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Observations on groundwater leakage through cracks and segment joints in a concrete segmental tunnel lining

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ABSTRACT: Facing with the combined challenge of aging sewer infrastructure and rapid population growth, the Abu Dhabi Sewerage Services Company (ADSSC) embarked on a bold approach to build an enhanced sewer infrastructure with a deep gravity sewer tunnel at the center. The primary lining of the new 41 km-long deep sewer tunnel has been completed by tunnel boring machines (TBM). The concrete segmental lining has an internal diameter varying from 4 m to 5.5 m internal diameter, and is located at depths of 20 m to 80 m below the ground surface. This paper attempts to quantify the results of observations made on groundwater infiltrations and response to repair works experienced at different stages in construction of concrete segmental lining for deep sewer tunnels. The observations presented below have been based on the actual data obtained from a total of 10,872 rings installed in three TBM drives in Contract T-02 of Abu Dhabi's Strategic Tunnel Enhancement Programme (STEP).

1 INTRODUCTION

In the area of concern of this paper, an earth-pressure balance (EPB) TBM-driven tunnel has been built with a total length of approximately 15.2 km in three separate drives between four work shafts. The total number of rings installed is 10,872 on the alignment of Contract T-02 where the overburden varies from 35 to 60 m. The lining comprises 5 + 1 universal trapezoidal segments with an inner diameter of 5500 mm. The bore diameter (with disc cutter) is 6360 mm. The segment length is 1400 mm, and the lining thickness is 280 mm.

The tunnel lining in Contract T-02 was made up of Steel Fiber Reinforced Concrete (SFRC) segments along with traditional steel reinforcing cages which were used only at both segment joints against bursting stresses under permanent loads. The concrete class is C50 ($f_{ck} = 50$ MPa). A corrosion protection (secondary) lining (CPL) system was formed inside the segmentally lined tunnel by a 250 mm thick cast-in-place plain concrete into which a 2.5 mm thick HDPE membrane anchored at the finished intrados.

Across the tunnel alignment the bedrock consists of weak sedimentary rocks with potential for karst features and comprises comprise interbedded layers of mudstone and gypsum and included sandstone, calcarenite, siltstone and claystone. The rocks ranged in strength from being very weak to moderately strong with the massive gypsum layers being the hardest and strongest, having unconfined compressive strength, UCS(σ_{ci}) = 12 MPa on average, with geological strength index, GSI = 60–70. Superficial deposits comprise marine sands and silts in the coastal zones and some superficial Sabkha deposits. The groundwater is close to the ground surface level and is hyper saline. The temperature in the city ranges from about 10°C to 48°C and the average annual precipitation is 89 mm.

TBM2 for Drive 2 was launched 84 days after TBM1 started up for Drive 1, and the third machine (TBM3) broke into the ground 19 days after TBM2. All three machines have the same diameter, specifications and the made. The TBMs were driven with three separate crews. Drive 3 was excavated underneath Al Wathba Wetland Reserve.

Generally, when a TBM is first launched there exists the potential for damage to the segments. This occurs primarily because the operators are unfamiliar with the equipment and the segments. This was the case in Contract T-02, with damage occurring to different degrees in all three tunnel drives. Following a learning curve, the situation improved in all three TBM drives.

Majority of the defects recorded during lining construction were accompanied with groundwater infiltrations. Cracking and infiltrations accelerate deterioration of the lining which have an impact on structural and service ability performance of the tunnel. Therefore, an extensive hard work had to be undertaken for identification, evaluation, recording of defects, and in-tunnel repair of the damaged segmentsand for sealing ground water infiltrations.

2 CONTRACT REQUIREMENTS FOR DURABILITY AND WATERTIGHTNESS

The durability design objective of STEP requires that the lining segments have adequate structural capacity throughout the 80 year service life and in addition must be watertight. The contract specification set the leakage rate to 1 liter/m^2/day for segment (and ring) joints. Any leakage through segmental (primary) lining prior to installation of corrosion protection (secondary) lining (CPL) had to be sealed as per the contract requirement.

