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Comparative Analysis of Precast and *In situ* Concrete and Their Effectiveness on Koga Irrigation Structures

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Authors' contributions

This work was carried out in collaboration among all authors. All authors read and approved the final manuscript.

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ABSTRACT

Most construction projects in Ethiopia are built by using conventional cast in situ concrete. However, the latest technology reveals various priorities of the precast construction method over the cast in situ concrete. The main aim of this study is to analyze better irrigation lining concrete among two types of casting by conducting visual inspection methods and non-destructive tests to characterize defects. Koga irrigation main canal, which is found in Northwest Ethiopia, was used as a study area because it is made of both in situ and precast concrete linings. The study reveals that the lower strength, severe defects, and lower uniformity due to the high level of difficulty in pouring and

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vibrating the concrete on the side slopes of the canal were observed in the *in situ* lined canal. However, the growth of grass in the joints, and sealing of closely spaced and less water-tight joints are observed at the precast part of the canal lining. Providing reinforcement bars on the *in situ* canals and providing water-tight joints for the precast concrete part are viable solutions.

Keywords: Canal; In situ concrete; rebound hammer; precast; non-destructive test.

1. INTRODUCTION

Concrete is a composite material that has been used in almost all types of civil structures including buildings, bridges, hydraulic structures, roads, and many others. According to Agunwamba and Adagba [1], Concrete structures can be directly affected by dynamic events such as earthquakes, storms, flooding, ground movement, etc. with possibly catastrophic consequences. Since concrete structures are designed for a long useful service lifetime, they can gradually lose strength, whilst the quality and durability of such structures can deteriorate with time [2]. Concrete is a mixture of cement, aggregates, and water. According to Katuwal [3], concrete property depends on the shape, size, density, and soundness of aggregate.

Concrete compressive strength (CCS) is an important property because it is essential for designing a structural member or calculating its load-bearing capacity. The CCS does not have a constant value during the concrete life due to curing at an earlier age and internal cracks developed due to environmental factors at later ages [4]. Therefore, assessing the *in situ* concrete compressive strength is required in many situations.

According to Agunwamba and Adagba [1], Concrete is subjected to different environmental degradation factors which tend to shorten its service life. This has brought the need for different test methods to evaluate the in-place properties of concrete for quality evaluation and assurance of existing concrete conditions. Since concrete tests are expected not to upgrade the function of the structure and allow for testing at the same location to evaluate the changes in the property at some other point at a time, these methods must be non-destructive. Concrete deterioration has a substantial impact on structure performance and serviceability. Poor construction, overloading, ageing, corrosion, chemical reactions, and other factors can cause the degradation of concrete [5].

The operation and maintenance manual, [6] states that the main types of concrete used in the canal lining of the Koga Dam and Irrigation Project (KDIP) are *in situ* concrete and precast concrete. Both kinds can be unreinforced plain concrete, ordinary reinforced concrete using steel mesh or bars, or special kinds of reinforced concrete using other types of reinforcement. Most canal (MC) linings are class C20 unreinforced concrete, though C25 reinforced concrete is sometimes used for reinforced structures or in other places where large relative movements in the subgrade are likely. Koga irrigation canal is chosen as a study area for this project because it is made of 15.2 kilometer (km) *in situ* and 4.5 km of precast concrete linings.

Usually, some joints are constructed as expansion joints, with compressible filler 10 to 20 mm thick between adjacent panels to allow for subsequent expansion of the concrete. A concrete lining, especially if it is unreinforced, will usually crack sooner or later due to shrinkage, thermal stresses, loading, settlement, or another movement in the subgrade. After its shrinkage period, the concrete will normally have a slight opening of the joints, and if joints are too widely spaced it will have cracks as well. Precast concrete slabs are laid in a canal using a crane or, by handling individual slabs manually if they are small enough. Most of the concrete's shrinkage takes place before the slabs are placed, so provision is only needed for thermal expansion in the joints between them. Precast concrete linings have much closer joint spacing than cast *in situ* linings.

