Structural Assessment of Underground Water Reservoirs

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ABSTRACT

This paper presents the methodology and results for the investigation of the structural safety of 40 aged underground fresh and brackish water reservoirs, to support the weight of photovoltaic (PV) systems that will be placed on their roof slab panels. The procedure included (1) review of available drawings, specifications, maintenance history, and design calculations; (2) visual inspection of the roof slab panels; (3) carrying out a series of noninvasive and invasive nondestructive tests; (4) strength, serviceability, and durability verification; and (5) analysis of results.

Out of the 40 reservoirs, 24 tanks passed the visual inspection phase and were considered for further investigation. The roof slab panels of the underground water reservoirs that passed the visual inspection were subjected to a series of noninvasive and invasive tests that included infrared thermography, impact echo, ultrasonic pulse velocity, Schmidt hammer, concrete core compressive strength, water soluble chlorides content, and sulfates content. The study demonstrated that the infrared thermography is an economic and useful tool in the visual inspection of large concrete surfaces. It was concluded that out of the 40 inspected tanks, 23 tanks are found to be adequate to support an extra weight of PV systems on their roof slab panels.

1. INTRODUCTION

Engineering experience shows that the assessment of existing structures can be divided into three phases as shown in Fig. 1 (RILEM 2001). Each of these phases should be completed in its specific unit and own merit with the required deterministic or reliability-based analyses.

This paper presents the results of investigating 40 aged underground fresh and brackish water reservoirs to support the weight of photovoltaic (PV) systems that will be

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placed on their roof slab panels. The reservoirs are distributed in 10 various locations in Kuwait.

The investigated underground reservoirs typically consist of cast-in-situ bottom slab (raft), precast and/or cast-in-situ columns with enlarged sections at both ends, cast-in-situ external walls and internal partitions, and cast-in-situ roof slab panels that have a typical plan dimension of 10 m x 10 m. Each slab panel is supported on four columns at 6.5 m c/c span and 1.75 m cantilever in each direction (Fig. 2). The slab panels are structurally separated without any reinforcement crossing the joints separating them. Fig. 2a shows a typical unit of the roof slab system and Fig. 3 shows the roof system for a complete reservoir with an overall dimension of 200m x 200m x 6m with a typical capacity of 45 MIG (Million Imperial Gallon).

Typically, the roof slab panels of the water tanks are covered with a four-layers drainage system that consists of water insulation (polyethylene sheets or bituminous paint), a 10 cm thick gravel layer, a geotextile filter fabric, and a 20 cm thick sand layer (Fig. 4). However, the roof slab panels for five reservoirs are completely exposed.

The assessment process of the reservoirs consisted of five main tasks (1) data collection, (2) visual damage assessment, (3) field and laboratory testing, (4) structural analysis, and (5) data analysis and conclusions.