The Abu Dhabi's STEP Project is located in an environment with high concentrations of chlorides and sulphates present in soil and groundwater. The results of groundwater and soil tests indicated that the chloride concentration in groundwater is 90,000 mg/l (9%), the sulphate concentration is 5,000 mg/l, the pH value of the tested soil samples range from 7.85 to 9.35, the groundwater temperature is approximately 25°C, and 80% humidity in the tunnel and shafts was considered. These values conclude that an exposure class S3 is designated by the "Guide to the Design of Concrete Structures in the Arabian Peninsula" [1], which is comparable to exposure class XA3 (Highly aggressive chemical environment) according to BS/EN206-1.

This extreme environmental exposure, combined with the required service life of 80 years, sets demanding requirements to design and construction as well as the defect repair works. The deterioration mechanism is governed by aggressive substances from the surrounding environment penetrate into the concrete hence corrosion of steel is initiated with time. Therefore the main focus of repair strategy was to seal water ingress through the defects and to protect conventional carbon steel reinforcement at the edges of the segments to satisfy the durability as well as watertightness requirements.

3 CLASSIFICATION OF CRACKS AND TUNNEL LEAKAGE

Cracking, wedging, spalling, chipping, stepping and lipping were the common defects types experienced in the tunnel. The focus of this paper is the cracks and segment (and ring) joints through which the majority of the groundwater infiltrations were recorded. The cracks have been categorized in four groups as shown in Table 1.

The damage that occurred to the segmental lining was of concern in three main aspects which were also interrelated:

- structural integrity of the lining as a system (usually short term)
- watertightness (short-term and long-term)
- durability (long-term)

The majority of the cracks occurred to be of Type 1 and 2. Any crack, with any of these potential impacts specified, was repaired. Small cracks outside

Table 1. Crack Types.

Type	Description of the Cracks
1	Cracks (outside reinforcement zone) with widths, $w_c^* \leq 0.3$ mm
2	Cracks (outside reinforcement zone) with widths, 0.3 mm $< w_c \leq 3$ mm
3	Cracks (outside reinforcement zone) with widths, $w_c \geq 3$ mm
4	Cracks inside reinforcement zone regardless of the width

*Measured crack width in millimeters

Table 2. Description of Groundwater Infiltrations.

Class	Description of the Leakage
D	Damp – Minor leakages; discoloration of the lining surface, moist to touch, damp patches.
F	Flowing – Major leakages; Continuous dripping or stream of water.

the reinforcement zone (Type 1 & 2) have no structural or durability implications. Cracks inside the reinforcement zone (Type 4) affect the durability. The larger cracks (Type 3) were considered on a case by case basis for the structural design, but in the majority of cases had no structural implications. Leakage through segment joints and cracks (regardless of the width) were checked against the contract requirements for water tightness, and treated accordingly. In all inspections, dry and wet (leaking) cracks were recorded and mapped separately regardless of the widths. Table 2 shows common descriptions of tunnel leakages used for identification and mapping.

4 MAJOR CAUSES OF INFILTRATIONS AND LEAKAGE CONTROL STRATEGY

The leakage occurred due to two main causes; defects, and ineffective primary grouting of the tail annulus void. The latter usually caused larger amounts of leakage problems compared to the former. The majority of infiltrations varied from damp patches to minor dripping occurred through the cracks. The cases with notable dripping, streaming or with measurable inflow were usually located at the segment (and ring) joints.

The mutual interaction between cracks, infiltrations and response to repair works is a complex phenomenon, and it is hard to monitor precisely. The inter-relation among potential causes, path of infiltration, and corresponding rectification works in relation to the inspection phases are illustrated in Figure 1. For simplicity, the entire process is idealized by dividing into two major components; repair of cracks (either by filling or injection), and sealing of segment joints by suitable grouting.

