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Runway 10-28 of the International Airport of Thessaloniki, Greece: The Implementation of an Extensive Embankment in the Sea

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Abstract: The extension of runway 10-28 of the International Airport of Thessaloniki by 1000 m to the sea comprises the construction of a 5.500.000 m^3 embankment with its perimetric armouring, ground improvement techniques, measures of inducing/accelerating settlement. The main construction issues stemmed from the fact that the embankment material was transported by sea, therefore it was built by equipment from the sea, as well as from the poor subgrade conditions (compressible clay) causing settlement of considerable size. A critical design parameter for the operation of the project was the attainment of primary consolidation and stabilization of the settlement rate at 3 mm/month, which proved very challenging because of wide fluctuation. The whole process of settlement stabilization took about 8.5 years to complete, secondary settlement being still in progress.

Key words: sand-gravel, ground improvement, settlement rate, surcharge

1. Introduction

Thessaloniki International Airport has two intersecting runways, 10-28 which was extended by 1000 m into the sea and rebuilt in the existing land section, and 16-34, which served the airport while 10-28 was being extended and rebuilt. The two projects gave the airport of Thessaloniki a runway of uniform strength and characteristics 3440 m long, a full length parallel taxiway and full lighting integrated with the second runway.

The project aims at expanding the airport market to non-stop long haul destinations and upgrading the airport to a competitive hub

Project Owner: Civil Aviation Authority

Supervising Authority: Ministry of Infrastructure and Transport

Construction: AVAX SA – AKTOR SA

Total project cost amounted to: 187.935.000€, funded by national contribution, European Union contribution (European Regional Development Fund 2000-2006/Cohesion Fund 2007-2013) and private funding (by the CAA).

The scope of this paper is firstly to introduce a detailed technical description of the embankment works, according to the approved technical study and tender documents, and then to analyze the specific issues caused by the fact that the infill material was transported by sea, subsequently building the embankment with equipment from the sea, managing such a volume of construction material, and finally comment on the critical design parameter for the operation of the project, i.e., attainment of primary consolidation and stabilization of the settlement rate.

2. Project profile

2.1 Technical Characteristics

Total runway length after the extension: 3440 m.

Additional runway strip: 150 m.

Runway width: 60 m (including shoulders).

Taxiway width: 44 m (including shoulders).

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Distance between runway and taxiway centerline: 182.5 m.

Perimetric service road: 2700 m, width 5.5 m.

The structure was implemented by an extensive embankment, of 1063 m on the longest side, 906 m average length and width of 400÷450 m.

The embankment is armoured by a perimetric concrete wall seated on rockfill, to prevent erosion from incoming waves. Total length of the perimetric armouring 2262 m.

The new runway is equipped with full lighting installation integrated with the second runway 16-34, approach lighting for ILS II operations from the new threshold 10 installed on a 750 m pier based on an iron pile system.

The structure extends to a sea depth of 14 m.

