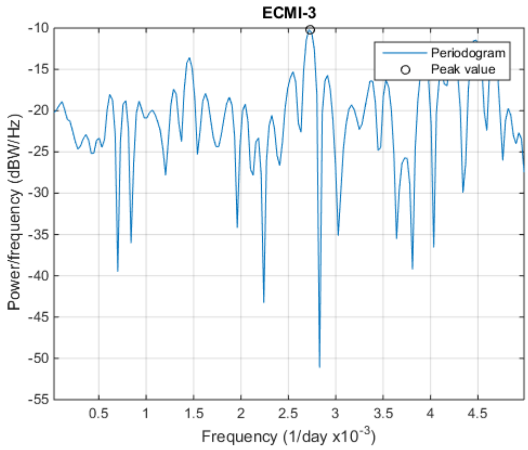

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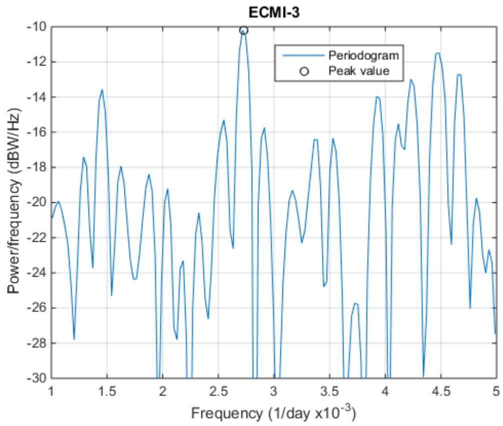


(d)

Figure 6. Lomb periodogram: (a) reservoir level, (b) reservoir level (detail), (c) vertical displacements and (d) vertical displacements (detail).

7. Structural finite-element analysis

Since a relation between reservoir level changes and dam settlements has been identified, now the next step is to assess the relative effects of consolidation and reservoir level changes on the dam displacements. A relevant finite element analysis was subsequently conducted in order to further examine the long-term behaviour of the dam and understand the mechanisms of the observed long-term settlements of the dam crest. In particular, a comparison between the relative effects of soil consolidation and reservoir level fluctuations can be made.

Two-dimensional (2D) plane-strain (adequate due to the high length/height ratio=5.1) non-linear elasto-plastic transient coupled-consolidation finite element (FE) analyses were performed with the FE software ABAQUS/Standard [73]. The FE mesh employed, which includes the embankment and part of the foundation soil is shown in Figure 7 (a) and it consists of 2540 8-noded quadratic iso-parametric elements with 16 displacement and 4 pore water pressure degrees-of-freedom (CPE8RP). The FE mesh geometry includes the central clay core, the upstream and downstream filters, the rockfill shells, the grout curtain and two layers of foundation material. All the materials are assumed to behave in an elasto-plastic consolidating manner. The elasto-plastic constitutive models adopted are the Mohr-Coulomb (MC) and the Modified Cam-Clay (MCC) (which is a critical state type model). The MCC is used for the cohesive central clay core, whereas the MC is used for the other granular soils such as the rockfill shells. The elasto-plastic models are coupled with a hyperbolic small-strain stiffness that dictates the non-linear stiffness degradation of shear modulus, G, with the shear strain, γ, [74] given by Equation 1 (implemented in ABAQUS using a user-defined subroutine) and combined with a constant Poisson's ratio, ν.

$$\tau = \frac{G_{max} \cdot \gamma}{1 + \lambda \cdot \gamma}$$

Equation 1

Where, G is the soil shear modulus (=E/2(1+ν)), E is the soil Young's modulus, G_{max} is the maximum value of G for zero shear strain, γ is the shear strain and λ is a model calibration parameter. The latter parameter dictates shear stiffness degradation with strain. However, due to lack of relevant experimental data (the design of the dam was performed in the 1970-80s), the hyperbolic model was calibrated against appropriate empirical curves from the literature, using Vucetic & Dobry [75] were used for the cohesive clay core and the foundation materials (including the grout curtain) and the Rollins et al. [76] curves for the granular filters and the rockfill shells. Similar and even simpler constitutive modelling strategies have been followed by other researchers for modelling dam deformations [37] [10] and they have reproduced well the observed field data. The adopted material properties and model parameters are summarised in Table 1. Some material properties related to the strength (c, φ) and permeability (k) were obtained from the WDD [48], whereas typical representative values of soil stiffness parameters (Young's Modulus) from the literature [46] [36] [37] [77] [78] [38] [79] [41] [34] [80] were adopted.

Table 1. Material properties used in the FE analysis.

Material (Mohr Coulomb model)	Unit Weight, γ' [kN/m³]	Young Modulus, E_{max} [kPa]	Poisson's ratio, ν []	Friction Angle, φ [°]	Dilation Angle, ψ [°]	Cohesion, c [kPa]	Permeability coefficient, k [m/s]	Model parameter, λ []
Sand Filters	20	300 000	0.3	45	15	370	$1x10^{-3}$	2500
Rockfill Shells	22	400 000	0.3	45	15	370	$1x10^{-3}$	2500
Grout Curtain	20	700 000	0.3	45	15	350	$1x10^{-11}$	900
Upper foundation	19	400 000	0.3	45	15	350	$1x10^{-8}$	900
Lower foundation	19	700 000	0.3	45	15	350	$1x10^{-11}$	900
Material (Modified Cam-clay model)	**Unit Weight, γ' [kN/m³]**	**Young Modulus, E_{max} [kPa]**	**Gradient of swelling line, κ []**	**Gradient of compression line, λ []**	**Specific volume, v_1[]**	**Critical state ratio, M []**	**Permeability coefficient, k [m/s]**	**Model parameter, λ []**
Clay core	20	300 000	0.05	0.6	2.5	1.2	$1x10^{-9}$	900

The analysis was executed in several stages in an attempt to simulate the true stress history of the dam. Firstly, initial stresses were initiated with level ground (i.e. foundation) and the dam embankment (clay core, filters and shells) was constructed in five successive layers over a period of 12 months. Then, the reservoir was impounded and its fluctuations followed the monitored time-history of the level of the reservoir, as shown in Figure 3 (c), over 19 steps. Each of these latter steps was modelled over a number of increments in order to achieve numerical stability of the water seepage and consolidation problem [81]. The boundary conditions employed were full fixity along the bottom foundation boundary and horizontal fixity along the lateral foundation boundaries. Zero water flow (i.e. impermeable boundary) was specified along the bottom of the FE mesh, which is assumed to be the interface of the foundation soil and stiff bedrock; whereas zero pore water pressure was specified along the top and downstream sides of the embankment throughout the whole analysis [82]. The upstream embankment slope had prescribed values of pore water pressure according to the hydrostatic variation of the water within the reservoir at each reservoir level. For equilibrium, an equal-value hydrostatic external boundary stress was applied on the upstream boundary to simulate the load from the upstream reservoir.

Figure 7 (b) and (c) show the contours of resultant displacements and the (magnified) deformed shape of the dam respectively, when the reservoir is at its highest level (1/3/2004). It is shown that the dam deforms both in the vertical direction and the horizontal downstream direction as a result of construction, consolidation and reservoir impoundment. Moreover, Figure 7 (d) shows the pore water pressure distribution within the dam at the same time. The latter figure exhibits the saturation of the upstream rockfill and the drop of hydraulic head within the dam due to the seepage through the low-permeability clay core and grout curtain.

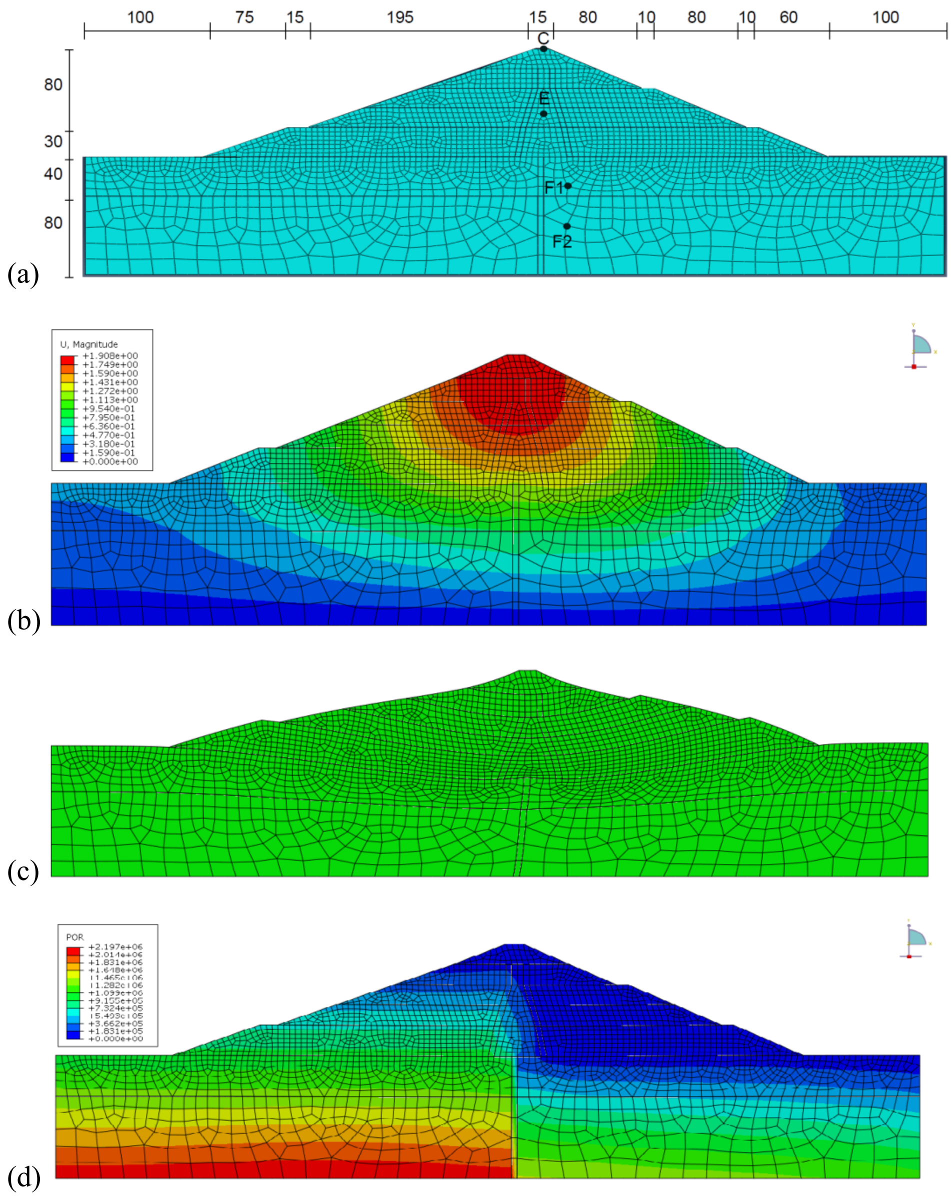
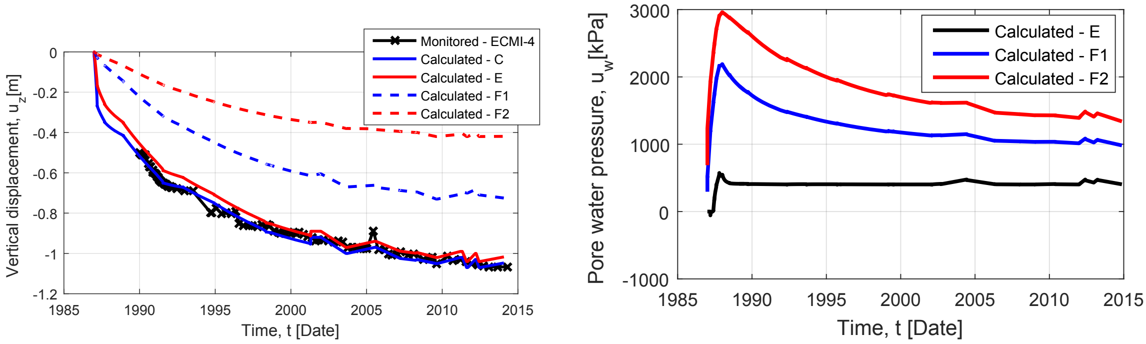