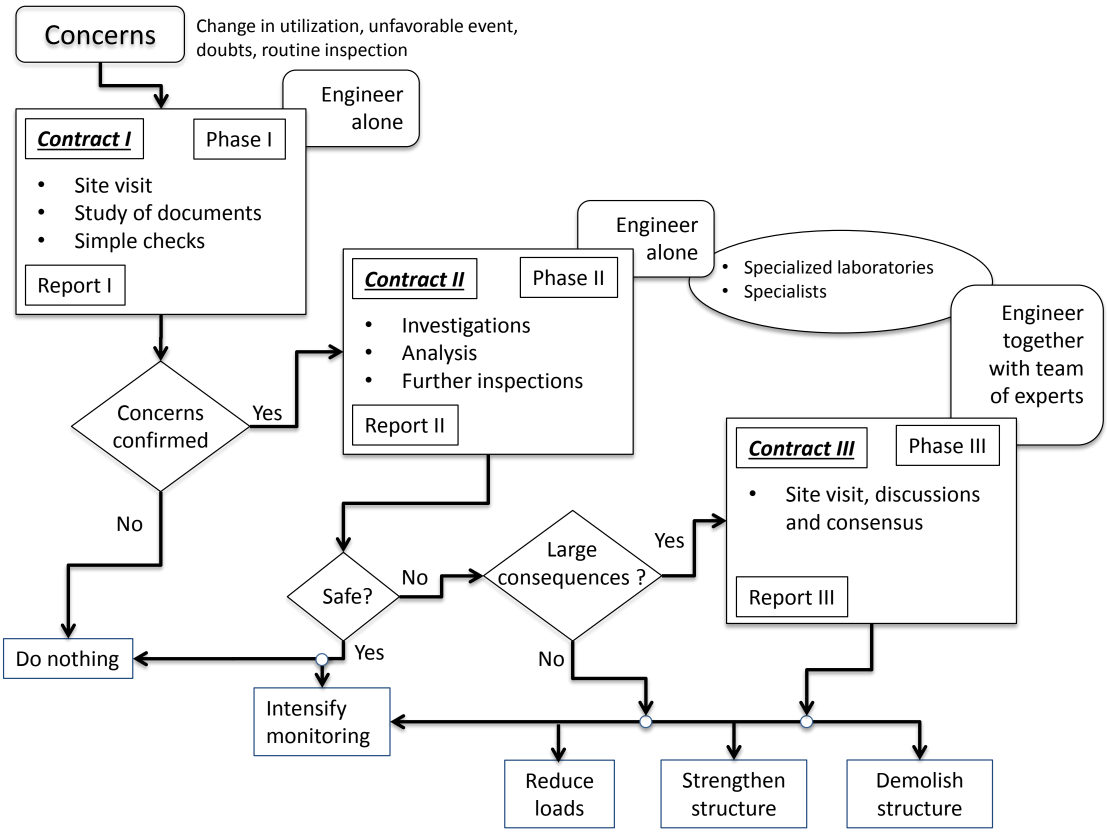


Fig. 1. Illustration of the three phases approach (RILEM 2001).

The retention of water within liquid retaining structures is obviously the most important function of water reservoirs. Therefore, the testing program was designed mainly for analyzing material properties according to the American Concrete Institute (ACI) 228.1R and 228.2R (ACI 228.1R 2003; ACI 228.2R 2013) from nondestructive (ND) invasive and noninvasive testing techniques, which are considered the best sources of information to evaluate the condition of such structures without jeopardizing their functionality and safety. However, invasive ND tests, such as core tests, are important means for more accurate data. Therefore, a testing program that relies mainly on noninvasive ND tests with a minimal number of invasive ND tests was developed. The results of the cores compressive strength was used to correlate the Schmidt hammer and the indirect ultrasonic pulse velocity (UPV) tests with concrete compressive strength. The assessment process in this paper was mainly according the RILEM and ACI 364.1 (ACI 364.1R, 2007; RILEM, 2001).

(a) (b)

Fig. 2. Typical unit of the roof slab panels supported on four precast columns.

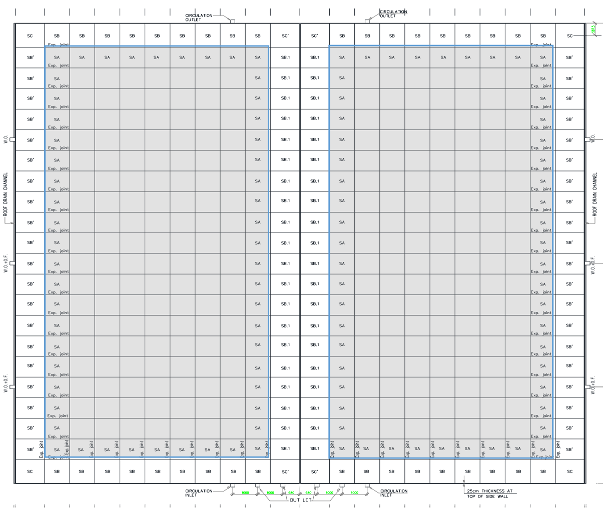


Fig. 3. Arrangement of roof slab panels of a typical 200x200 m underground water reservoir.