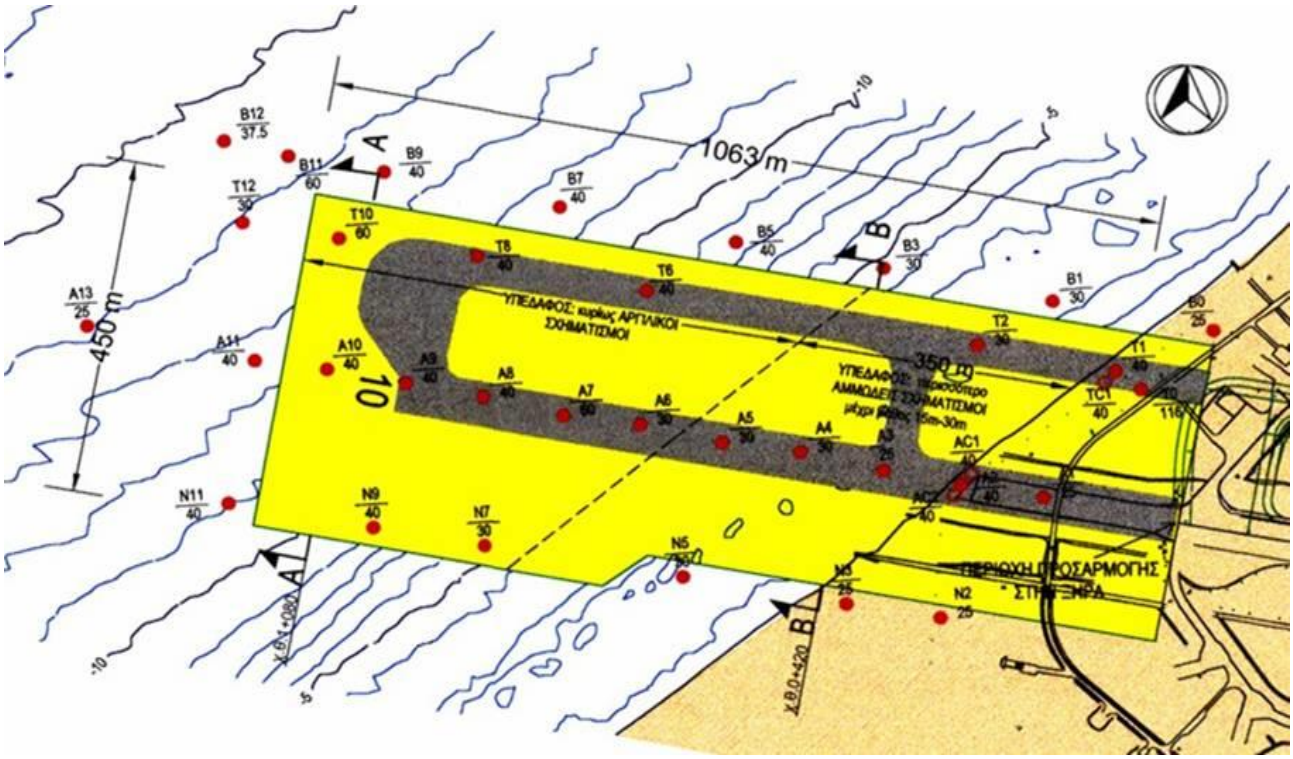

Fig. 1 Extension of runway 10-28.

2.2 Main Works Quantities

Land and sea bottom excavation: total volume 1.540.000 m^3

Total sand-gravel used to build the embankment: 4.780.000 m^3

(taking into account material loss due to settlement, approximately): 5.500.000 m^3

Perimetric armouring concrete (concrete blocks and concrete wall): 65.000 m^3

Rockfill and natural stone blocks of perimetric armouring: 850.000 m^3

Aggregate pavement layers: 112.000 m^3

Asphaltic layers: 42.000 m^3

2.3 Earthworks and Ground Improvement

To investigate underground conditions, a geotechnical study was carried out, with 35 boreholes at land and sea, total length 1324 m, maximum depth 116 m. The underground consists of sand to a depth of 15-30 m, at a distance up to 350 m from shore, changing to compressible clay with enclaves of sand at random depths, further than 350 m from shore.

The materials utilized for construction were natural sand-gravel for the main embankment, sand at the bottom of the embankment, serving as a drainage layer (sand blanket) for the vertical drains, rockfill and natural and precast concrete blocks for the perimetric

armouring. The natural sand-gravel should have a fines content of less than 7%, a smooth grain size distribution and maximum particle size 200 mm. The material should be free of organic and clay matter, plasticity index PI ≤ 6.

Given the underground conditions and the below sea granular infill, ground improvement techniques were required to:

- improve poor underground conditions
- improve the engineering properties of the sand-gravel and rockfill mass.

Measures for ground improvement were:

- installation of vertical plastic drains, at the extents of the compressible clay (at a distance 350 m from shore)
- deep vibroflotation of the granular embankment from a level +1.0 to +2.0 m
- dynamic compaction at the land and shallow sea section (up to 5 m depth), to eliminate liquefaction risk of the subgrade
- surcharging of all bearing sections (runway, taxiways, perimetric armouring), using the building materials in segments of 100-200 m.

Works commenced by excavating the inappropriate soil material to a depth of 1m to 4 m. Then the sand blanket, 1m thick, was laid followed by the vertical plastic drains which were installed at the extents of the compressible clay, i.e., where sea depth exceeded 5.0 m (at a distance of 350 m from shore) (Fig. 2).