The repair of cracks was generally made in wet conditions due to presence of water. In principle,

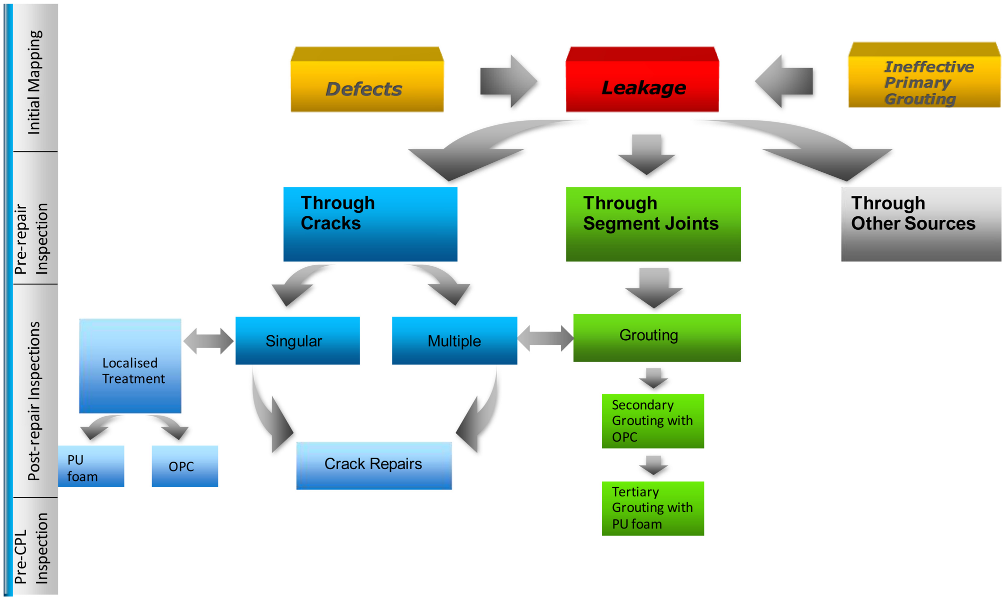


Figure 1. Simplified flow path for leakage control strategy.

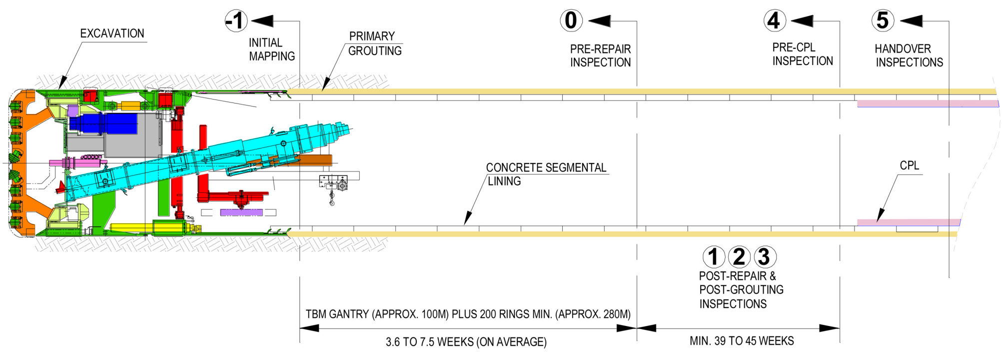


Figure 2. Inspection phases illustrated with approximate locations (Sketch not to scale).

for singular cracks, in addition to crack filling or injection with suitable materials and methods, PU (Polyurethane) foam, and occasionally OPC grouting had to be used to temporarily control damp patches and divert minor drippings (Class "D") off the repair area. This localized treatment was done in order to achieve an effective crack repair.