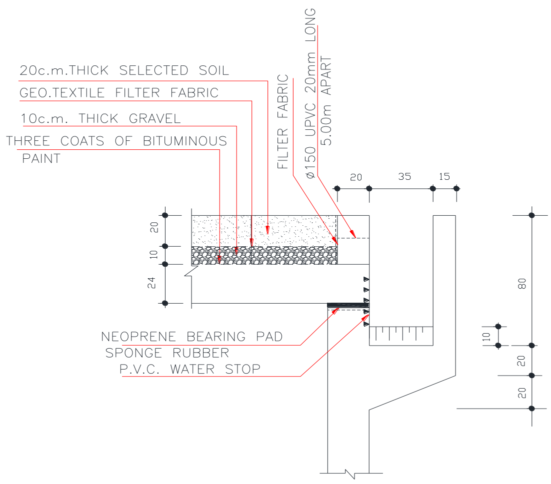


Fig. 4. Typical section through the gutter and the roof drainage system.

2. Results

2.1 Data collection

During this phase, the investigating team collected and reviewed all submitted documents, drawings, specifications, and calculations that are related to the concerned reservoirs. The main reviewed documents were design calculations, structural design drawings, records of major damages and maintenance history, concrete mix designs, and construction method statements.

The data collection task revealed that not all needed documents were available as the reservoirs are very old and some of the drawings and documents are missing. Consequently, some extra tasks were added to the fieldwork plan to collect further data.

2.2 Visual Damage Assessment

The visual inspection of the reinforced concrete (RC) roof slab panels was conducted according to ACI 201.1R (ACI 201.1R, 2008) and it was a challenging task as the top surface of 35 reservoirs were completely covered with a 30 cm thick drainage system (Fig. 4) and because some of the reservoirs are painted with bitumen. Most of the reservoirs had a surface are of 40,000 m^2. It was impossible and very costly to expose the top surfaces of the RC slab panels and the team was not permitted to use any remote controlled small boats inside the tanks. Therefore, we decided, in agreement with the owner, to expose the top surface of the reinforced concrete slab panels of the reservoirs at most thought critical locations, especially near the water inlets and outlets. In average, the top surfaces of each tank was exposed at 6 locations with a total number of 230 locations in all the tanks. The exposure area at each location was approximately 5x5 m (Fig. 5).

The general condition of the inspected slab panels was mainly judged based on the spalling of concrete, cracks, deposit formations, and the condition of the reinforcing steel. Twenty-two tanks were found to be in good condition, seven tanks were satisfactory, and eleven tanks were in poor condition, as listed in Table 1. Fig. 6 shows the top surface of an RC slab for one of the reservoirs that is suffering from severe concrete spalling caused by advanced reinforcement corrosion.

In this task, it was observed that 11 tanks have bitumen paint and 12 tanks have polyethylene sheets. It was also noticed that 17 tanks had neither polyethylene nor bitumen paint (including the 5 completely exposed tanks), and 14 tanks only were provided with geotextile filter.

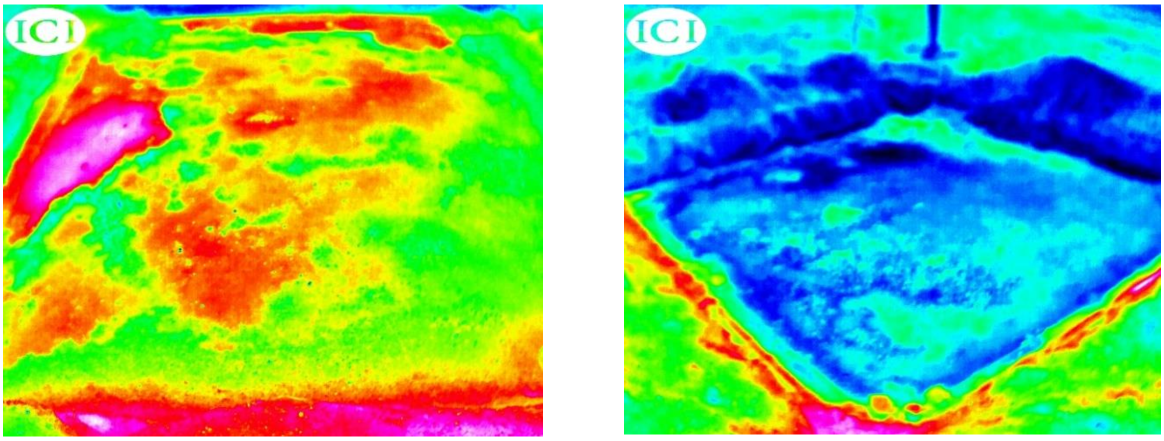
The visual inspection of the bitumen painted surfaces was very challenging because we were not able to see and examine the concrete surface condition. To expose these surfaces, we tried to use two methods; solvents and sandblasting. The latter method was faster and much more effective; however, it was very expensive. Therefore, we decided to use infrared thermography (IRT). Fig. 7 shows IR images for two of the inspected reservoirs.