Figure 7. Finite-element analysis of the dam: (a) FE mesh (dimensions in metres), (b) dam displacements, (c) deformed shape and (d) pore water pressures.

The time-history of vertical displacements at the crest of the dam (Point C in Figure 7 (a)) is shown in Figure 8 (a). The difference in the time of the monitored and calculated data is because the available settlement monitoring data started from 1990, whereas the reservoir level data started from 1987. It was thus assumed that the operation of the dam started from 1987, after the construction of the embankment and therefore the initial monitored settlement data is anchored at the calculated settlements at that time (1990) for direct comparison. It is shown that there is generally a good match between the field-monitored (from ECMI-4) and numerically-predicted displacement values. It is also shown that there is a general trend of downward movements due to consolidation of the dam materials and that the reservoir fluctuations do not seem to have a severe impact on the dam settlements. On the same figure, the vertical displacements in the embankment and the two foundation layers (Points E, F1 and F2 in Figure 7(a)) are included too. It is shown that the consolidation settlement values of the foundation layers are considerably large and comparable to those of the dam crest, which would suggest that the consolidation settlements within both the dam embankment and the foundation contribute significantly to the overall crest settlements. This is due to the seepage of the excess pore water pressure generated in the embankment and the low-permeability foundation because of the construction of the large 110m embankment. This also suggests that the effects of the reservoir level fluctuations are not very significant and the dominant mechanism inducing long-term vertical crest displacements is indeed consolidation of the soil materials of both the dam foundation and the embankment. The above-mentioned observations compare quite well with similar observations reported in the literature [4] [7] [8], which do not reveal an important impact of the reservoir level fluctuations on the long-term dam settlements.

Additionally, Figure 8 (b) shows the development of pore water pressures, u_w, in the embankment core and foundation layers (Points E, F1 and F2 in Figure 7 (a)). As it may be observed, large values of pore water pressure develop due to the construction of the dam embankment, which subsequently dissipate with time due to consolidation and hence induce vertical settlements. This is in agreement with the observations of Kyrou et al. [7], who measured high values of pore water pressure development due to embankment construction at the base of the (similar but smaller) Lefkara dam in Cyprus, which subsequently dissipated with time. The magnitude of the peak excess pore pressures in the foundation (points F1 and F2; $u_w \approx 2000\text{kPa} \approx \gamma_w \cdot h \approx 9.81\ \text{kN/m}^3 \cdot 110\text{m}$) suggests that a great portion of the additional load due to the construction of the embankment is initially carried by the water. Moreover, it is shown that larger values of water pressures develop in the foundation layers (≈ 2000kPa) than in the embankment (≈ 500kPa), because of the height of the overlying soil (and hence, higher applied stresses) and possibly due to the small values of foundation permeability [83] [84] which exhibit undrained behaviour.



(a) (b)

Figure 8. Computed time-histories of: (a) vertical displacements and (b) pore water pressures.

8. Conclusions

The long-term settlement behaviour of earth dams and its dependence on reservoir level changes and consolidation was investigated. This is performed by studying the monitored response of Kouris embankment dam in Cyprus. The examination includes a critical assessment and statistical analysis of the monitored data of dam displacements from three independent sets of instrumentation over a period of more than 25 years after its construction and full operation. A subsequent relevant nonlinear, coupled hydro-mechanical finite element analysis, simulating the stress history of the dam, aimed to assess the relative effects of reservoir level fluctuations and soil consolidation on the settlements of the dam.

The findings of this study may be summarised as follows:

a) The settlements of the dam crest increase rapidly during the first 5-7 years after the construction of the dam's embankment and the rate of increase tends to reduce with time. This is found to be in agreement with earlier observations reported in the literature, and it is attributed to soil consolidation.

b) For this dam case, two evident (unexpected) "step-changes" in the monitored displacement time-histories are observed and attributed to two strong seismic events, which occurred very close to the dam and they are believed to have contributed to some permanent settlements or even perhaps localised dam slope failures.

c) It was shown that consolidation of the dam embankment and foundation materials following the construction of the embankment seems to be the main reason for the long-term dam settlements. In addition, large values of excess pore water pressure develop in the dam and the foundation during and after construction due to the additional load from the embankment weight.

d) There seems to be a correlation between the vertical dam displacements and reservoir level fluctuations, since their dominant frequencies are identical, which suggests seasonal variations of dam settlements.

e) Total horizontal displacements experienced by the dam were found to be small compared to the vertical ones. It was found that the contribution of the two seismic events to the horizontal displacements was larger than that of consolidation.

f) The contribution of the fluctuations of the reservoir level on both the vertical and horizontal displacements of the dam was found to be small compared to that of soil consolidation and earthquake accelerations.

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References

[1] R. Clements, "Post-construction deformation of rockfill dams," *Journal of Geotechnical Engineering, ASCE*, vol. 110, no. 7, pp. 821-840, 1984.

[2] O. Dascal, "Post-construction deformation of rockfill dams," *Journal of Geotechnical Engineering*, vol. 113, no. 1, pp. 46-59, 1987.

[3] G. Hunter and R. Fell, "The deformation behaviour of embankment dams. R-416.," The University of New South Wales, Sydney, 2003.

[4] S. Pytharouli and S. Stiros, "Ladon dam (Greece) deformation and reservoir level fluctuations: evidence for a causative relationship from the spectral analysis of a geodetic monitoring record," *Engineering Structures*, vol. 27, no. 3, pp. 361-370, 2005.

[5] L. Pagano, S. Sica and A. Desideri, "Representativeness of measurements in the interpretation of earth dam behaviour," *Canadian Geotechnical Journal*, vol. 43, no. 1, pp. 87-99, 2006.

[6] P. Tedd, J. A. Charles, I. R. Holton and A. C. Robertshaw, "The effect of reservoir drawdown and long-term consolidation on the deformation of old embankment dams," *Geotechnique*, vol. 47, no. 1, pp. 33-48, 1997.

[7] K. Kyrou, A. D. M. Penman and C. C. Artemis, "The first 25 years of the Lefkara dam, Cyprus," *Proc. ICE Geotechnical Engineering*, vol. 158, no. GE2, pp. 113-122, 2005.

[8] G. T. Dounias, K. Anastasopoulos and A. Kountouris, "Long-term behaviour of embankment dams: seven Greek dams.," *Proc. ICE Geotechnical Engineering*, vol. 165, no. GE3, pp. 157-177, 2012.

[9] P. Vaughan, "Assumption, prediction and reality in geotechnical engineering," *Geotechnique*, vol. 44, no. 4, pp. 573-609, 1994.

[10] L. Wen, J. Chai, X. Wang, Z. Xu, Y. Qin and Y. Li, "Behaviour of concrete-face rockfill dam on sand and gravel foundation," *Proceedings of the ICE - Geotechnical Engineering*, 2015.

[11] L. Pelecanos, S. Kontoe and L. Zdravkovic, "Dam–reservoir interaction effects on the elastic dynamic response of concrete and earth dams," *Soil Dynamics and Earthquake Engineering*, vol. 82, pp. 138-141, 2016.

[12] J. F. Hall and A. Chopra, "Hydrodynamic effects in earthquake response of embankment dams," *Journal of the Geotechnical Engineering Division*, vol. 108, no. 4, pp. 591-597, 1982.

[13] L. Pelecanos, S. Kontoe and L. Zdravkovic, "A case study on the seismic performance of earth

dams," *Geotechnique.*, vol. 65, no. 11, pp. 932-935, 2015.

[14] G. Gazetas, "Seismic response of earth dams: some recent developments," *Soil Dynamics and Earthquake Engineering*, vol. 6, no. 1, pp. 3-47, 1987.

[15] L. Pelecanos, S. Kontoe and L. Zdravkovic, "Numerical modelling of hydrodynamic pressures on dams," *Computers and Geotechnics*, vol. 53, pp. 68-82, 2013.

[16] J. A. Charles, "The engineering behaviour of fill materials: the use, misuse and disuse of case histories," *Geotechnique*, vol. 58, no. 7, pp. 541-570, 2008.

[17] L. Barden, A. McGown and K. Collins, "The collapse mechanism in partly saturated soil," *Engineering Geology*, vol. 7, no. 1, pp. 49-60, 1973.

[18] E. Alonso, A. Gens and A. Josa, "A constitutive model for partially saturated soils," *Geotechnique*, vol. 40, no. 3, pp. 405-430, 1990.

[19] S. Pytharouli and S. Stiros, "Reservoir level fluctuations and deformation of Ladon dam," *The International Jounral on Hydropower and Dams*, vol. 11, no. 3, pp. 82-84, 2004.

[20] R. Mahinroosta, A. Alizadeh and B. Gatmiri, "Simulation of collapse settlement of first filling in a high rockfill dam," *Engineering Geology*, vol. 187, pp. 32-44, 2015.

[21] S. Pytharouli, V. Kontogianni, P. Psimoulis, A. Nickitopoulou, S. Stiros, C. Skourtis, F. Stremmenos and A. Kountouris, "Geodetic monitoring of earthfill and concrete dams in Greece," vol. 14, no. 2, 2007.