For the Main Canal and the secondary canals, the need to minimise seepage losses was recognised in the feasibility study, and a provision was made to line the canals with concrete. To come up with high levels of soil permeability in the project area, it is important to provide a more significant and effective lining system that can be depended upon to prevent or significantly reduce seepage losses throughout the lifetime of the project. If lined with concrete alone, the seepage losses would increase over a relatively short period of time until they reached rates perhaps 20% or 30% less than a completely unlined canal [6].

The feasibility study of KDIP reveals that the total water losses on Main Canal and secondary canals would exceed 50%. According to the infiltration test results, the seepage losses from the conveyance system without lining alone would exceed 50% with more than 4 m^3/sec lost through seepage before the delivery to the field channels. The feasibility study assumes that water losses in the conveyance system would be only 20%, equivalent to 1.7 m^3/sec (20% of 8.5 m^3/sec). This low seepage rate may be possible immediately after construction. However, after a few years, as the concrete lining deteriorates, the losses may reach the equivalent of more than 3m^3/sec within the delivery system [6]. For this reason, geo-membrane and geotextile lings are provided under the concrete linings of the Koga main canal to minimize the seepage through the canal.

There are 19.7 km main and 42.0 km secondary concrete-lined canals in the Koga irrigation scheme. A total of 16.41 hectares of plain concrete is used for the construction of the main canal lining. Considering the relatively large area of the Main Canal and secondary canals, the use of cast *in situ* concrete was preferred for most of these linings [7]. The concrete lining was formed as five meter panels, with alternative panels cast initially. The method of placing is in such a way that the concrete is vibrated to obtain a well-compacted finish by using a vibrator.

These structures are constructed 14 years ago (2008). In 2019, a 32 meter long crack was observed at the bed of the concrete lining of the main canal which was cast on-site due to the uplift force of the sub-surface water. The sub-surface drainage perforated pipes under the bed of the canal, which were designed to take the sub-surface water to cross drainage structures were blocked by fine-graded filter materials. Abay Basin Authority hired a contractor and reconstructed the canal by demolishing and removing the old canal concrete lining. But the crack was observed again after six months. During the reconstruction time, most of the damaged *in situ* part of the concrete canal lining is demolished and removed but not re-used. So that it is important to conduct a study that compares and identifies more sustainable and convenient types of concrete for lined canals.

In situ concrete linings are placed by hand. The usual method is to trim the subgrade approximately to the required shape, erect stop ends to divide the lining into panels and begin by placing the concrete in alternate panels only. According to Mott [7], a top shutter is not normally used and is not needed. The side slope for Irrigation canals is not steeper than about 1:1, the more common slope being 1.5 horizontal to 1.0 vertical.

As per the study [8], a precast construction system includes those constructions, where the structural components are standardized and manufactured in plants at a location away from the building. A precast concrete structure is made by assembling precast elements, when suitably assembled, make a 3D framework that can resist dynamic loads. The framework has an advantage for buildings, car parks, schools and other such buildings which require internal obstruction and multifunctional space [9]. The precast concrete industry use increases, and it is expected to replace the on-site vibrated concrete in many applications because of its advantages. The precast concrete technique has higher toughness, improved thermal properties, and better development speed.

Since the appearance of cast-in place concrete is directly dependent upon the quality of formwork, concrete placing operation must be continuous for each section of the work [10]. According to Chandlukar [9], most of the concrete for buildings is cast *in situ* in which the wet mix is produced at the place where the finished concrete is desired. There are different deterioration mechanisms or defects that cause the degradation of concrete performance.

A crack is an incomplete separation into one or more parts with or without space between and negatively affects the structural integrity of an element [11,12]. According to Committee [12], cracking of concrete can be reported based on crack widths and the type of crack. Cracks can be classified as fine, thin, medium, wide, and very wide and their width varies from 0.1 to 2.0 millimeter (mm) or greater. Continuous cracks with a lined direction and magnitude can be a sign of a structural deficiency. These types of cracks are usually longitudinal, transverse, diagonal, vertical, or spiral and will often continue to grow in depth, and the length of the structural defect [13]. The inadequate form supports, improper concrete construction practices and improper placement of construction joints contribute to cracks in concrete [11]. The settlement of forms causes cracks because the concrete has not hardened to support its weight. Construction joints placed at points of high stress can cause cracks.