It was also noticed that the coarse aggregate particles in the drain system were sticking and penetrating the bitumen paint in the tanks that are painted with bitumen.

Overall, it was concluded from this stage that the presence of the bitumen paint or polyethylene sheets was crucial for the protection of the concrete against moisture and, consequently, reinforcement corrosion.

Fig. 5. Typical exposed area of roof slab panels.

Fig. 6. Concrete spalling due to reinforcement corrosion in one of the severely damaged reservoirs.



(a) (b)

Fig. 7. Infrared images of (a) a damaged slab panel, and (b) a slab panel in good condition.

Table 1. Overall condition of roof slabs of each reservoir

Site	*Number of tanks*	*Overall Tank Condition*		
		Good	*Satisfactory*	*Poor*
Sulaibikhat	6	4, 5, 6	1	2, 3
Mutlaa High	5	A, B, E	D	C
Mutlaa Low	4	A, B, C, D	-	-
Subhan	4	D	A	B, C
West Funaitees	8	G, H	F	A, B, C, D, E
Mina Abdullah	7	B, C, D, E, F, G	A	-
Sulaibiya	2	A (37 MIG)	B (30 MIG[1])	-
Shagaya	1	-	A (15 MIG)	-
Wafra	1	-	-	A (37 MIG)
Umm Gudair	2	A, B		
Total	40	22	7	11

[1] Million Imperial Gallon

2.3 Field and Laboratory Testing

Several noninvasive and invasive ND tests were conducted to evaluate the concrete quality of the roof slab panels. Table 2 lists the field and laboratory tests that were carried out in this project. These tests included the UPV test, Schmidt hammer test, concrete compressive strength by core testing, carbonation depth, chlorides content test, sulfates content test, and half-cell potential test. The concrete cores were extracted from places that do not affect any of the reservoir's functionality, serviceability, and durability.

In total, 166 locations were tested for the indirect UPV and the rebound number. Four UPV readings were taken at each location. In total, 93 concrete cores were extracted and tested for their compressive strength. The indirect UPV and rebound number tests were measured at the same locations where the cores were extracted.

2.3.1 Concrete Compressive Strength

In total, 93 concrete core samples were extracted from all the investigated sites. The cores were tested according to American Society for Testing and Materials (ASTM) C42 (ASTM C42/C42M, 2013) and the average compressive strength was found to be 502 kg/cm^2 with a standard deviation of 105 kg/cm^2. Table 3 summarizes the average (μ) compressive strength and standard deviation (σ) of the tested core samples for each site. The compressive strength of all tested cores was above the design value of 250 kg/cm^2, except one outlier core sample, which had a compressive strength of 94.4 kg/cm^2.

Table 2. List of conducted field and laboratory tests

Site	*No. of reservoirs*	*Tests*								*Thickness*		
		Ultrasonic PV	*Schmidt Hammer*	*Compressive Strength*	*Carbonation*	*Chlorides*	*Sulfates*	*Corrosion*	*IR Thermography*	*By Drilling*	*By Impact Echo*	*By Measurement*
Sulaibikhat	6	34	34	6	6	2	2	6	-	-	-	-
Mutlaa High	5	6	6	25	25	2	2	1	14	10	-	-
Mutlaa Low	4	0	0	22	22	2	2	0	11	8	-	-
Subhan	4	20	20	5	5	2	2	5	-	-	-	-
West Funaitees	6	36	36	6	6	2	2	6	-	-	12	-
Mina Abdullah	6	36	36	6	6	2	2	6	-	12	-	-
Sulaibiya	2	12	12	2	2	1	1	2	-	-	-	4
Shagaya	1	6	6	1	1	1	1	1	-	-	-	2
West Funaitees	2	10	10	2	2	1	1	2	-	-	4	-
Wafra	1	-	-	5	5	1	1	-	-	2	-	-
Mina Abdulla	1	6	6	1	1	1	1	1	-	2	-	-
Um Gudair	2	-	-	12	12	1	1	-	-	4	-	-
Total	40	166	166	93	93	18	18	30	25	38	16	6