[22] C. Skourtis, F. Stremmenos, S. Pytharouli, V. Kontogianni, A. Nickitopoulou, P. Psimoulis and S. Stiros, "Long-term Geodetic Monitoring of Two Dams in Western Greece," Nottingham, UK, 2004.

[23] P. Michalis and S. R. S. Pytharouli, "Long-term Deformation Patterns of Earth-fill Dams based on Geodetic Monitoring Data: the Pournari I Dam Case Study," Vienna, 2016.

[24] P. J. Venda Oliveira, L. J. Lemos and P. A. L. F. Coelhon, "Behavior of an Atypical Embankment on Soft Soil: Field Observations and Numerical Simulation," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 136, no. 1, pp. 35-47, 2010.

[25] B. Z. D. L. H. Xu, "Three-dimensional simulation of the construction process of the Zipingpu concrete face rockfill dam based on a generalized plsicity model," *Computers and Geotechnics*, vol. 43, pp. 143-154, 2012.

[26] S. Pytharouli and S. Stiros, "Ladon dam (Greece) deformation and reservoir level fluctuations: evidence for a causative relationship from the spectral analysis of a geodetic monitoric record," *Engineering Structures*, vol. 27, no. 3, pp. 361-370, 2005.

[27] A. D. M. Penman, J. B. Burland and J. A. Charles, "Observed and predicted deformations in a large embankment during construction," *Proceedings of the ICE - Geotechnical Engineering*, vol. 49,

no. 1, pp. 1-21, 1971.

[28] H. G. Poulos, J. R. Booker and G. J. Ring, “Simplified calculation of embankment deformations,” *Soils and Foundations*, vol. 12, no. 4, pp. 1-17, 1972.

[29] G. T. Dounias, P. D. M. and P. R. Vaughan, “Analysis of progressive failure and cracking in old British dams,” *Geotechnique*, vol. 46, pp. 621-640, 1996.

[30] R. A. Day, D. W. Hight and D. M. Potts, “Finite element analysis of construction stability of Thika Dam,” *Computers and Geotechnics*, vol. 23, pp. 205-219, 1998.

[31] L. Pagano, A. Desideri and F. Vinale, “Interpreting settlement profiles of earth dams,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 124, no. 10, pp. 923-932, 1998.

[32] D. M. Potts and L. Zdravkovic, Finite element analysis in geotechnical engineering: application, London: Thomas Telford, 2001.

[33] N. Kovacevic, K. G. Higgins, D. M. Potts and P. R. Vaughan, “Undrained behaviour of brecciated Upper Lias Clay at Empingham Dam,” *Geotechnique*, vol. 57, no. 2, pp. 181-195, 2007.

[34] N. Kovacevic, D. W. Hight, D. M. Potts and I. C. Carter, “Finite element analysis of the failure and reconstruction of the main dam embankment at Abberton Reservoir, Essex, UK,” *Geotechnique*, vol. 63, pp. 753-767, 2013.

[35] D. M. Potts, “Using numerical analysis in geotechnical engineering,” in *1st Leonardo Zeevaert Lecture*, Mexico, 2012.

[36] P. Lollino, F. Cotecchia, L. Zdravkovic and D. M. Potts, “Numerical analysis and monitoring of Pappadai dam,” *Canadian Geotechnical Journal*, vol. 42, pp. 1631-1643, 2005.

[37] V. Gikas and M. Sakellariou, “Settlement analysis of the Mornos earth dam (Greece): Evidence from numerical modelling and geodetic monitoring,” *Engineering Structures*, vol. 30, pp. 3074-3081, 2008.

[38] M. Karstunen and Z.-Y. Yin, “Modelling time-dependent behaviour of Murro test embankment.,” *Geotechnique*, vol. 60, no. 10, pp. 735-749, 2010.

[39] W. Zhou, J. Hua, X. Chang and C. Zhou, “Settlement analysis of the Shuibuya concrete-face rockfill dam,” *Computers and Geotechnics*, vol. 38, pp. 269-280, 2011.

[40] J. M. Duncan and C. M. Chang, “1970,” *J. Soil Mechanics and Foundations Division, ASCE*, vol. 96, no. SM5, pp. 1629-1653, 1970.

[41] L. Chai, Y. Ihaya, T. Hino and J. Carter, “Finite element simulation of an embankment on soft clay,” *Computers and Geotechnics*, vol. 48, pp. 117-126, 2013.

[42] T. Bayrak, “Modelling the relationship between water level and vertical displacements on the Yamula Dam,” *Nat. Hazards Earth. Sci.*, vol. 8, pp. 289-297, 2007.

[43] E. Nobari and J. Duncan, "Effect of reservoir filling on stresses and movements in earth and rockfill dams.," TE.72-1. Office of Research Services. University of California, Berkeley, 1972.

[44] D. J. Naylor, E. Maranha das Neves, D. J. Mattar and A. A. Veiga Pinto, "A back-analysis of Beliche Dam," *Geotechnique*, vol. 47, no. 2, pp. 221-233, 1997.

[45] D. J. Naylor, E. Maranha das Neves, D. J. Mattar and A. A. Veiga Pinto, "Prediction of construction performance of Beliche Dam," *Geotechnique*, vol. 3, pp. 359-376, 1986.

[46] E. E. Alonso, S. Olivella and N. M. Pinyol, "A review of Beliche Dam," *Geotechnique*, vol. 55, no. 4, pp. 267-285, 2005.

[47] Cyprus Water Development Department, "Dams of Cyprus," Cyprus Ministry of Agriculture, Rural Development and Environment, Nicosia, Cyprus, 2009.

[48] Cyprus Water Development Department, *Personal Communication*, 2014.

[49] A. Chrzanowski, Y. Chen and J. Secord, "On the strain analysis of tectonic movements using fault crossing geodetic surveys," *Tectonophysics*, vol. 97, pp. 297-315, 1983.

[50] J. M. Rueger, "Overview of Geodetic deformation measurements of dams," in *ANCOLD conference*, 2006.

[51] C. Constantinou, "Monitoring the defromation of Kouris dam in Cyprus by using GNSS measurements. Diploma thesis (in Greek)," Cyprus University of Technology, Cyprus, 2013.

[52] C. Temenos, "Monitoring the deformations of Kouris dams in Cyprus. Diploma thesis (in Greek)," National Technical University of Athens, Greece, 2007.

[53] A. Pantelidou, "Monitoring the vertical displacements of Kouris dam in Cyprus by usng modern accurate methods. Diploma thesis (in Greek)," Cyprus University of Technology, Cyprus, 2013.

[54] E. Lambrou, "Accurate definition of altitude difference with geodetic total stations (in Greek)," *Technika Chronika*, vol. 10, no. 1-2, pp. 37-46, 2007.

[55] E. Lambrou and G. Pantazis, "A convenient method for accurate height differences determination," Sofia, 2007.

[56] E. Lambrou and G. Pantazis, Applied Goedesy, Athens: Ziti, 2010.

[57] Cyprus Geological Survey Department, "Cyprus Ministry of Agriculture, Rural Development and Environment - Geological Survey Department - Historic Earthquakes," 2014. [Online]. Available: http://www.moa.gov.cy/moa/gsd/gsd.nsf/dmlHistEarthquakes_en/dmlHistEarthquakes_en?OpenDocument. [Accessed 1 October 2014].

[58] A. W. Elgamal, R. F. Scott, M. F. Succarieh and L. Yan, "La Villita dam response during five earthquakes including permanent deformations," *Journal of Geotechnical Engineering*, vol. 116, no. 10, pp. 1443-1462, 1990.

[59] A. W. Elgamal and R. V. Gunturi, “Dynamic behaviour and seismic response of El Infiernillo dam,” *Earthquake Engineering & Structural Dynamics*, vol. 22, no. 8, pp. 665-684, 1993.

[60] N. M. Newmark, “Effects of earthquakes on dams and embankments,” *Geotechnique*, vol. 15, no. 2, pp. 139-160, 1965.

[61] H. B. Seed, “Consideration in the earthquake-resistant design of earth and rockfill dams,” *Geotechnique*, vol. 29, no. 3, pp. 215-263, 1979.

[62] L. Pelecanos, S. Kontoe and L. Zdravkovic, “Static and dynamic analysis of La Villita Dam in Mexico,” in *Proceedings of the 2nd International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, Taormina, Italy, 2012.

[63] N. N. Ambraseys and J. M. Menu, “Earthquake-induced ground displacements,” *Earthquake Engineering & Structural Dynamics*, vol. 16, no. 7, pp. 985-1006, 1988.

[64] S. K. Sarma, “Stability analysis of embankments and slopes,” *Journal of the Geotechnical Division, ASCE*, vol. 105, no. GT5, pp. 1511-1524, 1979.

[65] G. Gazetas and N. Uddin, “Permanent deformation on pre-existing sliding surfaces in dams,” *Journal of Geotechnical Engineering*, vol. 120, no. 11, pp. 2041-2061, 1994.

[66] L. Pelecanos, “Seismic response and analysis of earth dams,” PhD thesis, Imperial College London, London, United Kingdom, 2013.

[67] G. Elia, A. Amorosi, A. H. C. Chan and M. J. Kavvadas, “Fully coupled dynamic analysis of an earth dam,” *Geotechnique*, vol. 61, no. 7, pp. 549-563, 2010.

[68] A. G. Papadimitriou, G. D. Bouckovalas and K. I. Andrianopoulos, “Methodology for estimating seismic coefficients for performance-based design of earth dams and tall embankments.,” *Soil Dynamics and Earthquake Engineering*, vol. 56, pp. 57-73, 2014.

[69] N. R. Lomb, “Least-squares frequency analysis of unequally spaced data,” *Astrophysics and Space Science*, vol. 39, pp. 447-462, 1976.

[70] J. D. Scargle, “Studies in astronomical time series analysis. II. Statistical aspects of spectral analysis of unevenly spaced data,” *Astrophysical Journal*, vol. 263, pp. 235-2853, 1982.

[71] S. Pytharouli and S. Stiros, “Dam crest settlement, reservoir level fluctuations and rainfall: evidence for a causative relationship for the Kremasta dam, Greece,” Lisbon, 2008.

[72] S. Pytharouli and S. Stiros, “Investigation of the parameters controlling the crest settlement of a major earthfill dam based on the threshold correlation analysis,” vol. 3, no. 1, 2009.

[73] I. ABAQUS, “ABAQUS V. 6.12 User's Manual,” Dassault Systemes, Vélizy-Villacoublay Cedex - France, 2014.

[74] J. H. Prevost, “Mathematical modelling of monotonic and cyclic undrained clay behaviour,”

International Journal for Numerical and Analytical Methods in Geomechanics, vol. 1, no. 2, pp. 195-216, 1977.