A secondary grouting campaign with ordinary Portland cement (OPC) and water mix was carried out behind the rings in case;

a) having identified potential areas where the grout take was below the theoretical anticipated amount based on the TBM-generated grouting records,
b) there was major inflow (Class "F") running through the segment joints
c) there was continuous dripping through multiple cracks over consecutive rings

For those minor infiltrations that continued after the secondary grouting, a tertiary grouting campaign was carried out to with the PU foam to seal leaking segment joints prior to CPL installation.

If the leaks through cracks or segment joints are distantly spaced then each leak was treated locally and case by case. This generally resulted inuse of larger amountsof PU foam to isolate the repair area from the water ingress. After that, either crack injection/filling with a suitable material or void grouting with OPC groutmix behind the rings, or both, were implemented as necessary.

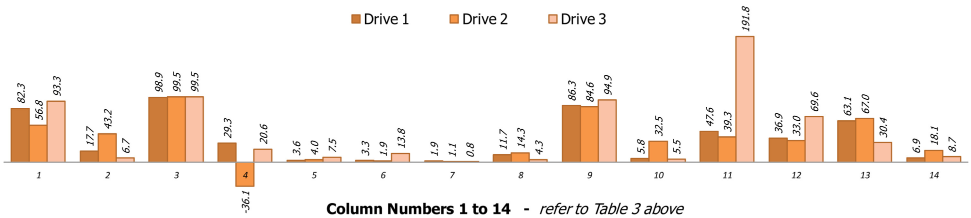
5 INSPECTION PLAN

The inspections held cover the initial mapping, before and after-repair observations, and a final check for infiltrations prior to CPL installation. The phases are illustrated in a schematic manner in Figure 2.

An "*initial mapping*" over 10 rings (i.e. shift production) was undertaken by the tunnel field engineer

Table 3. Results of analysis of crack and leakage records for identification and rectification

	IDENTIFICATION (PRE-REPAIR INSPECTION)					RECTIFICATION								POTENTIAL SELF-HEALING
						CRACKS - FILLING & INJECTION					SEGMENT JOINTS - GROUTING			
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*	*14*
	Percentage of cracks required repair	Percentage of cracks that did not require repair	Ratio of wet cracks to total no of cracks which required repair	Percent change in no of wet cracks	Time lag	Percent change in anticipated repair work for wet cracks	Percent structural repairs	Percent durability repairs	Percent water-tightness repairs	Cracks potentially sealed by SG (Type 1-4)	Percent change in anticipated grouting works (SG+TG)	SG/(SG+TG)	TG/(SG+TG)	Percentage of potentially self-healed cracks (Type 1 only) wc < 0.3mm
	Type 1-4 (wet + dry)	All dry, minor and outside reinf. zone		From initial mapping to pre-repair inspection						SG: secondary grouting		SG: secondary grouting	TG: tertiary grouting	
	[%]	[%]	[%]	[%]	[weeks]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]
Drive 1 *(3751 rings)*	82.3	17.7	98.9	29.3	3.6	3.3	1.9	11.7	86.3	5.8	47.6	36.9	63.1	6.9
Drive 2 *(3685 rings)*	56.8	43.2	99.5	-36.1	4.0	1.9	1.1	14.3	84.6	32.5	39.3	33.0	67.0	18.1
Drive 3 *(3436 rings)*	93.3	6.7	99.5	20.6	7.5	13.8	0.8	4.3	94.9	5.5	191.8	69.6	30.4	8.7



who recorded and categorized any defects during the ring build. After initial mapping, once the TBM gantry passed and cleared a distance of at least 200 rings, a "*pre-repair inspection*" was jointly held by the contractor's quality consultant, the designer and the Engineer. In this inspection, initial mapping records were checked for completeness, also changes were recorded. These records formed the basis for defining the need for repairs and sealing infiltrations for that section of the tunnel lining.

The penetration of groundwater containing aggressive agents such as chlorides and sulfates through cracks in the primary lining has a significant impact on the rate of deterioration. Therefore repairs were made for the cracked segments to stop water ingress and to protect conventional carbon steel reinforcement at the edges of the segments to satisfy the durability as well as watertightness requirements.