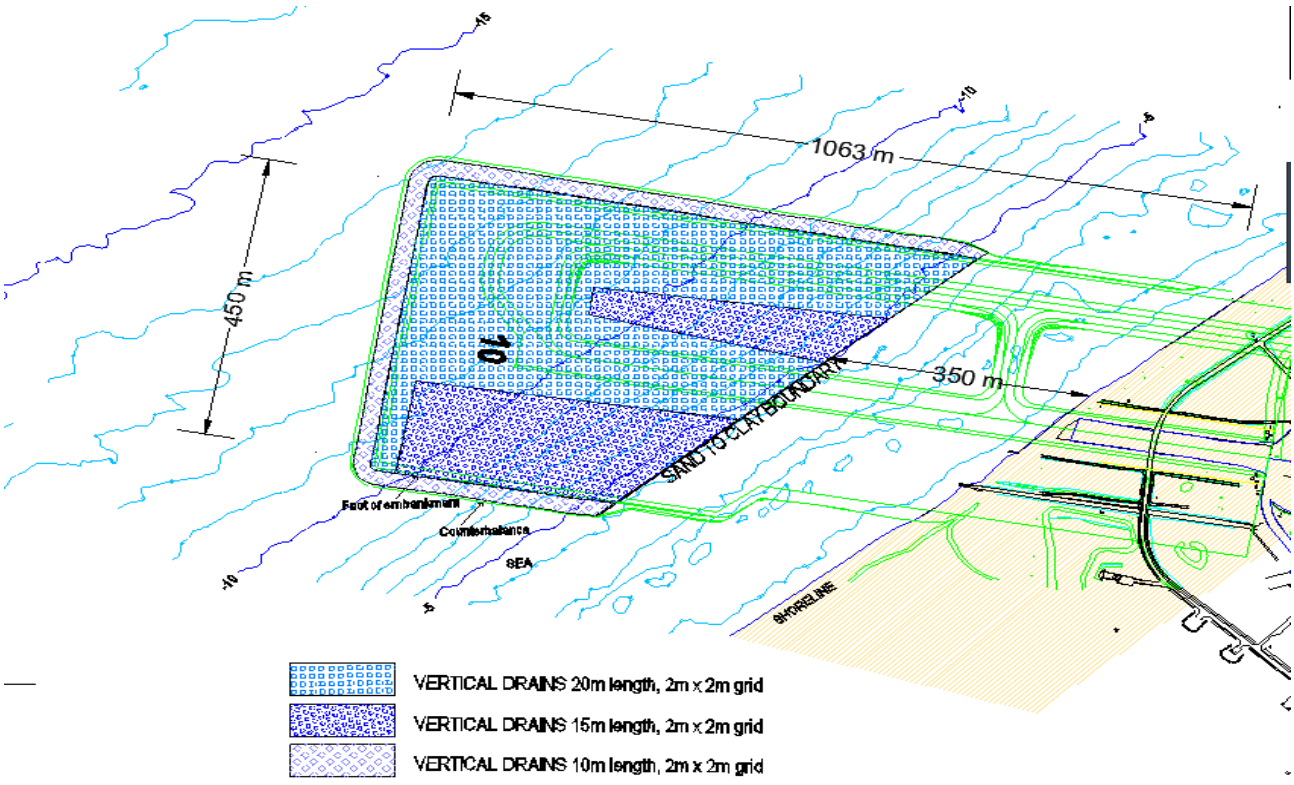

Fig. 2 Installation of vertical plastic drains.

The drains are composed of a plastic core encased by a geotextile. As construction load gradually increases, and subsequent surcharge is applied, pore water pressure initially increases but gradually pore water flows to the nearest drain and to the overlying sand blanket, thus expediting consolidation of the slow draining clay. Drains were installed in a 2 m × 2 m grid, and length: 10-15-20 m.

Built-up of the embankment followed using natural sand-gravel as the infill material, in conjunction with the perimetric armouring piers.

When the embankment reached level +1.0 m -2.0 m, deep vibroflotation was implemented to compact the loose granular embankment infill below mean sea level. A vibrating probe using water and mechanical vibrations caused the grain structure to collapse thereby re-arranging and densifying the treated area. Vibroflotation parameters (grid, duration of vibration) were determined by site trials in order to adjust to the specific equipment and material. The grids finally chosen were triangular 3.5 m for the most powerful S-700 vibroflot (power 180 W) and 3.0 m for the smaller S-340 vibroflot (power 150 W), further adjusted based on achieved results and power of vibroflot.

The craters created were backfilled with the same granular material. Required compaction degree:

≥ 75% at runway/taxiway ($N_{SPT} \geq 30$ to a depth of -10m and $N_{SPT} \geq 40$ to a depth -10 m to -20 m)

≥ 65% at non bearing sections ($N_{SPT} \geq 20$ to a depth of -10 m and $N_{SPT} \geq 30$ to a depth -10 m to -20 m)

Compaction tests were carried out in a 25×25 m grid by cone and standard penetration tests (CPT, SPT).

Dynamic compaction was conducted to eliminate liquefaction risk of the subgrade at the runway/taxiway embankment of the land section and up to 350 m from shore, where vertical drains were not installed. The technique assumes the controlled fall of a steel plate, hoisted on a crane, in a gradual process from a thinner to a denser grid. Dynamic compaction was implemented from level +1.0 to +2.0 m. The specific parameters were determined by site trials and consisted of the controlled fall of a 15t steel plate, in a 4 steps process:

- initial grid 7 m × 7 m, drop height H = 17 m
- second phase, compaction at the center of the above grid
- third phase grid 5 m × 5 m, drop height H = 15 m
- last phase 2 m × 2 m, drop height H = 4 m

All bearing structures (runway, taxiways, perimetric armouring) were surcharged, using the building materials:

- the sand-gravel at runway/taxiways to erect a 5m high temporary embankment
- the core rockfill at the perimetric armouring up to the final level of each section

Surcharging was specified by the tender documents to be implemented in segments of 100-200 m.

Total length of surcharging: r/w + t/w: 2560 m

armouring: 2210 m

The criterion for surcharge removal was the stabilization of the settlement rate at less than 3 mm/month.

The intent of ground improvement and means of accelerating settlement was to induce the greatest part of the expected settlement during construction and diminish the long-term differential settlement (secondary and creep deformation) to the internationally accepted (ICAO) standard of 4 cm per 50 m of runway/taxiway.

2.4 Marine Works

The embankment is armoured against erosion from incoming waves by a perimetric concrete wall seated on rockfill. Total length of the perimetric armouring: 2262 m.

A typical cross-section comprised:

- Excavation: 1.0 m to 4.0 m
- Refill With Sand-Gravel
- Rockfill of Various Gradings
- Concrete Wall consisting of: 1 to 3 precast concrete blocks and cast in situ upper structure providing a curved finish to deflect and redirect sea water away from the structure and reduce the overtopping potential (Fig. 3).

Vertical plastic drains, 10 m and 20 m long, were also installed at a distance 350 m from shore.

The footing was reinforced with rockfill and natural stone blocks to formulate a 25 m wide counterbalance to the embankment forces.

The core rockfill was initially built up to the final level of the upper structure, serving as preloading for at

least three months, while settlement rate was monitored. The preloading material was removed and moved forward to another segment, when the rate of settlement stabilized at ≤ 3 mm/month. Then a levelling course was laid and the precast concrete blocks were subsequently placed into position on the levelling course. In total 902 concrete blocks columns were erected, the upper structure cast on top of them.

The highest level of the perimetric armouring is +4.58 m above mean sea level at the deepest sea segment of the embankment (western side), decreasing to 2.04 m and 1.80 m at the northern and southern edge, respectively.