[75] M. Vucetic and R. Dobry, "Effect of soil plasticity on cyclic soil response," *J. Geotechnical Engineering*, *ASCE*, vol. 117, no. 1, pp. 87-107, 1991.

[76] K. Rollins, M. Evans, N. Diehl and W. Daily, "Shear modulus and damping relations for gravel," *J. Geotechnical and Geoenvironmental Engineering*, *ASCE*, vol. 124, no. 5, pp. 396-405, 1998.

[77] S. Sica, L. Pagano and A. Modaressi, "Influence of past loading history on the seismic response of earth dams," *Computers and Geotechnics*, vol. 35, no. 1, pp. 61-85, 2008.

[78] S. Rampello, E. Cascone and N. Grosso, "Evaluation of the seismic response of a homogeneous earth dam," *Soil Dynamics and Earthquake Engineering*, vol. 29, no. 5, pp. 782-798, 2009.

[79] P. Dakoulas, "Nonlinear seismic response of tall concrete-faced rockfill dams in narrow canyons," *Soil Dynamics and Earthquake Engineering*, vol. 34, no. 1, pp. 11-24, 2012.

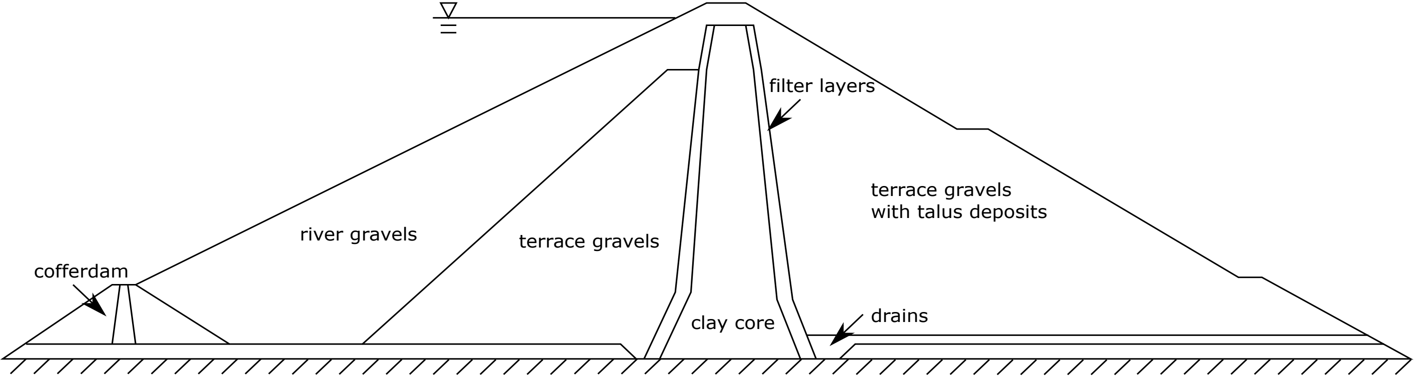
[80] M. Fragiadakis, D. Loukidis and P. Papanastasiou, "Seismic capacity assessment of Kouris earth dam," in *Second European Conference on Earthquake Engineering*, Istanbul, 2014.

[81] P. A. Vermeer and A. Verruijt, "An Accuracy Condition for Consolidation by Finite Elements," *International Journal for Numerical and Analytical Methods in Geomechanics*, vol. 5, pp. 1-14, 1981.

[82] D. M. Potts and L. Zdravkovic, Finite element analysis in geotechnical engineering: theory, London: Thomas Telford, 1999.

[83] K. Terzaghi, Erbaumechanik auf Bodenphysikalischer Grundlage, Leipzig und Wein: Franz Deuticke, 1925.

[84] R. E. Gibson, G. L. England and M. J. L. Hussey, "The theory of one-dimensional consolidation of saturated clays," *Geotechnique*, vol. 17, no. 3, pp. 261-273, 1967.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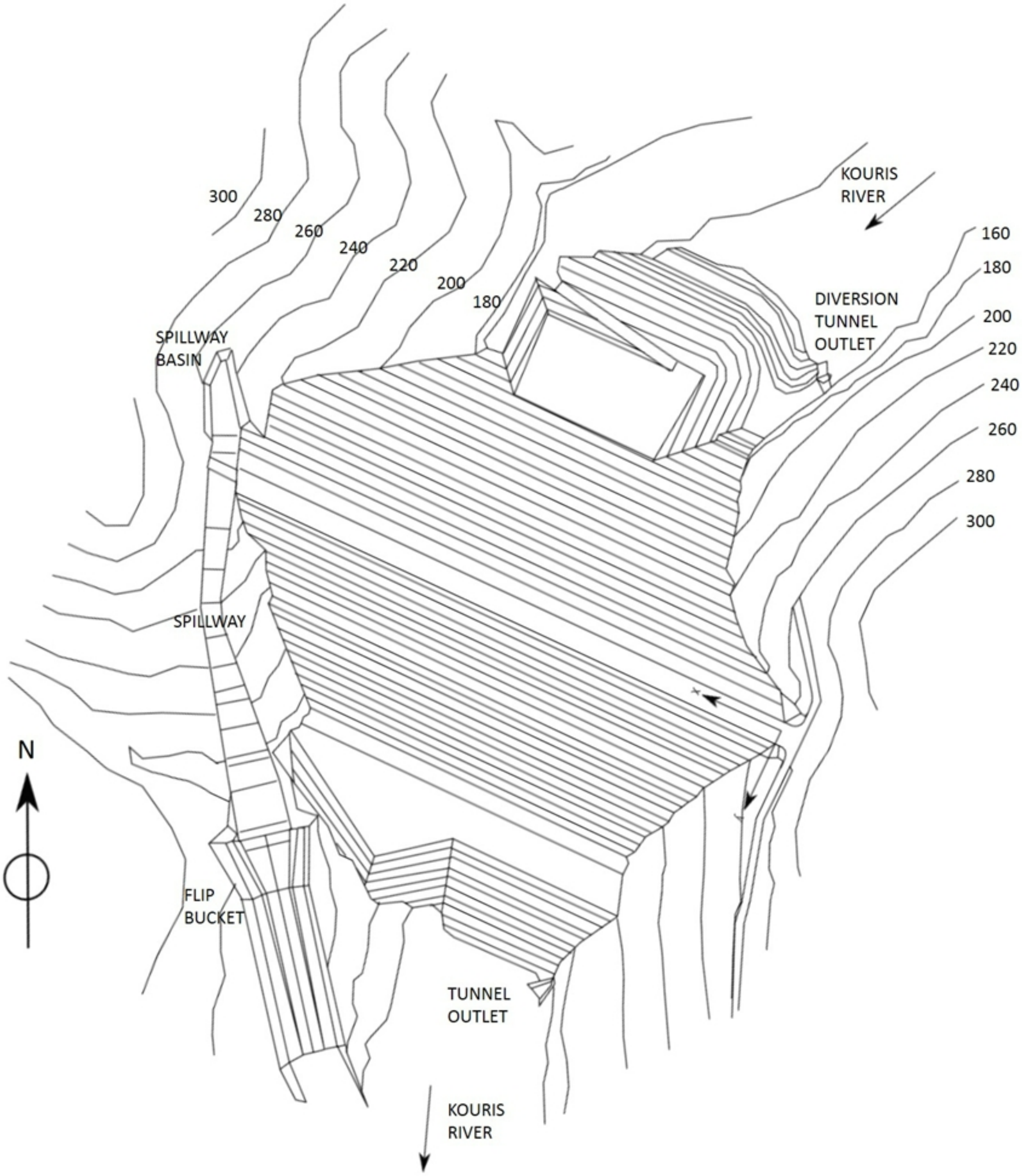
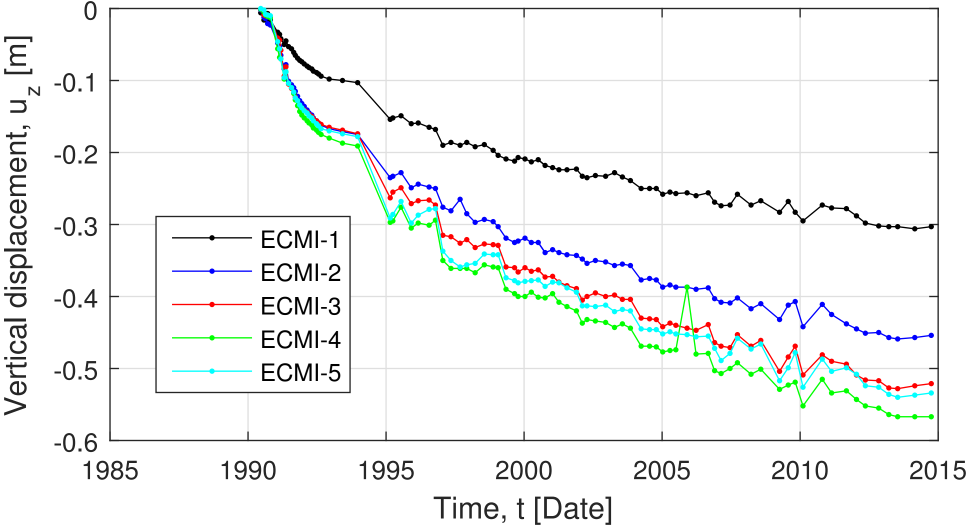
terrace gravels
with talus deposits
river gravels
terrace gravels
cofferdam
drains
clay core



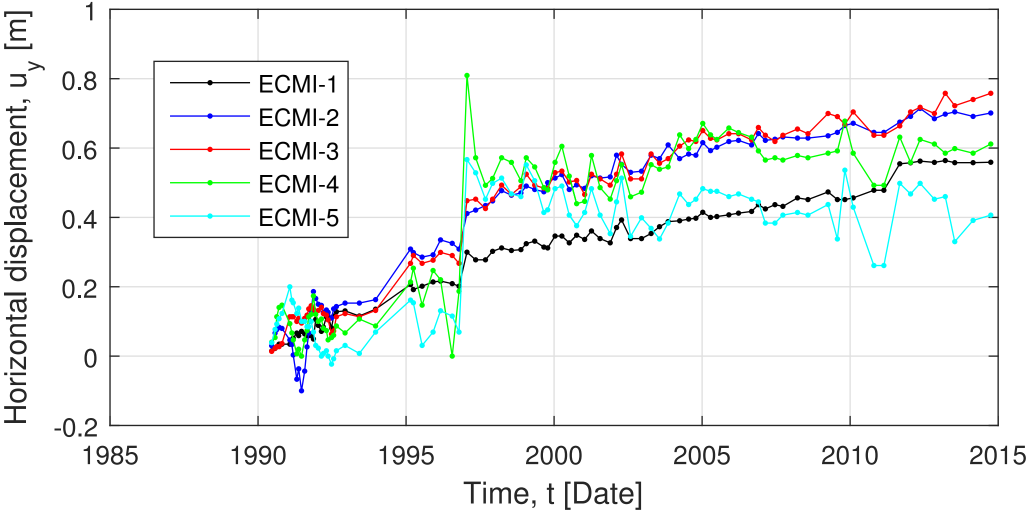
KOURIS RIVER
300
280
260
240
220
200
180
160
180
200
220
240
260
280
300
DIVERSION TUNNEL OUTLET
SPILLWAY BASIN
SPILLWAY
N
FLIP BUCKET
TUNNEL OUTLET
KOURIS RIVER