Plastic shrinkage cracks are due to differential volume changes in plastic concrete. Rapid water loss during hardening is the primary cause of cracks of this type, consequently the concrete surface contracts. Because of the restraint, the tensile stresses develop in the weak point and stiffen plastic concrete [14].

Drying shrinkage occurs after the setting process when the concrete is hardened. This type of shrinkage becomes less effective as time goes longer in concrete. If the relative humidity of the environment is greater than the concrete, it dries out and water dissipates from the porous veins in the concrete. This lowers the pressure in the pores and the concrete shrinks [15].

Where patches or overlays exist, crack surveys of structures are difficult to perform and are likely to be unreliable. Cracks beneath these repairs may represent an obvious or partial failure at greater depth [16]. Surface mapping starts at one end of the structure and continues gradually throughout the concrete structure. On the other hand, Pattern cracking is formed on concrete surfaces repeatedly, resulting from shrink of the material near the surface, an increase in the volume (swell) of the material below the surface, or both [12]. Map cracks generally result from a decrease in the volume of the material near the surface or an increase in sub-surface material volume [17,13].

According to Stowe and Thornton [11], Crazing is the result of excessive bleeding of concrete, premature trowelling finish, and high water content on the surface of the concrete. Crazing is characterized by closely spaced fine cracks in the surface, it is primarily a non-structural defect in cement paste, mortar, or concrete [12].

Discolouration concrete is a change in colour due to improper concrete mix specifications; agents or aggregates of differing alkalinity in the mix may cause colour changes in the concrete material [13]. Including aggregates in the mix that are reactive with the cement can cause a deposit of salts to form on the surface as bleed water rises to the surface and evaporates, excessive amounts of water in the mixed compound are the condition [11].

Inadequate provision for structural movement can cause pop-outs. Internal stresses caused by corrosion of reinforcement, cement aggregate reactions, or internal ice crystal formations can cause pop-outs [11]. Localized internal pressure inside concrete leaves a shallow, depression with a broken coarse aggregate at the bottom causing the breaking of small portions of a concrete surface [12].

Continuous abrasion causes scouring of material, exposing the reinforcing bars, or reducing the mass enough to critically affect the surface of the structure. Abrasion-erosion defect may have resulted from the abrasive effects of waterborne gravel, rocks, and other debris are being circulated over a concrete surface, which is different from the holes and pits formed by cavitation erosion. Spillway aprons and stilling basins are mainly sensitive to abrasion [11]. Scaling is the peeling away of the surface of concrete or mortar caused by freezing and thawing [11]. The loss of structural material can lead to inadequate load-bearing capacity or exposure of the reinforcing steel to corrosive forces [12].

Spalling is depression due to fragments, having a flake shape, removed from a concrete surface by weathering, pressure, or expansion within the larger mass [12]. Active spalling occurs because of changes in internal temperatures, corrosion of reinforcing steel, chemical reactions, and freezing and thawing, and can continue to spread. Passive spalling can be repaired, but active spalling indicates potentially dangerous structural problems.

Stratification is the separation of concrete materials into their parts during the placement of concrete as the result of high water content. The heavier concrete ingredients settle to the bottom portion of the member, forcing finer particles to the top. Uneven distribution of concrete materials or layers that have not properly bonded can alter the structural performance characteristics of the concrete which are aggravated by moisture penetration, cracking, severe efflorescence, or rusting occurring along with stratification [13].

The non-destructive test (NDT) is a test used for examining the integrity of a material, or structure or quantitatively measuring some characteristic of the material without destroying the testing member. NDT techniques can save a lot of time, effort, and cost [18]. NDT is important in ensuring the economical operation, safety, and reliability of the component [19].

The rebound hammer (RH) is a surface hardness tester for which a mathematical correlation has been established between concrete strength and rebound number. The RH method is rough estimation of strength based on rebound depends on the surface hardness against which the mass strikes. It is known and repeated in all the Schmidt hammer manuals that the concrete surface has to be smoothed before performing the test & is enough to make the surface plane, but it is not enough separate one of the features of concrete structures [20].