The specific dimensions and shape of the cantilever resulted from laboratory research carried out by the Laboratory of Harbor Works of the NTUA (Fig. 4).

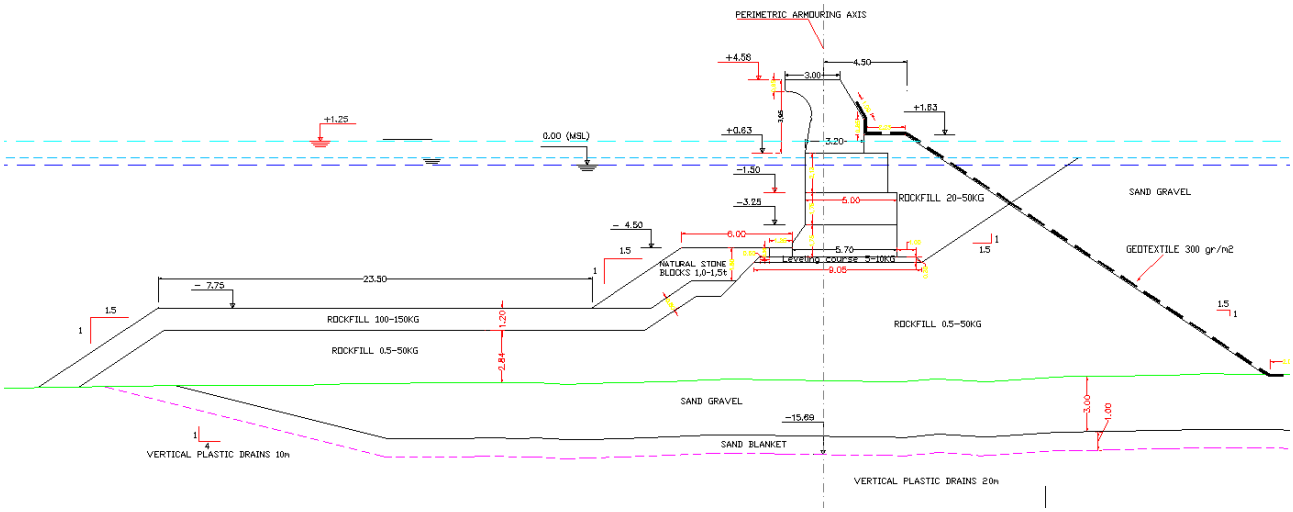

Fig. 3 Perimetric Armouring (typical cross-section at the deepest sea segment).

Fig. 4 Final shape of the upper structure and special mold utilized.

2.5 Geotechnical Instruments

The progress of the settlement and the general behaviour of the construction was monitored via the installation of:

Vibrating Wire Piezometers, either driven in place or in boreholes, Casagrande Piezometers, to monitor dissipation of pore water pressure.

Settlement Plates on the sea bottom surface and Surface Settlement Markers, to directly record settlement values.

Magnetic Extensiometers, Inclinometers, to monitor horizontal shifts of the embankment and the perimetric armouring (Fig. 5).