CMI6
CMI5
CMI4
CMI3
CMI2
CMI1
FIX
R6
R5
R4
R3
R2
R1
0
40
80
160
240
320
Meters
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

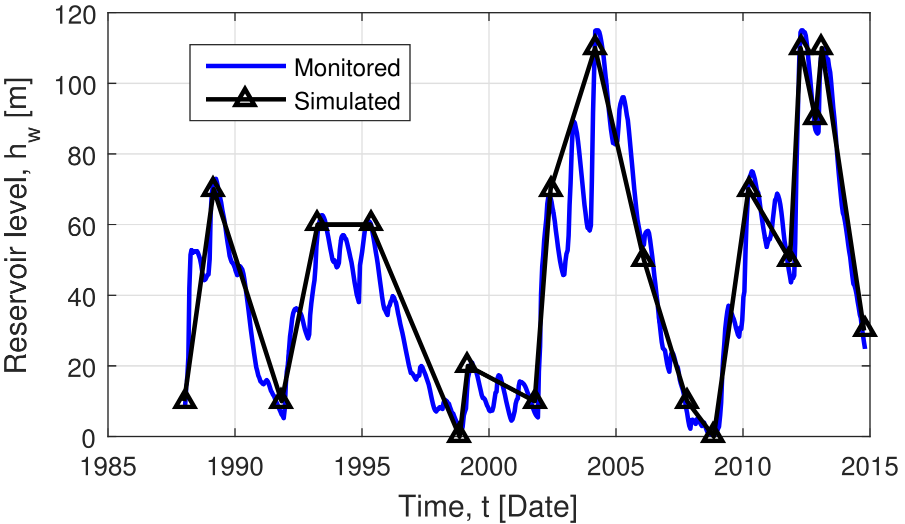

T4
T3
T6
T2
T5
T1
0 125 250 500 750 1.000
Meters
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community



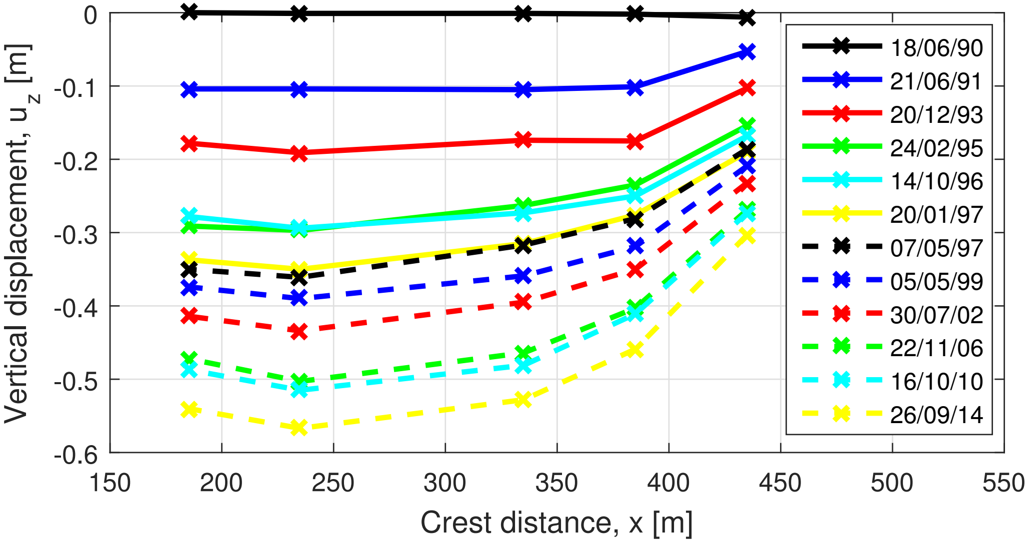
Vertical displacement, u
z
[m]
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
1985
1990
1995
2000
2005
2010
2015
Time, t [Date]
ECMI-1
ECMI-2
ECMI-3
ECMI-4
ECMI-5



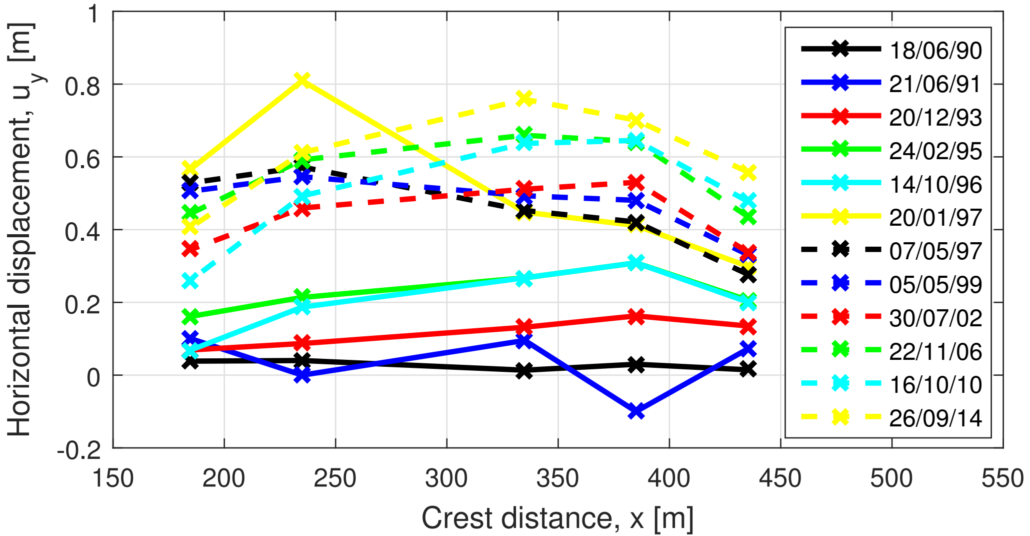
Horizontal displacement, u_y [m]
Time, t [Date]
ECMI-1
ECMI-2
ECMI-3
ECMI-4
ECMI-5
1
0.8
0.6
0.4
0.2
0
-0.2
1985
1990
1995
2000
2005
2010
2015



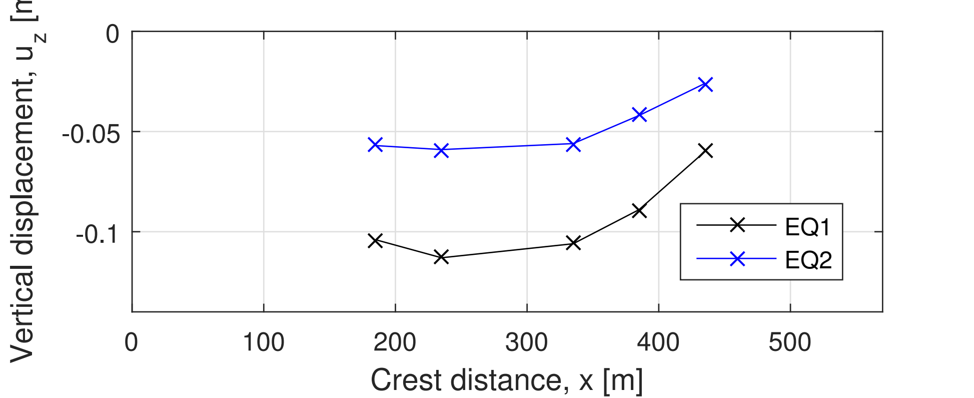
Monitored
Simulated
Reservoir level, h_w [m]
Time, t [Date]
0
20
40
60
80
100
120
1985
1990
1995
2000
2005
2010
2015



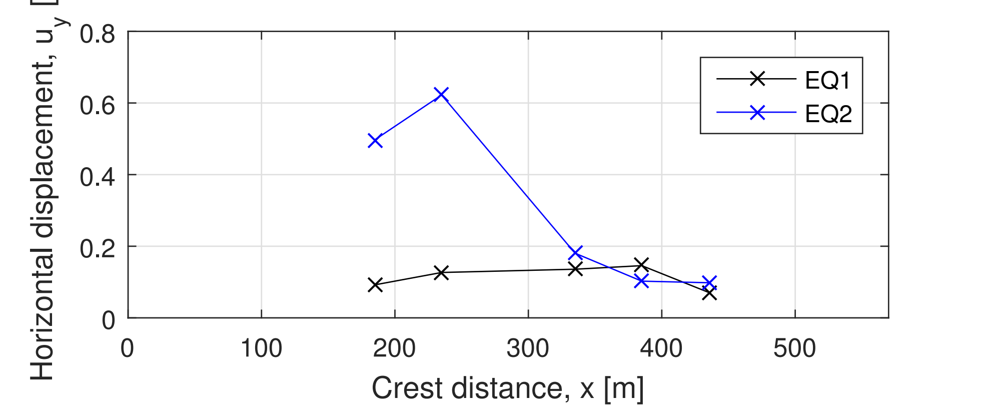
Vertical displacement, u_z [m]
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
150
200
250
300
350
400
450
500
550
Crest distance, x [m]
18/06/90
21/06/91
20/12/93
24/02/95
14/10/96
20/01/97
07/05/97
05/05/99
30/07/02
22/11/06
16/10/10
26/09/14



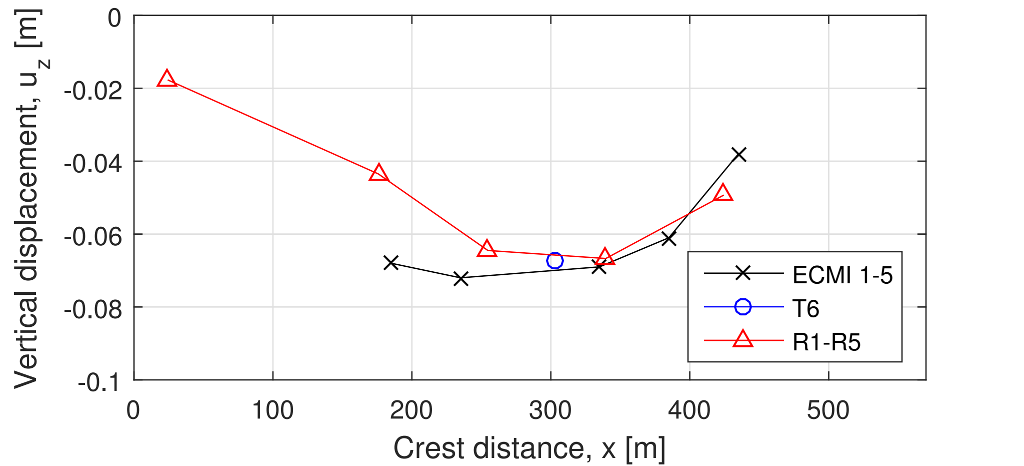
Horizontal displacement, u_y [m]
Crest distance, x [m]
1
0.8
0.6
0.4
0.2
0
-0.2
150
200
250
300
350
400
450
500
550
18/06/90
21/06/91
20/12/93
24/02/95
14/10/96
20/01/97
07/05/97
05/05/99
30/07/02
22/11/06
16/10/10
26/09/14