Based on the compressive strength results of the concrete cores, the UPV (ASTM C597, 2009) and the rebound number (ASTM C805/C805M, 2013), three different formulae were derived, using linear and nonlinear best fit, to estimate the concrete compressive strength. These equations are as shown later on and the results are summarized in Table 4.

- Linear equation that relates the UPV to the concrete cube strength

$$f_{ck} = 0.0134V + 483 \tag{1}$$

- Linear equation that relates the rebound number to concrete cube strength

$$f_{ck} = 6\,R + 190 \tag{2}$$

- Combined SonReb nonlinear equation (Cristofaro et al., 2012) that relates the UPV and the rebound number to the concrete cube strength

$$f_{ck} = 4.88V^{0.107}\,R^{0.933} \tag{3}$$

where,

f_{ck} = Concrete compressive strength (kg/cm^2), V = Ultrasonic pulse velocity (m/s), and R = Rebound number.

2.3.2 Carbonation Depth

The carbonation depth was measured for all the 93 extracted concrete cores, according to the British Standard European Norm (BS EN) 14630 (BS EN 14630, 2006). The average carbonation depth was found to be 2.5 mm. The results are summarized in Table 5.

2.3.3 Half-Cell Potential

Corrosion of reinforcing steel is an electro-chemical process and the behavior of the steel can be characterized by measuring its half-cell potential. The greater the potential the higher the risk that corrosion will initiate. The half-cell potential test was conducted at 30 different locations, according to ASTM C876 (ASTM C876 2009). The results indicated that the reinforcement corrosion is highly unlikely to initiate in all tested tanks.

2.3.4 Thickness Measurements

The thickness of the roof slab panels was measured by three different methods depending on site conditions. The thickness of slab panels for the completely exposed tanks in Sulaibiya and Shagaya were measured directly through the 60 cm x 60 cm ventilation openings that existed at the middle of the slab panels. Impact Echo (IE) test was used to measure the thickness of the slab panels of the tanks in West Funaitees area. All other measurements were made by making holes through the whole thickness of the slab panels. The results are summarized in Table 6. Fig. 8 shows the IE test results for one of the locations in West Funaitees area.

2.3.5 Chlorides Content

Chlorides in concrete can accelerate the corrosion process of reinforcing steel. Steel is naturally protected from corrosion in the high pH (alkaline) environment when embedded in concrete. When chloride ions are present near reinforcing steel, they override this passivation causing the initiation of the corrosion process. Chloride limits are expressed based on chloride ion (Cl^-). However, not all chlorides in concrete contribute to corrosion. Some chlorides are chemically bound in the cement hydration products.

In total, 18 concrete cores were tested for water-soluble chlorides content, according to the ASTM C1218 (ASTM C1218 / C1218M, 2015) where the chlorides concentration profile was measured within 4 slices; 0-15 mm, 15-30 mm, 30-45 mm & 45-60 mm. The profile data of waster-soluble chlorides content for all tested samples are analyzed using Fick's law of diffusion to estimate the service life. The analysis showed that the chlorides content in the roof slab panels of Tank E in Mutlaa High has already exceeded the corrosion threshold level at the reinforcement level of 30 mm and that tank A in Mina Abdalla and the tank in Wafra will reach the reinforcement corrosion threshold level in less than 10 years.