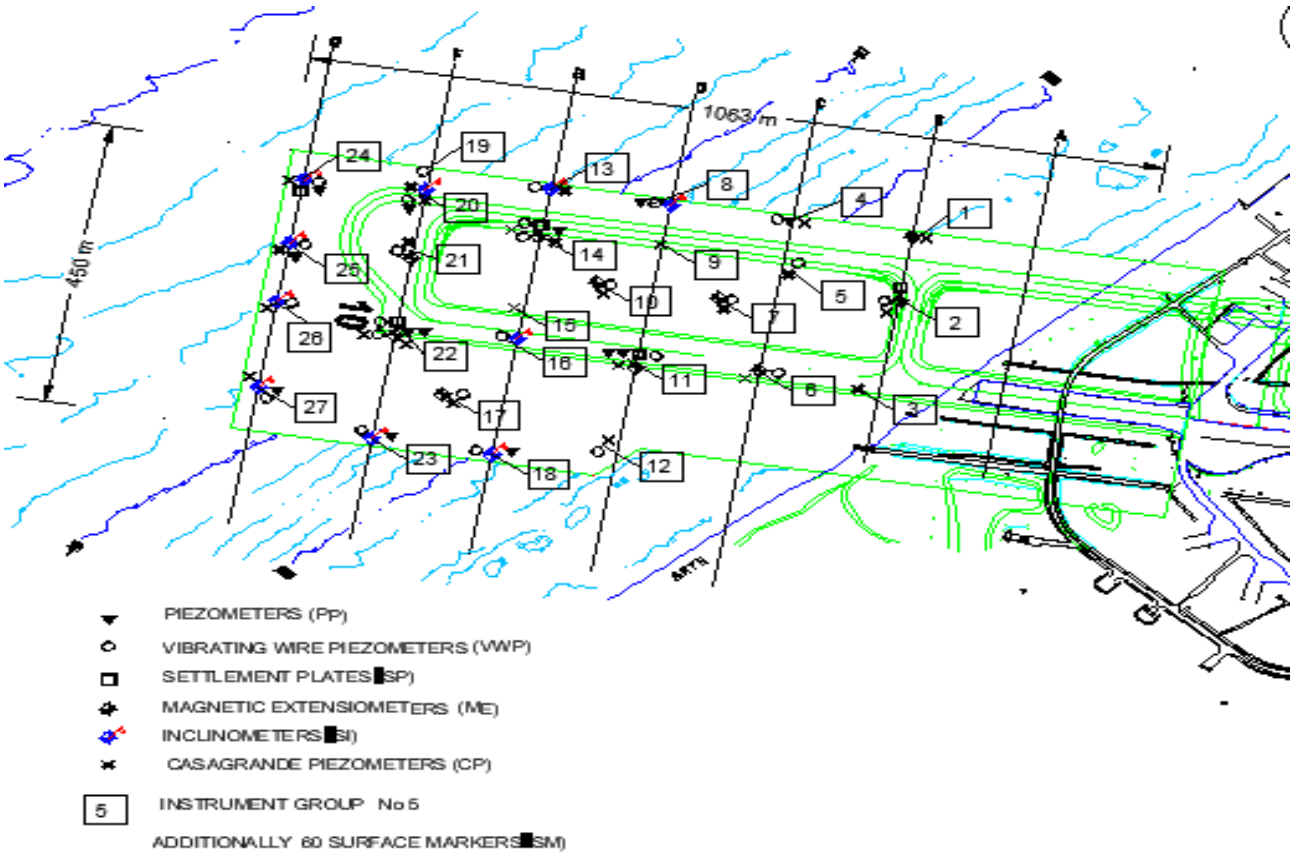

Fig. 5 General layout of geotechnical instruments positions.

Piezometers were drilled into the sea bottom at various depths, settlement plates were installed on the sea bottom and the rest of the instruments were installed in boreholes from level +1.0 - +2.0 m.

Monitoring the settlement progress was crucial because the criterion for surcharge removal was the stabilization of the settlement rate at less than 3 mm/month.

3. Construction Material Management

3.1 Sand-gravel Origin – Sea Transportation

Availability of such a great volume of construction material, inevitably altered the basic construction parameter given by the geotechnical study and tender documents, which was building the embankment from land.

Natural sand-gravel came from the licensed natural quarry of Litohoro Pieria, 120 km away from the construction site. Therefore, sea transportation was the only appropriate means because of the long distance, due to greater transport capacity and to avoid excessive road traffic load (app. 500 truck routes per day to transport the required volume of sand-gravel (~5.500.000 m^3).

Building the embankment from sea meant:

- direct discharge from the split barges to the operational depth of the barges, -3.50 m,
- the remaining quantity from -3.50 to the final layers dragged ashore by dragline and transported to position by trucks.

Two dragline handling positions were utilized, inside the perimeter of the project, for construction economy reasons (direct discharge of the barges, minimization of the truck routes).

The perimetric armouring was being developed at the same time, except for the western side (BC), being used as an inlet for the barges (Fig. 6).

Fig. 6 Dragline handling positions – Barges inlet into the project perimeter.

Handling the incoming sand-gravel from the sea, caused local accumulation of fine particles, which formed enclaves of fine particles (silt, sand, particle size < 0.075 mm) in various contents up to 30% (>>> of the specified 7%) and in various heights (from 0.40 m up to 3.5 m) on the bottom as well as in-between the body of the embankment. The accumulation of fine particles in the body of the embankment resulted from material wash-out during the discharge and dragging process, but the accumulation of fine particles at the bottom of the embankment was mainly sea bottom sediment deposited into the excavation trench in the meantime until commencement of sand-gravel transportation.

The accumulation of fine particles at the runway and taxiway bearing embankments prohibited the achievement of the required compaction degree.

To resolve the resultant situation:

- the sand-gravel handling positions were moved outside the perimeter of the project, where sea depth was 6.5 m to 7.5 m.
- the fine particle buildup before the dragline handling positions, rising to 2.5 m, was removed and fresh, sound material was brought in place. Volume of accumulated material removed and replaced: 220.000 m^3.
- the perimeter of the project was sealed, implementing the rockfill core and required preloading of the armouring at full length. This led to an excess rockfill 0.5-50 kg requirement of 75.000 m^3 which was in the end utilised in the construction of the embankment.

Areas where fine particles had already been trapped into the body of the embankment:

- were initially re-treated by vibroflotation (in the centre of the triangular grid) with poor results. The vibroflot could not penetrate deeper than 4 m, but also vibroflotation is

effective in pure granular soils, but not in the presence of high content of fines,

- a course of dynamic compaction was subsequently implemented, which demonstrated satisfactory results up to a depth of 8 m,
- the presence of large accumulations of fines deeper than 8m was investigated after the preloading process which at the end exhibited remarkable results,
- evaluation of the corrective measures was supported by drilling investigative boreholes, supplementary to penetration testing.

The performance of the embankment on seismic as well as operational loading was investigated and proved satisfactory:

- liquefaction potential at the base of the embankment was very low and only where there was sand-clay enclaves, estimated to give post-earthquake settlement ranging from 3.5 to 11.6 cm, situated locally, therefore not affecting the overall safety of the structure,
- analysis under the operational loads estimated that the presence of enclaves up to 2 m at depths > 6 m, did not affect the geotechnical performance of the overall structure. But zones ≥ 3 m not satisfactorily compacted were retreated and re-evaluated after preloading.
- overall, the extent of the enclaves of fine particles did not compromise the safety of the overall structure.