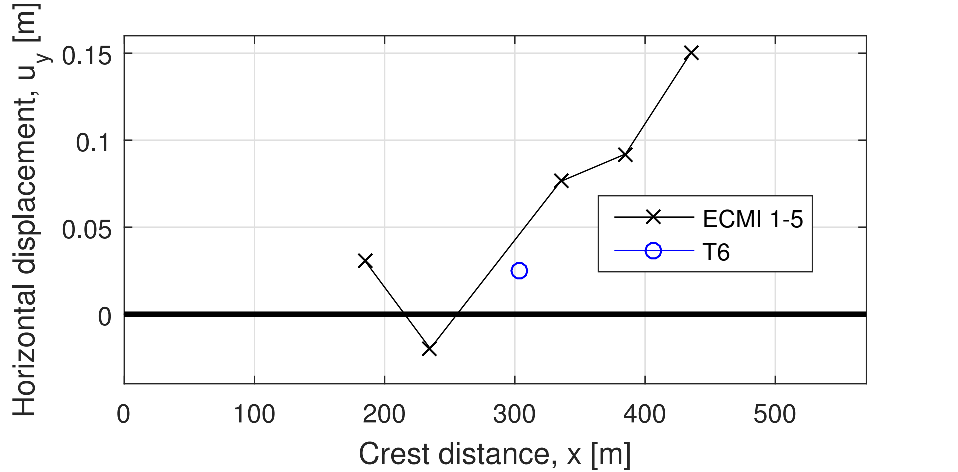
Vertical displacement, u_z [m]
0
-0.05
-0.1
0
100
200
300
400
500
Crest distance, x [m]
EQ1
EQ2



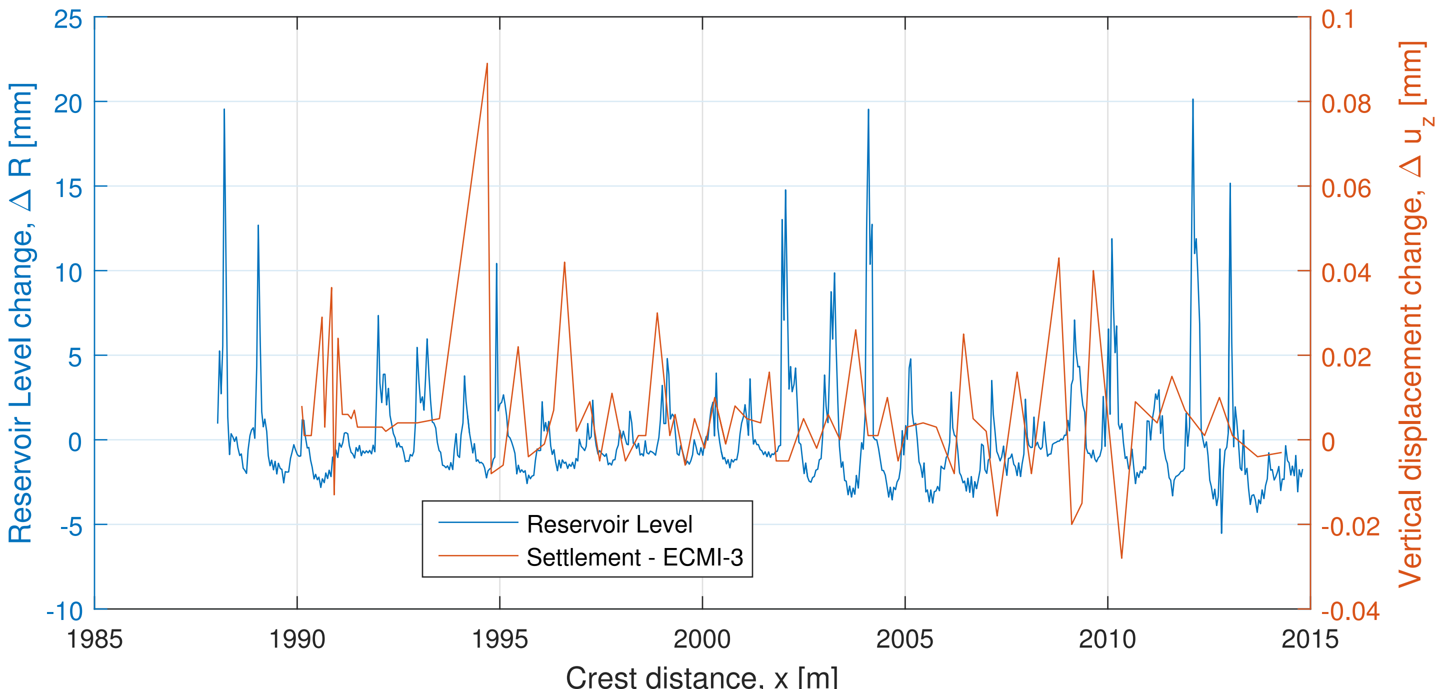
Horizontal displacement, u_y [
0.8
0.6
0.4
0.2
0
0
100
200
300
400
500
Crest distance, x [m]
EQ1
EQ2



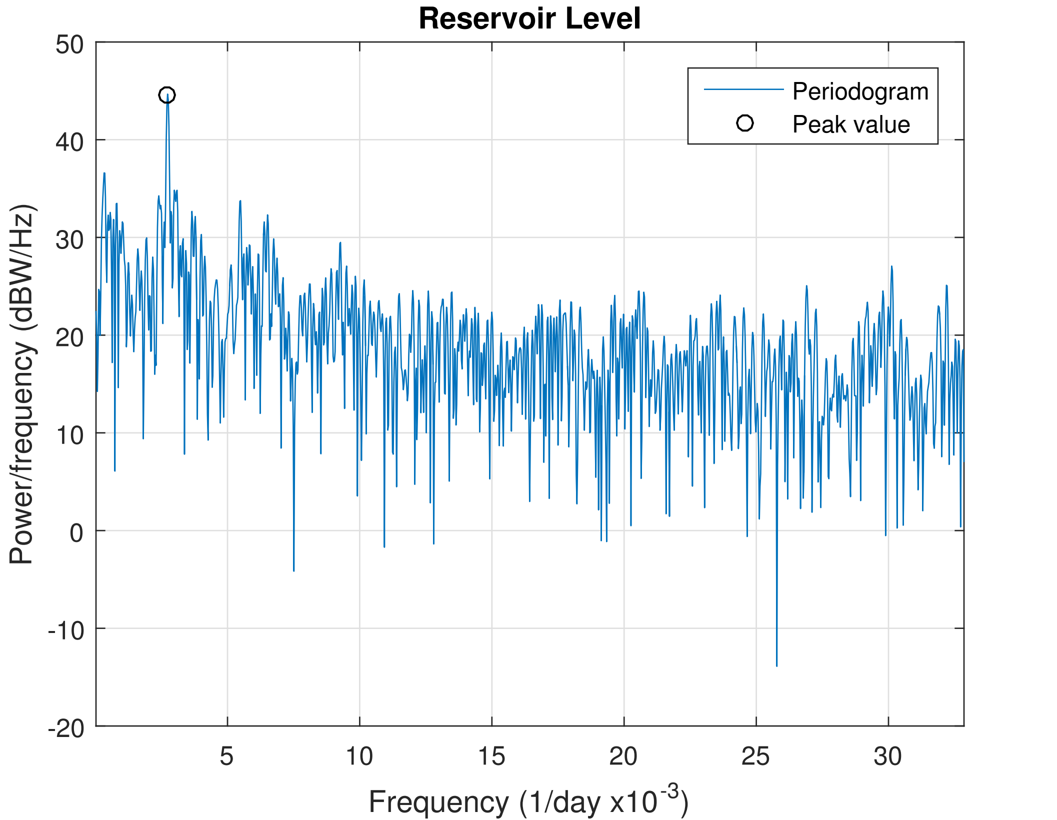
Vertical displacement, u_z [m]
0
-0.02
-0.04
-0.06
-0.08
-0.1
0
100
200
300
400
500
Crest distance, x [m]
ECMI 1-5
T6
R1-R5



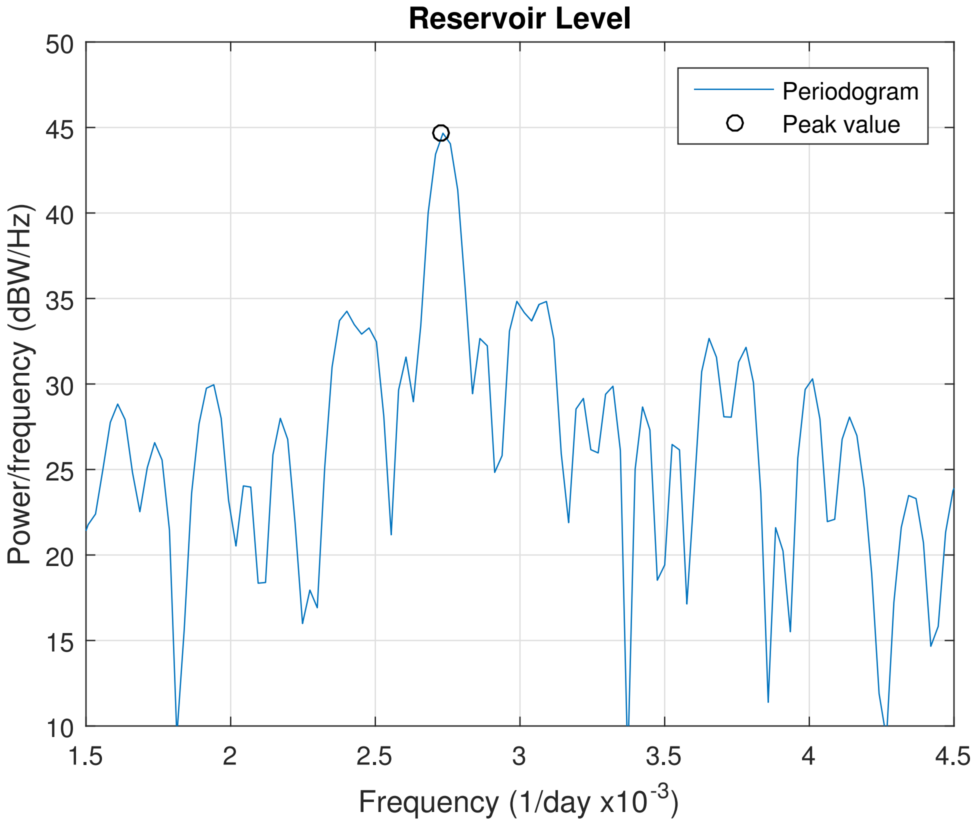
Horizontal displacement, u_y [m]
0.15
0.1
0.05
0
0
100
200
300
400
500
Crest distance, x [m]
ECMI 1-5
T6