Following the repair works completion for cracks and other defects, a series of "*post-repair inspections*" for every 200 rings on the segmental lining were made to see whether the identified repairs were satisfactory, recorded infiltrations were successfully treated, and whether there were any increase in number, width of cracks or in leakages. "*Post-grouting inspections*" (for secondary and tertiary grouting) were held every 1000 rings. Since early inspections do not make allowance for the possible recharge of the water table, a final inspection was held to assess whether any further defects and infiltrations developed since the pre-repair assessment. This final post-repair inspection was done immediately prior to starting the CPL works. The "*pre-CPL inspection*" was held for every CPL block pour length, i.e. 12 m of tunnel.

6 ANALYSIS OF CRACK AND LEAKAGE RECORDS

The analysis of actual data, obtained from the crack and leakage records for a total of 10,872 rings built, yielded to the results which were consolidated in Table 3. A certain number of cracks (Type 1–4) occurred in the concrete segments. These cracks were picked up and recorded at the initial mapping phase. Some of them were wet (damp or leaking) cracks, some were dry. The pre-repair inspection was commenced once the TBM cleared a distance of 200 rings 3.6 to 7.5 weeks on average after the initial mapping.

From the initial mapping to pre-repair inspections, one of the first notable changes was that the number of wet cracks increased by 29.3% in Drive 1 in 3.6 weeks, and by 20.6% in Drive 3 in 7.5 weeks (Column 4 and 5 in Table 3). The increase does not necessarily imply new cracks appeared or that some of the cracks underwent progressive opening. As the water pressure built up around the newly erected rings, some of the cracks which were initially dry started to visibly leak. In contrast to Drive 1 and 3, the wet cracks decreased by 36.1% in Drive 2 within 4 weeks. There were four main factors likely to influenced this:

- The TBM performed successful and effective primary grouting (confirmed by regular proof drillings through segments)
- Attempt of "proactive" secondary grouting with OPC-water mix to seal multiple cracks
- Some of the Type 1 and 2 cracks closed up permanently in compression after leaving the shield
- Self-healing of Type 1 cracks

As mentioned in Section 4 above, continuous dripping through multiple cracks over consecutive rings was treated by secondary grouting with OPC mix. The Drive 2 crew took a slightly different approach in leakage control, and attempted a "proactive" secondary grouting with OPC for multiple leaking cracks prior to the pre-repair inspection. This sealed and reduceda number of wet cracks. This grouting was intensively applied in the initial drive then considerably reduced and discontinued. The number of cracks through which leakage stopped after secondary grouting was recorded in all three drives.

It is evident according to these records that the number of cracks that was potentially sealed by secondary grouting in Drive 2 was a few times greater than the other two, i.e. 5.8% and 5.5% (col. 10).

In the pre-repair inspections, the need for repair for Type 1–4 cracks was evaluated. This included both wet and dry cracks. 56.8% of the cracks required repair in Drive 2. The ratio was higher (82.3%) for Drive 1 and the highest (93.3%) for Drive 3. In other words, the majority of the cracks occurred in Drive 1 and 3 had to be repaired whereas Drive 2 required minimal crack repair (col. 1). The observation records confirmed that almost all of the cracks (up to 99.5%) that required repair were in wet condition (meaning either Class "F" or "D"). The numbers indicate a good agreement on this ratio (col. 3).

Those minor cracks that located outside the conventional reinforcement zone of segments with either dry or past moist, or negligibly damp condition were accepted with no repair. Only 6.7% of the identified cracks fell in this category in Drive 3. In Drive 2, 43.2% of the cracks did not need any repair. In Drive 1, the ratio was 17.7% (col. 2).