3.2 Gross Settlement

According to the geotechnical study, settlement derives from the following main mechanisms:

A) Primary consolidation of the subgrade. Accounts for 90-95% of total settlement of the clay subgrade and 80-85% of total vertical deformation. Maximum primary settlement was estimated 2.4 m. Surcharging and soil draining by the vertical drains aim at expediting primary consolidation, so that the largest part is assumed during construction (within 1.5 year after each segment is finished).

B) Secondary consolidation. It was estimated to account for 5-10% of total settlement and evolves in a period of 50 years.

C) Deformation of the embankment granular soil. Results from rearranging the grains of the construction material and was estimated in the order of 1% of the embankment height, i.e., maximum deformation approximately 20 cm. Deep vibroflotation and surcharging aim at assuming this deformation during construction.

D) Long-term creep deformation of the granular soil was estimated in the order of 0.5% of the embankment height, i.e., maximum deformation approximately 10 cm, assumed in a period of 10 years.

Total vertical deformation estimated by the geotechnical study (including primary and secondary consolidation, creep) ranged from 18cm at the land section to 2.95 m at the deepest sea section.

The settlement plates installed at the sea bottom surface and the surface markers installed at the base of the surcharge loading provided the most straightforward and quantifiable measures of settlement, verifying the estimations of the geotechnical study. The gross settlement recorded by the settlement plates ranged from 0.60 m to 2.5 m, while the settlement derived from surface markers readings ranged from 5 cm to 67 cm. The overland section of the embankment exhibited 5 cm to 19 cm settlement, as derived from the installed surface markers. Table 1 summarizes the recorded gross settlement values.

A generally uniform relation of the size of settlement to the height of the embankment was observed, regardless of the location of the embankment, s/h ratio ranging from 0.11 to 0.15, whereas the lower s/h ratio are observed at the edge of the upperstructure. The greatest part of total settlement, accounting to 70% to 85%, took place during construction, whereas the settlement induced by the surharging accounted for a

small portion of total settlement, progressing at a very slow rate, as will be shown in the following chapters.

Table 1 Gross Settlement.

Settlement Plate	Installation Level	Level of Embankment	Settlement s (m)	s/H	Surface Markers (m)
LAND	-	2.4	0.60	0.15	0.051-0.188
SP-2	-1.5	4.9	1.42	0.14	0.11-0.14
SP-11	-5.6	5.3	1.85	0.11	0.22-0.33
SP-14	-11.7	5.3	1.53	0.11	0.20-0.37
SP-22N	-9.0	5.3	2.34	0.11	0.31-0.56
SP-24	-16.2	4.8	1.10	0.07	0.30-0.44
SP-28	-11.2	4.58	2.51	0.13	0.27-0.57
SP-29	-14.1	4.58	1.07	0.06	0.29-0.56
SP-30	-12.0	4.58	0.99	0.06	0.57-0.67
SP-31	-	-	-	-	-

3.3 Surcharging Duration

The duration of the surcharge loading was estimated by the geotechnical study at least 3 months per segment of application, 100-200 m long, and 39 months in total.

The actual stay, though, of the surcharging load proved dramatically longer: placement and actual overall stay of the surcharge lasted from September 2009 until 02-05-2018, i.e., 103.4 months or 8.5 years.

To shorten the surcharge stay, it was applied at much longer segments:

- at the runway/taxiway embankment the surcharge length varied from 280 m to 1010 m, averaging 528 m per 3/month periods,
- at the perimetric armouring the surcharge length varied from 130 m to 1220 m, averaging 680.5 m per 3/month periods.

The surcharge lengths were increased to the extent possible, depending on the rate of the incoming sand-gravel from the quarry and the material equilibrium, so as no excess (redundant) quantity of sand-gravel was transported. To this intent, soil excavated from an adjacent construction site was utilised in a segment of the taxiway.

Additionally, at the deeper segment of the runway, approximately 260 m long, the height of the surcharge was raised to 8 m, in order to expedite the rate of settlement.

At the perimetric armouring, excess rockfill 0.5-50 kg was brought on site to materialize increased lengths. The excess rockfill, amounting to 75.000 m^3, was utilised in the construction of the embankment.

By all the afore mentioned measures, the duration of surcharging:

a) ranged between 7.7 and 14 months at the land and shallower sea section, while at the deepest sea section reached 38 months at the runway/taxiway embankments, with a mean surcharge duration of 17.8 months. Even in the last segment of the runway where increased surcharge load was implemented, surcharging duration was limited only to 25.8 months.

b) ranged between 7 and 60 months at the perimetric armouring, with a mean surcharge duration of 30.9 months.

3.4 Monthly Rate of Settlement/Removal of Surcharge

According to the geotechnical study and the tender documents “The surcharge load is removed when the settlement-time curve acquires a gentle slope, when the rate of settlement stabilizes at 3mm/month”.

Stabilization of the rate of settlement at the required 3 mm/month, in order to remove surcharge and proceed to pavement layers, proved in practice very challenging to achieve:

- exhibited very slow progress,

- a small portion of the total settlement required α disproportionately long time to be assumed, time which was counterproductive, as opposed to the primary consolidation the greatest part of which was assumed during construction, thus not affecting the construction time schedule.

Fig. 7 is an example of the variation of the monthly rate of settlement in a segment of the runway with a sea depth 7-10 m: while the rate of settlement takes about one year to decrease from 40-60 mm/month to 10mm/month, stayed below 10 mm/month for up to 16 months before the upper limit of 3 mm/month was reached.