Table 3. Compressive strength results of the tested cores

Area	*n*	μ (kg/cm^2)	σ (kg/cm^2)	*Minimum* (kg/cm^2)	*Maximum* (kg/cm^2)
Sulaibikhat	6	514	87	394	603
Mutlaa High	25	465	110	94.4	664
Mutlaa Low	22	533	119	289	746
Subhan	5	525	84	455	629
West Funaitees	8	526	85	427	662
Mina Abdullah	7	560	101	366	679
Sulaibiya	2	503	-	432	575
Shagaya	1	410	-	410	410
Wafra	5	561	89	488	699
Um Gudair	12	445	54	368	541
Overall	93	502	105	94.4	746

Table 4. Estimated compressive strength

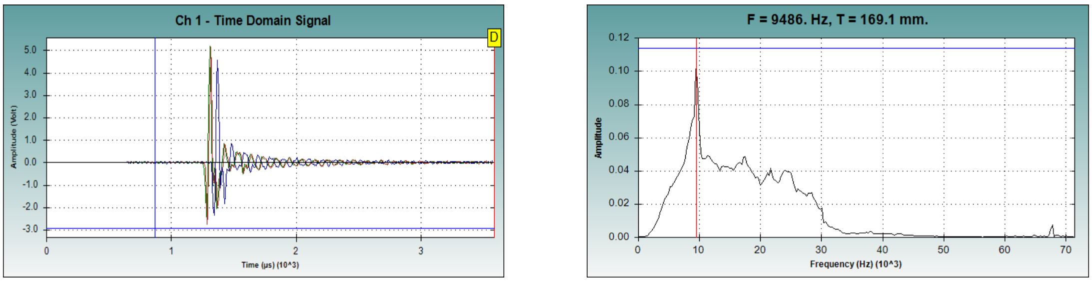
Area	*n*	f_{ck} *Using Eq. (1)*		f_{ck} *Using Eq. (2)*		f_{ck} *Using Eq. (3)*	
		μ (kg/cm^2)	σ (kg/cm^2)	μ (kg/cm^2)	σ (kg/cm^2)	μ (kg/cm^2)	σ (kg/cm^2)
Sulaibikhat	34	523	6	529	15	495	24
Mutlaa High	6	517	3	546	17	508	25
Mutlaa Low	0	-	-	-	-	-	-
Subhan	20	526	8	532	23	503	29
West Funaitees	46	529	8	533	19	506	27
Mina Abdullah	42	527	6	549	13	527	21
Sulaibiya	12	525	13	534	16	499	39
Shagaya	6	527	6	493	22	450	33
Wafra	0	-	-	-	-	-	-
Um Gudair	0	-	-	-	-	-	-
Overall	166	526	8	535	21	506	31

Table 5. Carbonation depth test results

Area	*n*	μ (kg/cm^2)	σ (kg/cm^2)	*Minimum (kg/cm^2)*	*Maximum (kg/cm^2)*
Sulaibikhat	6	3	6	0	15
Mutlaa High	25	1.44	4.44	0	20
Mutlaa Low	22	1.13	1.01	0	15
Subhan	5	5.60	6.19	0	15
West Funaitees	8	8.38	8.07	2	25
Mina Abdullah	7	3.14	5.55	0	15
Sulaibiya	2	13.0	18.4	0	26
Shagaya	1	20	-	20	20
Wafra	5	0	0	0	0
Um Gudair	12	0	0	0	0
Overall	93	2.45	6	0	26

Table 6. Average thickness of slab panels

Area	*n*	μ cm	σ cm	*Minimum cm*	*Maximum cm*
Sulaibikhat	0	-	-	-	-
Mutlaa High	10	20.8	3.10	15	23
Mutlaa Low	8	23.5	0.72	23	25
Subhan	0	-	-	-	-
West Funaitees	16	17.9	2.44	13.4	21.3
Mina Abdullah	14	22.2	2.61	17	25.5
Sulaibiya	4	20.0	0	20	20
Shagaya	2	15	0	15	15
Wafra	2	23	0	23	23
Um Gudair	4	24	2	22.5	26
Overall	60	20.7	3.0	13.4	26



(a) Time domain (b) Frequency domain

Fig. 8. Impact echo test results for one of the locations in West Funaitees area.

2.4 Structural Analysis and Capacity Verification

The installation of the PV systems on the roof slab panels is a long-term investment project, where the panels are required to support the PV systems for at least 30 years. To minimize the risk, a decision-making methodology was established for accepting or rejecting the reservoirs, which will be supporting the PV panels.