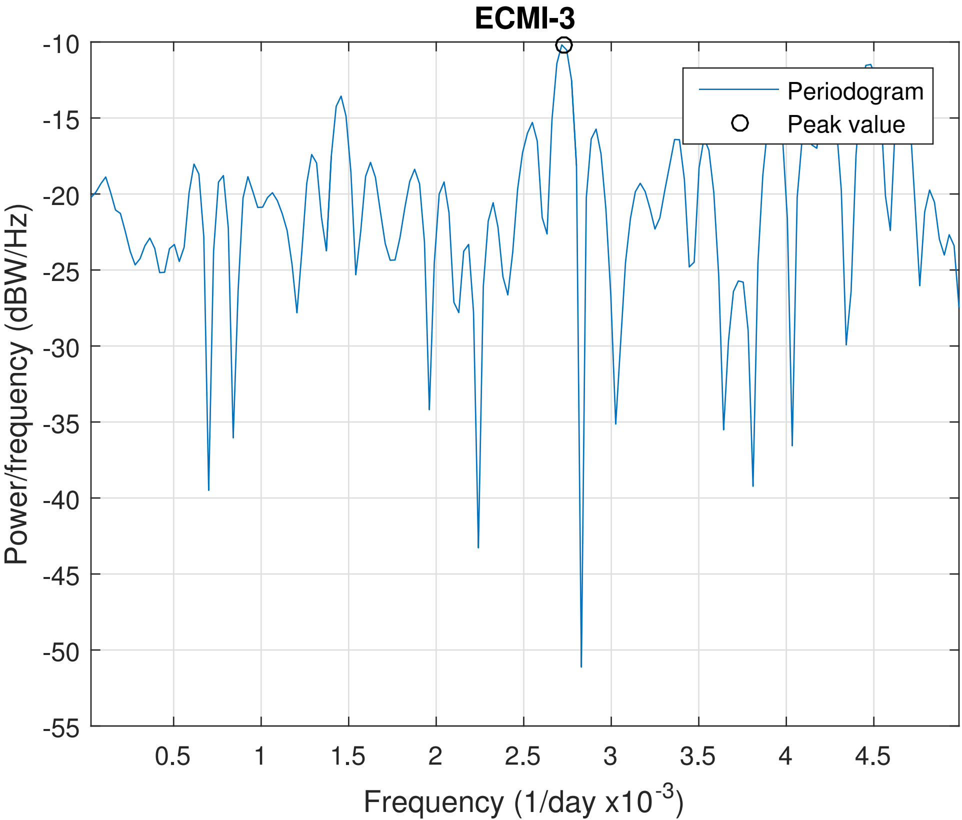
Reservoir Level change, Δ R [mm]
Vertical displacement change, Δ u_z [mm]
Crest distance, x [m]
Reservoir Level
Settlement - ECMI-3
25
20
15
10
5
0
-5
-10
0.1
0.08
0.06
0.04
0.02
0
-0.02
-0.04
1985
1990
1995
2000
2005
2010
2015

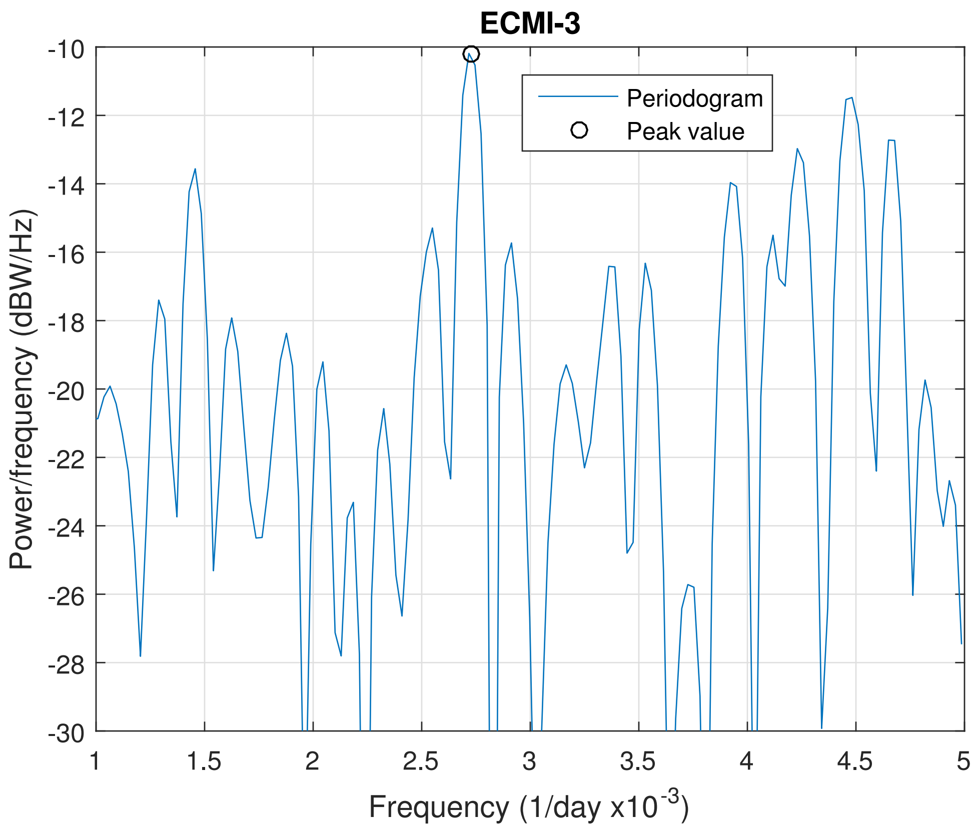


Reservoir Level
Periodogram
Peak value
Power/frequency (dBW/Hz)
Frequency (1/day x10^{-3})
50
40
30
20
10
0
-10
-20
5
10
15
20
25
30

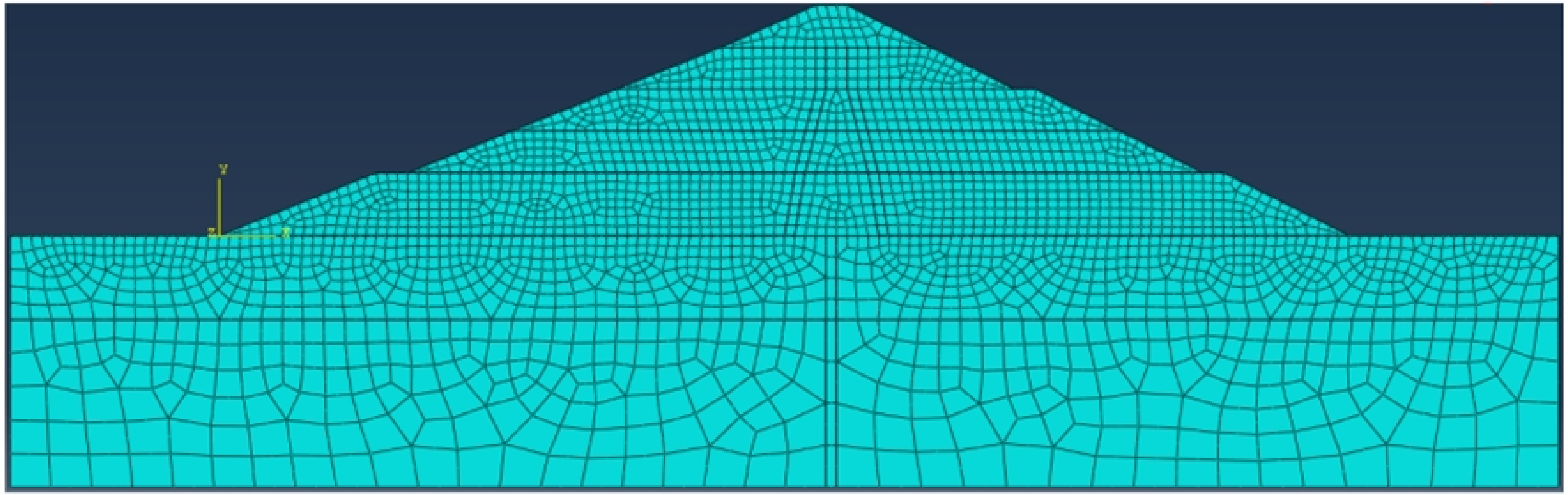


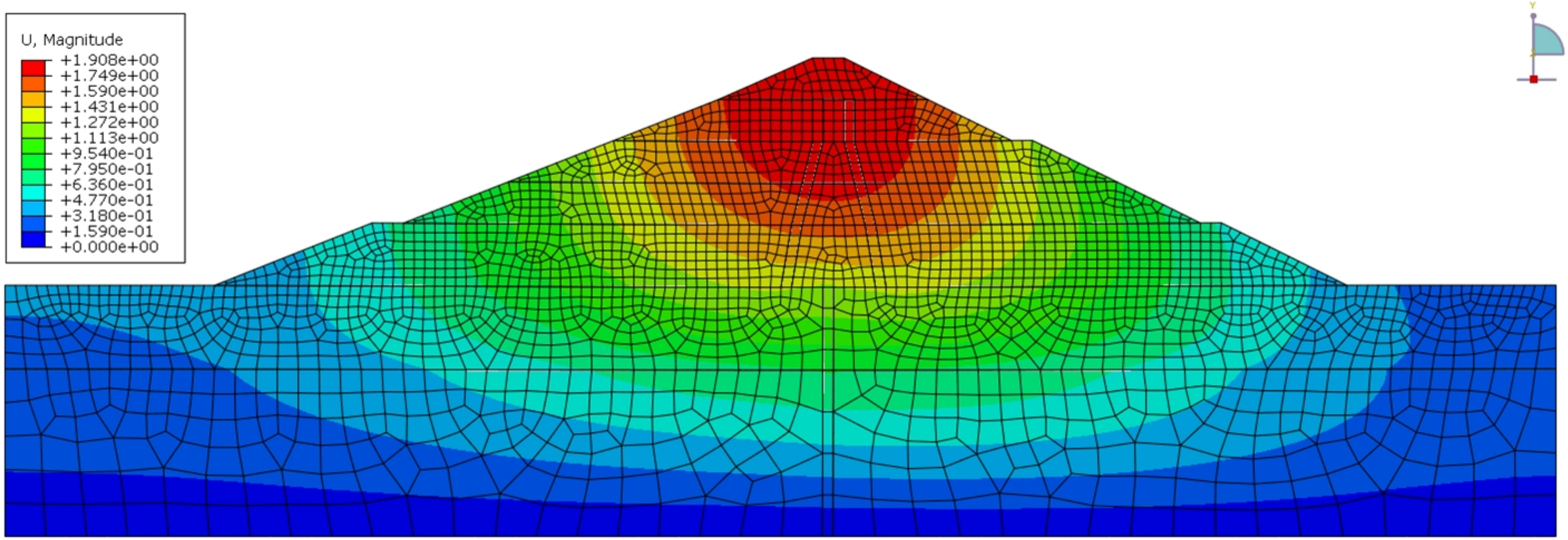
Reservoir Level
Periodogram
Peak value
Power/frequency (dBW/Hz)
Frequency (1/day x10^{-3})
50
45
40
35
30
25
20
15
10
1.5
2
2.5
3
3.5
4
4.5