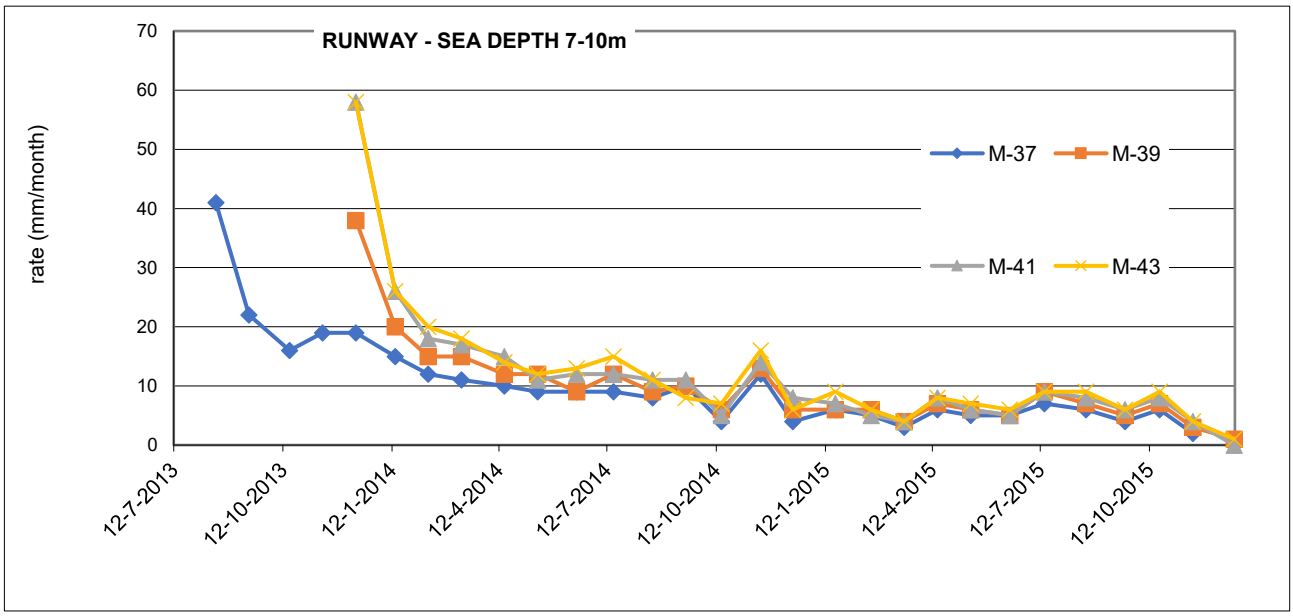

Fig. 7 Variation of settlement rate over time.

Given the settlement rate progress at the previously surcharged segments of the runway and taxiway, to expedite settlement at the deepest runway section, which was the most critical because of the sea depth, an additional 3 m surcharge load was implemented. The increased surcharge load confined the surcharge stay from 3 years (estimated by the historical data and the simulation model implemented) to 25.8 months. The additional load was implemented in conjunction with modifying the design criterion for removing the surcharge to 8 mm/month for 4 consecutive weekly measurements and provided that pore-water pressure stabilized for at least two months. Goes without saying that pavement layers could not be laid unless the 3 mm/month criterion was met.

Stabilization of the monthly rate of settlement at the deepest part of the runway was also influenced by the ongoing works at adjacent areas (surcharge removal, final embankment layers, rockfill works, pavement layers). Because of the height and magnitude of the embankments, this influence resulted in a wide fluctuation of the settlement rate at the surcharged segment comprising even negative figures (heave), as is distinctively shown in Fig. 8.

The monitoring of settlement continued during paving works by surface markers (installing new ones if needed) and finally asphalt layers were implemented only when:

- the treated area was not affected by surcharge removals at adjacent segments and exhibited stabilization of the settlement rate
- the settlement rate indicated by all surface markers in the treated area was below the required standard of 3 mm/month

- no signs of heave were indicated the unbound aggregate layers of adjacent areas had been completed and properly compacted.