Based on this criterion and the results of the visual inspection and tests results, 15 tanks were discarded after the visual inspection and 2 more tanks were discarded after the testing stage. The remaining 23 tanks needed further investigation by structural analysis and capacity verification to estimate the maximum extra live load that can be safely supported by their roof slab panels.

Mainly, strength and crack width were investigated for the 23 reservoirs. AutoDesk Robot software was used to model and structurally analyze the slab panels to estimate the maximum additional live load that can be supported by the roof slab panels and the columns for each reservoir. Strength and crack width were investigated according to the ACI 318 (ACI 318, 2014) and ACI 224R (ACI 224R, 2001), respectively. In the structural analysis, for each reservoir, the actual slab thickness and superimposed dead load (thickness of drain system) were used as listed in Table 7.

The concrete equivalent compressive strength was calculated based on Equation 6.4.3.1 in the ACI 562 (ACI 562, 2016), as shown below in Eq. 4.

$$f_{ceq} = 0.9\overline{f_c}\left[1 - 1.28\sqrt{\frac{(k_c V)^2}{n} + 0.0015}\ \right] = 430\,\frac{kg}{cm^2} = 42.2\ MPa \qquad (4)$$

where $\overline{\boldsymbol{f_c}}$=506 kg/cm^2, V=0.19 (after discarding the outlier core result of 94.4 kg/cm^2), k_c=1.02 (Table 6.4.3.1 ACI 562).

The structural investigation showed that the negative moment capacity (top reinforcement) of the roof slab panels was the controlling criteria for a total live load not exceeding 6 kN/m^2. However, for higher live load, the crack width exceeded the maximum crack width limit of 0.2 mm. The columns were found to be safe and didn't control the maximum live load placed on the reservoirs.

Fig. 9 shows top reinforcement requirements versus surface live load for completely exposed slab panels and those that are covered with drain system layers and the results are shown in Table 8.

Table 7. List of reservoirs that passed the visual inspection and testing stages

Site	*Reservoirs*	*Slab size*	*Thickness (cm)*	*Slab topping*
Sulaibikhat	4, 5, 6	10 x 10	18 (Assumed)	30 cm
Mutlaa High	A	10 x 10	21.5	30 cm
Mutlaa Low	A, B, C, D	10 x 10	23, 23.5, 23.7, 24	30 cm
Subhan	D	10 x 10	18 (Assumed)	35 cm
West Funaitees	F,G,H	10 x 10	20.9, 19, 19.2	Exposed, 30cm, 30cm
Mina Abdullah	B, C, D, E, F, G	10 x 10	22.5, 22, 22.5, 22.5, 23, 25.5	30 cm
Sulaibiya	A,B	10 x 10	20, 20	Exposed
Shagaya	A	10 x 10	15	Exposed
Umm Gudair	A, B	10 x 10	25, 22.8	30 cm