The repair works started following the completion of pre-repair inspection. The actual number of leaking cracks repaired in Drive 1 and 2 remained reasonably close to what was anticipated in the pre-repair assessment (col. 6). However, the repairs in Drive 3 exceeded the anticipated work by 13.8% due primarily to special hydrogeological conditions and the redistribution of water pressure around the tunnel. This will be further discussed in Section 9 below.

Looking at the distribution with respect to potential impact of a crack, one can say that the majority of the repair work was made to satisfy watertightness requirements in all three drives. 86.3% of the repairs in Drive 1, 84.6% in Drive 2, and 94.9% of the repairs in Drive 3 were performed to reinstate the watertightness of the lining (col. 9). The ratio of actual repairs for durability to all crack repairs was 14.3% in Drive 2 and it did not exceed this figure for the other drives (col. 8). The structural repairs remained in the range from 0.8 to 1.9% (col. 7).

Besides the cracks, the segment (also ring joints) were observed to be the secondary source of groundwater leakage. The infiltrations through them were generally treated by secondary grouting with OPC-water mix, and then a tertiary grouting with PU foam was employed in the case infiltrations persisted.

Table 4. Description of repair and grouting activities with respect to inspection phases.

Inspection Phase	Repair/Grouting Activity	Description
0		Pre-repair inspection
0-1	Crack repairs (Type 1-4) and SG as required	
1		1st pass post-repair inspection
1-2	TG	
2		2nd pass post-repair inspection
2-3	TG for localized minor leaks	
3		3rd pass post-repair inspection
3-4	None required	
4		Pre-CPL inspection

SG: secondary grouting, TG: tertiary grouting

The share of secondary grouting in the total grouting works (i.e. combined secondary and tertiary) for segment joints varied between 33.0 and 36.9% for Drive 2 and 1. However, it almost doubled up to 69.6% in Drive 3 (col. 12). The distribution of tertiary grouting in the total grouting works could be quantified as 63.1% for Drive 1, 67.0 for Drive 2, and 30.4% for Drive 3 (col. 13). Hence the leakage through segment joints in Drive 1 and 2 required less secondary but more tertiary grouting, whereas the majority of the treatment was done by secondary grouting in Drive 3.

The amount of work to seal the leaking segment joints by grouting exceeded what was estimated based on initial mapping at the pre-repair observations. The number of grouted segment joints increased by 39.3% in Drive 2, and 47.6% in Drive 1 for combined secondary and tertiary grouting. The actual grouting work almost tripled in Drive 3 (col. 11).

7 RESPONSE OF INFILTRATIONS TO CRACK REPAIRS AND GROUTING

In the time interval between the pre-repair to pre-CPL inspections, 39 to 45 weeks elapsed as minimum in construction. All repair works, including the trials, took place within this time span. The repair and grouting works were regularly monitored through three passes of post-repair and post-grouting inspections (Table 4).

The apparent redistribution of water pressure and the leaks caused more cracks and segment joints to be sealed. Despite the unexpected increase, the repair and grouting works were completed efficiently. 87% of the leaking cracks and more than 79% of the leaking segment joints were sealed in the first pass (Figure 3). 95% of all leakage was stopped in the second pass. The third pass resulted in very minor infiltrations below 1%. Between the third and fourth pass, no repair or grouting work was required.

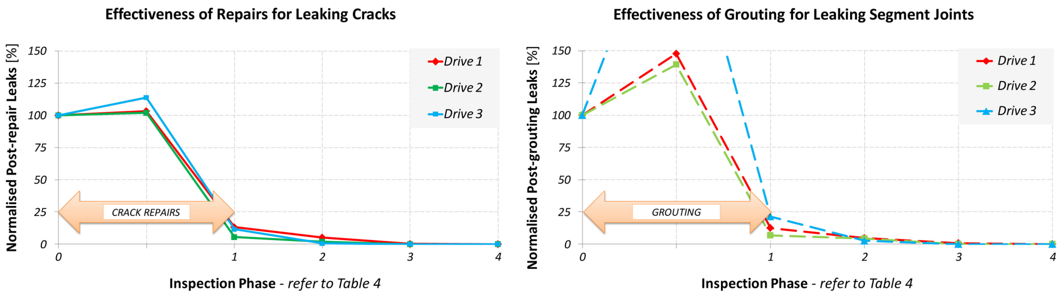


Figure 3. Post-rectification improvements on tunnel leakage.