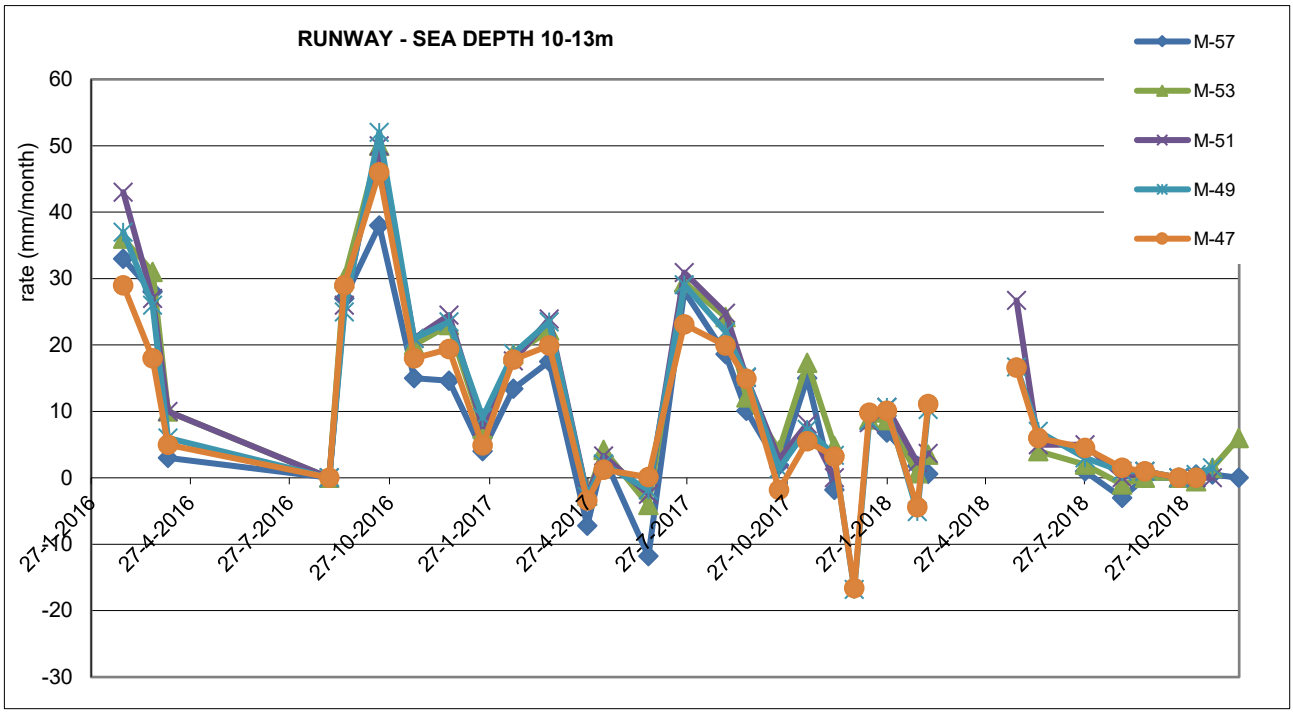

Fig. 8 Fluctuation of monthly settlement rate due to ongoing works at adjacent areas.

4. Post-construction behavior

The project was completed in 15-03-2019, and after all the necessary licencing processes, the runway was put in operation in September 2020.

According to the geotechnical study, secondary consolidation takes place over a period of 50 years after project completion, starting 1-2 years after completion of the embankment and estimated in the order of 30 cm at the end of the 50 year period, at a greater rate at first, diminishing over time (Fig. 9).

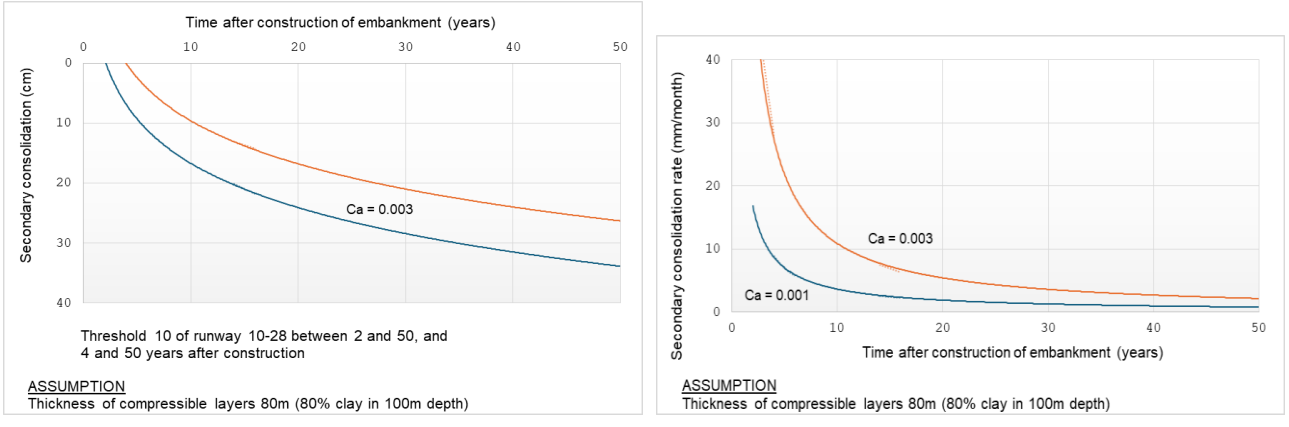

Fig. 9 Secondary consolidation over time.

According to the airport concessionaire measurements, the maximum value of secondary settlement recorded in 07.10.2022 (a period of approximately 43 months after project completion) was 125 mm.

Surveying measurements taken by the contractor at 8 control points at the runway southern shoulder in a period of approximately 20 months (17.04.2021 – 02.12.2022) exhibited a maximum settlement of 38.3 mm or 1.93 mm/month.

5. Conclusion

The overall behavior of the structure is very good, after five years of operation, no occurrence of horizontal shifts or slide or breakage of the subgrade or the embankment. The recorded secondary consolidation is in the order of the estimated by the geotechnical study. In any case, the structure is being monitored by the geotechnical instruments that remain after completion of construction and transfer readings to a dedicated workstation. These readings, in conjunction with settlement measurements from surface markers, form the basis of monitoring so as to ensure the average decrease of the recorded long term settlement and prepare in advance any mediation measures.

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References

[1] Sotiropoulos and Associates S.A., Geotechnical Consultant to the contractor, Technical Reports.

[2] Triton Consulting Engineers, Technical Consultant to the Ministry of Infrastructure & Transport, Technical Reports.