A pull-out test is conducted by using a ram, the force required to pull from the concrete, or a specially shaped steel rod whose elongated end has to be cast into the concrete to a depth of 7.6 centimeter (cm). The concrete is turned by turn and subjected to tension and shear, but the force required to pull the concrete out may be related to the compressive strength of the concrete. According to Committee [12], a pulse velocity method is a tool for establishing the uniformity of concrete. It can be applied to both existing structures and those under construction structures. High pulse velocity readings imply good quality concrete [21]. The ultrasonic test is conducted in transmission, reflection, and backscattering modes each of these uses a range of frequencies [22].

This study is done by comparing choices among *in situ* and precast forms of concrete for canal lining. The main aim of the study is to make a comparative quality analysis of cast *in situ* and precast concrete components of irrigation canals and to recommend which form of concrete is better for the construction of canals. Specifically, to evaluate the status of the Koga Irrigation concrete canal structures by visual inspection and to identify defects as per the ACI standards. Assessment of the current in-place compressive strength and uniformity of the *in situ* and precast concrete canals of the Koga Irrigation scheme by using a rebound hammer test is also part of this study. Finally, technical, and practical recommendations are forwarded based on maintainability and choice of cast *in situ* and precast linings, and their defects.

2. MATERIALS AND METHODS

2.1 Introduction

The Experimental analysis of this study is the comparison of the cast *in situ* and precast concrete according to their CCS and uniformity by using digital RH. This research involves both quantitative and qualitative approaches. The main sources of quantitative data are both primary data or direct field measurements and secondary data. The visual inspection was conducted based on the significance of defect types, especially on cracks, spalls, scaling, pop-out differential movements, removal of joints, and vegetation growth on joints and cracks. But the field rebound hammer test was conducted on six different features of concrete structures. For every six samples, 10 rebound hammer test trials were conducted at three separate places for each test. The rebound test was conducted at 180 test points.

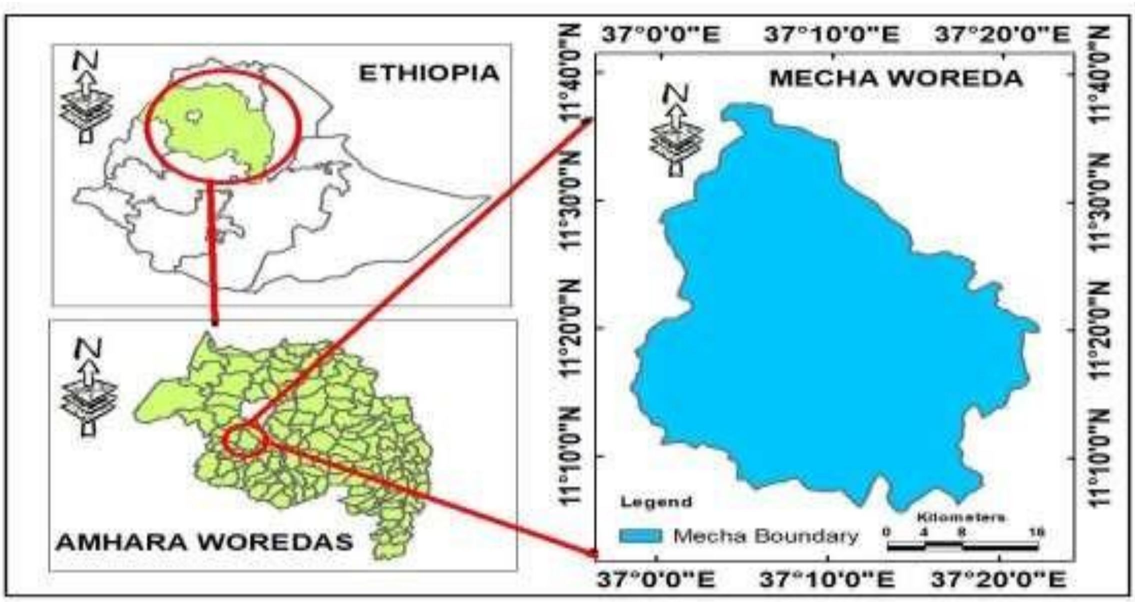

Fig. 1. Study area

The average and standard deviation of compressive strength results are computed and used for comparison. The study area is located in Amhara Region, in the West Gojjam Zone of Merawi Town which is 7200 meters far away from the asphalt road connecting Bahir Dar and Addis Ababa.