8 POTENTIALLY SELF-HEALED CRACKS (TYPE 1)

Cracks in concrete up to 0.20 mm width have a high probability for self-healing in the presence of moisture and absence of tensile stresses. Possible causes of self-healing can be formation of calcium carbonate or calcium hydroxide, sedimentation of particles, continued hydration, and swelling of the cement matrix [2]. Due to autogenous healing, the water flow through the cracks gradually reduces with time, and in some cases, the cracks seal completely. Edvardsen (1999) found that 25 to 50% of dormant cracks with 0.20 mm wide (mean value) healed completely after 7 weeks of water exposure [3]. Both BS 8007 (The design of concrete structures for retaining aqueous liquids), and the UK's Water Services Association Specification imply that cracks up to 0.20 mm wide will autogenously seal within 28 days; cracks up to 0.10 mm will seal within 14 days [4].

The deep tunnels in consideration were not exposed to ambient temperature variations. No susceptible expansion, contraction movements or joint distress was noted in observations. Throughout a long observation period of up to 49 weeks, from the erection of segmental lining until the installation of the CPL, no opening and closing cracks as a result of seasonal temperature changes, or no progressive widening of cracks were determined. The crack types remained to be within the same limits as classified in Table 1. However, some of the water infiltrations through narrow cracks (Type 1, $w_c \leq 0.3$ mm) were found to be either significantly reduced or completely dried out in only a few weeks' time. These cracks did not open upagain. This recalls the natural process of self-healing mentioned above.

Column no. 14 of Table 3 shows the percent dried out Type 1 cracks from initial mapping to the pre-repair inspection. In other words, 6.9 and 8.7% of the leaking cracks dried out in Drive 1 and Drive 3, from the initial mapping to pre-repair inspection. This ratio increased to 18.1% in Drive 2. The authors believe that the main reason for this was that the majority of Type 1 cracks experienced in Drive 2 were very narrow (i.e. $w_c \leq 0.10$ mm). It should be noted particularly for Drive 2 that the cracks which were sealed by intensive grouting at the initial drive (col. 10) were not considered in number countsfor self-healing assessment.

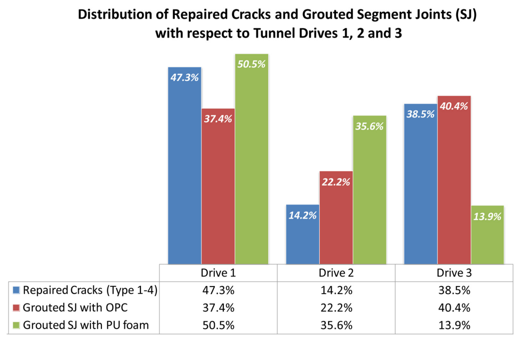


Figure 4. Distribution of actual crack repairs and grouting works with respect to tunnels.

9 DISCUSSION OF RESULTS

The distribution of all completed crack repairs and grouting works with respect to tunnels is summarized in Figure 4. Drive 2 had the least number of cracks and of major leaks hence delivered the best performance among the three. Due to the limited leakage observed, the least amount of the total OPC grouting took place in Drive 2 to seal the segment joints. However, the ratio of PU foam application realized to 35.6% being the second in order among the three. This is most likely because a number of minor leaks experienced through segment/ring joints distinctly so each of them had to be treated individually.

Drive 1 was the very first tunnelling operation in the programme, and the majority of the defects and leakages occurred within the initial drive during the learning curve. Drive 1 underwent the largest number of crack repairs, and that of PU foam grouted segment joints.