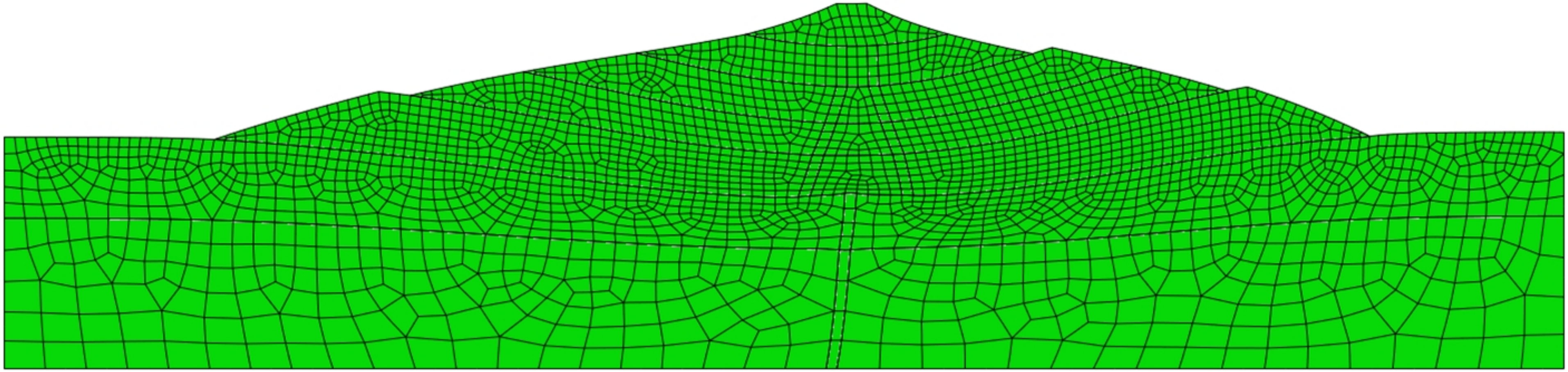

ECMI-3
Periodogram
Peak value
Power/frequency (dBW/Hz)
Frequency (1/day x10^{-3})
-10
-15
-20
-25
-30
-35
-40
-45
-50
-55
0.5
1
1.5
2
2.5
3
3.5
4
4.5

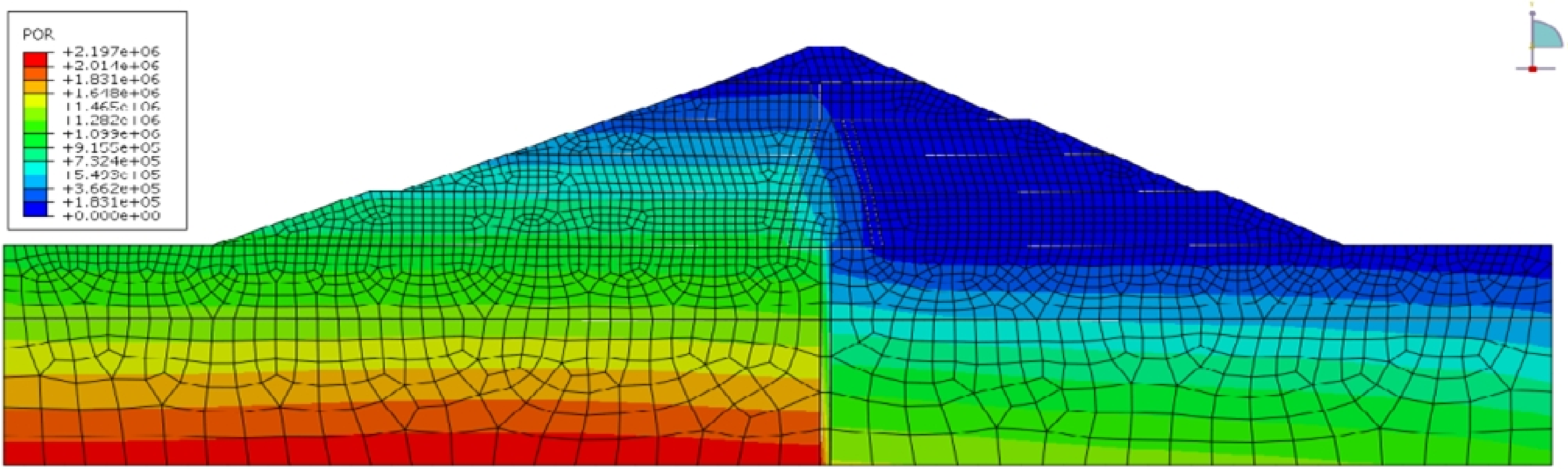


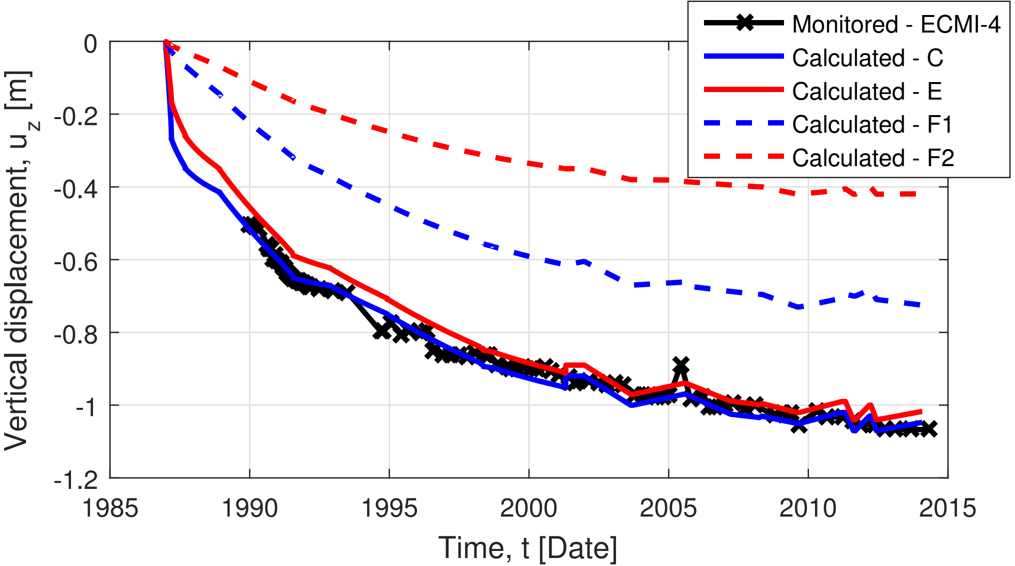
ECMI-3
Periodogram
Peak value
Power/frequency (dBW/Hz)
Frequency (1/day x10^{-3})
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
-30
1
1.5
2
2.5
3
3.5
4
4.5
5

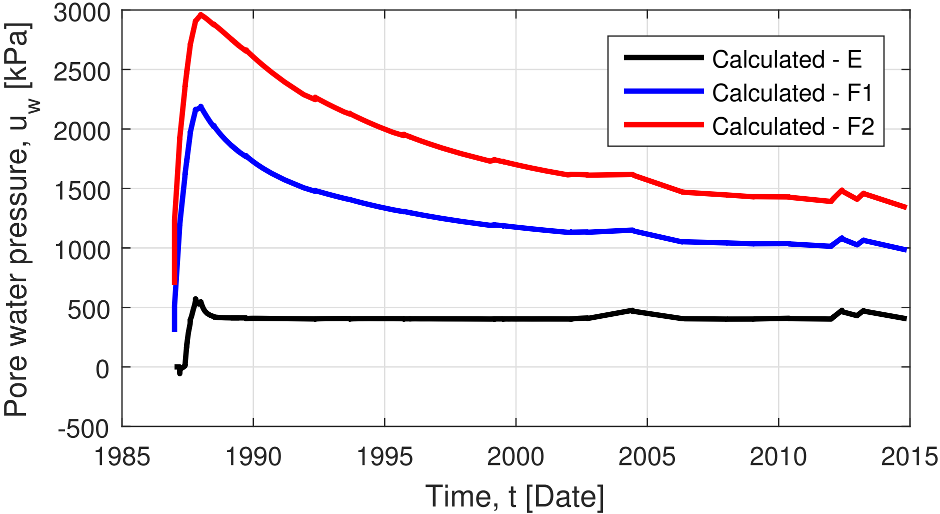



U, Magnitude
+1.908e+00
+1.749e+00
+1.590e+00
+1.431e+00
+1.272e+00
+1.113e+00
+9.540e-01
+7.950e-01
+6.360e-01
+4.770e-01
+3.180e-01
+1.590e-01
+0.000e+00




POR
+2.197e+06
+2.014e+06
+1.831e+06
+1.648e+06
+1.465e+06
+1.282e+06
+1.099e+06
+9.155e+05
+7.324e+05
+5.493e+05
+3.662e+05
+1.831e+05
+0.000e+00


Monitored - ECMI-4
Calculated - C
Calculated - E
Calculated - F1
Calculated - F2
Vertical displacement, u_z [m]
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
1985
1990
1995
2000
2005
2010
2015
Time, t [Date]



Pore water pressure, u_w [kPa]
3000
2500
2000
1500
1000
500
0
-500
1985
1990
1995
2000
2005
2010
2015
Time, t [Date]
Calculated - E
Calculated - F1
Calculated - F2