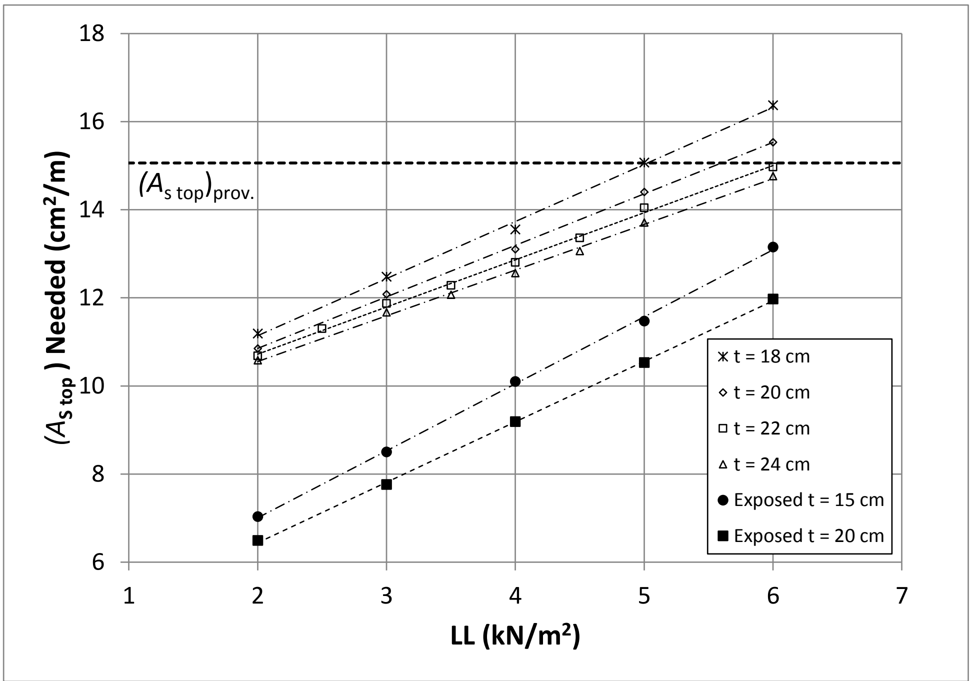


Fig. 9. Top reinforcement requirements versus surface live load for completely exposed slab panels and those covered with drain system.

Table 8 Maximum live load for each reservoir.

Site	*Tanks*	*Max. Total Live Load (kN/m^2)*	*Max. Weight of PV Panels (kN/m^2)*	*Max. Conc. Live Load (kN)*
Sulaibikhat	4, 5, 6	5	3.0	10
Mutlaa High	A	5.6		
Mutlaa Low	A, B, C, D	6		
Subhan	D	5		
West Funaitees	F	6		
West Funaitees	G, H	5.1		
Mina Abdullah	B, C, D, E, F, G	6		
Sulaibiya	A,B	6		
Shagaya	A	6		
Umm Gudair	A,B	6		

3. Conclusions

This paper presents the results of 40 investigated underground water reservoirs that are distributed in 10 different locations in Kuwait. In was concluded in the visual inspection stage that:

1. Twenty-two reservoirs were in good condition, seven tanks were satisfactory, and eleven tanks were in poor condition.
2. Out of the 40 tanks, the roof slab panels of 35 tanks were covered with a drain filter layer while the roof slab panels of the remaining 5 tanks are completely exposed.
3. A proper drain filter layer (polyethylene sheet or bitumen coat + gravel + geotextile filter + sand) is provided for 11 tanks, only while 24 tanks had improper filters and 5 tanks are completely exposed.
4. The overall concrete condition of the roof slabs of the water tanks that are covered with a proper filter system is generally good while the concrete condition of the roof slabs that lack the bitumen or polyethylene layer is generally poor and suffering from advanced reinforcement corrosion.
5. The bitumen coat was provided for 11 tanks, while 12 tanks were provided with polyethylene sheets, and the remaining 17 tanks had neither polyethylene nor bitumen coat.
6. The geotextile filter was provided for 14 tanks only.
7. The coarse aggregate particles were sticking and penetrating the bitumen coat in the tanks that have the bitumen coat. This caused reinforcement corrosion in some locations.
8. The most observed severe type of damage was advanced reinforcement corrosion that led to a large spalling of the concrete cover. Sagging of the slab panels was another very common type of distress.

In the testing stage, several noninvasive and invasive ND tests were done. The main conclusions of this stage are:

1. The results of the UPV, rebound number, and compressive strength of concrete

cores demonstrated that the concrete quality is very good and its compressive strength is high.
2. The measured carbonation depth in all tested samples was smaller than the thickness of the concrete cover.
3. The chlorides content was lower than the corrosion initiation threshold (0.3% of the cement weight according to ACI 318-14) in all tested reservoirs, except for Tank E in Mutlaa High. On the other hand, the sulphates content was high in all tested samples.
4. The results of the half-cell potential test suggested that the reinforcement corrosion is highly unlikely to initiate in all tested tanks.
5. Field measurements showed that the thickness of the top roof slab panels of 27 reservoirs is less than 24 cm (the design thickness that is stated in the design drawings).

Based on the visual assessment and testing stages, it was decided to discard 17 tanks and further investigate the structural capacity of the remaining 23 tanks. It was found in the structural analysis stage out of the 40 examined tanks, 23 tanks are were adequate to support PV panel systems, with a maximum weight of the PV system of 3.0 kN/m^2, and the maximum concentrated load for all the slabs should not exceed 10 kN.

4. Acknowledgments

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