Drive 3 experienced the most intensive leakage through the defects due to the hydrogeological conditions. It was built under the "Al Wathba Wetland Reserve", through a non-intact zone in which fissures and fractures reported in boreholes were apparently interconnected and enabled full hydrostatic conditions to develop quickly in front of the TBM. The average face pressures in the second half of the drive length increased by five times and nearly reached to full hydrostatic pressure.Drive 3 took the largest share of OPC grout up to 40.4% to its segment joints, being 3% in excess of that of Drive 1. Major water inflow were experienced over more than half of the alignment length in Drive 3 although crack repairs were less than in Drive 1 by 9%.

Increasing need for secondary grouting (with OPC) was observed with increasing number of "major" leaks through the segment joints. Increasing need for tertiary grouting (with PU foam) was observed with increasing number of "minor and distinctly located" leaks through segment/ring joints. One may state for overall behavior of the leakage control that the use of OPC and PU foam grouting varied reversely in each tunnel.

An interesting observation common to all three tunnels was that the location of leaks tended to shift forward and backward as the crack repair/grouting works and the CPL installation progressed in the tunnel. In other words, a tendency of repeated leakage, mostly class "F" through the segment joins, was observed over a length of approximately $+/-1$ to $+/-6$ rings ahead and behind of a certain section in the tunnel where the repair works or the CPL construction progressed. This redistribution of water pressure along the tunnel axis complicated the repair works by having caused several more passes to seal the newly appeared or repeated leaks in order to provide the watertightness and hence comply the contract requirements. If one considers the CPL as an installation of an impermeable pipe into the tunnel as a secondary shell, then it can be imagined that this would cause a change in the hydrostatic pressure regime around the tunnel for a short-term. It may be interpreted that an "undrained" section is formed on the side of completed works whereas on the opposite side the tunnel can be considered as a "drained" section in which the repairs, grouting and the CPL works are still underway. The redistribution occurs at the interface of the two. A personal communication in May 2013 with Robert Marshall of CH2M HILL (formerly Programme Director to Singapore's Deep Tunnel Sewerage System (DTSS) Project) revealed that similar observations on redistribution of water leaks were reported on DTSS constructed between the years 1997 and 2006.

10 CONCLUSIONS

Three tunnel drives having a total length of 15.2 km have been completed successfully. The results of analyses on groundwater infiltrations based on actual records obtained from 10,872 rings have been discussed. The lining was made of hybrid SFRC/RC (steel fiber and conventionally reinforced) concrete segments. The leakage occurred due to two main causes; defects, and ineffective primary grouting of the tail annulus void. The interaction between cracks, infiltrations and response to repair/grouting works has been found to be a complex phenomenon, and it is hard to monitor precisely as it contains uncertainties by its nature. The anticipated amount of crack repair work increased by 1 to 13%, and that of grouting increased in the range from 39.3 to 47.6% for the first two drives but almost tripled in Drive 3.

The PU foam in supplementary or tertiary grouting can be effectively used to assist making minor defect repairs, however, care should be taken not to fill the cracks but to use suitable crack repair products. Although the use of OPC grouting performs well to seal some of the cracks, a careful assessment should be made to employ suitable and durable methods for the repair works.

Good practice and careful segment installation for a TBM-bored tunnel should obviously be the target to eliminate or minimize damage to the lining thus the leakage in the first place. Thorough analyses should be performed for the design of SFRC segmental tunnel linings under heavy temporary loads which should take into account the imperfections that may occur during ring erection particularly at the ring joints.

The sealing of tunnel leakage has long been one of the most serious problems confronting the builders and operators of tunnel systems. The repair of cracks and other damage to concrete linings could be a costly and time-consuming exercise depending on the watertightness requirements, function, operation and design life of the tunnel. The consequences of continued leakage may be far more serious for running mass transit tunnels in which heavy electrical-mechanical equipment are used.

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