2.2 Data Collection Methods and Techniques

Data were collected from primary and secondary sources. These include observations, field non-destructive tests, and office and field document reviews. The field non-destructive tests of concrete were conducted to compare the strength and durability of the precast and cast *in situ* construction techniques. These are visual inspection and hammer rebound tests. These tests were conducted randomly at different points having different features.

2.2.1 Visual inspection techniques

Detailed visual inspection techniques including Global Positioning System (GPS) locations of defects were applied to observe cracks, surface spalling, surface scouring, surface pitting, differential movements, contraction, and expansion joints, and to observe algal and vegetation growth on the structure.

2.2.2 Rebound hammer test

Among the other NDT tests, the rebound hammer test was conducted due to its availability in the Bahir Dar University laboratory. To get a precise result, several experimental data on CCS, and rebound numbers from rebound hammer tests are required. The rebound test apparatus which was used for this experiment is digital and it can directly display the CCS in MPa. The experimental data were collected from 6 points for evaluating the hardened compressive strength of concrete was tested from a concrete constructed 14 years ago and the compressive strength of concrete is taken as the output. The rebound value can be calculated using innovative technology by considering the anvil speed before and after the loading. This working principle is similar to the traditional measurement of maximum spring length after the impact, but it provides higher accuracy and stability of the readings.

The setting of the impact angle is no longer required and checking of the device's reliability can be performed during each impact, even without the calibration anvil. Digital Rebound Hammer Model 58-C0181/DGT with Accessories is used for the study as shown in Fig. 2.

The test result is different for the same concrete depending on the uniformity of the concrete. A smooth, clean, and dry surface, the loosely adhering scale was selected. The point of impact is taken at least 20mm away from the edge or shape discontinuity.

2.2.3 Tests conducted by rebound hammer for comparison

Rebound tests were applied for comparison on the following three different forms of concrete.

a) 10cm thick *in situ* concrete versus 10cm thick precast mass concrete canal linings
b) 5cm thick *in situ* concrete versus 5cm thick precast mass concrete canal linings
c) 30 cm thick *in situ* reinforced concrete versus 30 cm thick precast reinforced concrete drop structures

Fig. 2. Digital rebound hammer device

Table 1. Methodology summary

No.	Objective	Data Required	Data Collection	Analysis
1	To evaluate the current condition of the concrete structure by visual inspection techniques.	Crack length, crack depth, type of pitting, scaling pullout, spalling, and other visual observations and location of visual inspection points	Visual inspection.	Evaluation of the condition of the structure based on the collected data as specified by the standards
2	To assess the in-place compressive strength of concrete.	10 readings of Rebound tests are noted from 6 different structures. IS CODE 13311(2) and IS 516 Part 5	readings from each sample site are taken using RH as specified by the code ACI 201.1R-08	Compressive strength was obtained from the average of 10 rebound results.
3	To assess the uniformity of concrete	10 readings of Rebound tests are noted from 6 different structures	Readings from each sample site are taken using RH	The measure of the uniformity of compressive strength from the standard deviation of rebound results
4	Forwarding technical and practical recommendations	Project documents, Experimental data, research reviews, and past experiences	Data was collected through different office and field investigations	The technical and practical recommendation is set to the consultants, contractors, and client based on the out coming of the research

2.2.4 Procedures used for the rebound hammer test

The site was visited to select experimental points and a field recording format was prepared. The selection of test points where readings were taken was marked. 30cm by 30cm surface area was prepared by cleaning and marking the area for readings at 10 points. To carry out the test, the digital hammer was held perpendicular to the face concrete, then it was pressed until a hammering sound was heard. After hearing the hammering sound, its lever was pressed, and the digital rebound hammer was released.

3. RESULTS AND DISCUSSION

3.1 Visual Inspection and Characterization of Defects

3.1.1 Crack

As per [12], the cracking of concrete should be reported based on crack widths and the type of crack. In this study, it was very difficult to record the fine and thin categories of cracks in Table 2 also, only some samples of the wide and the very wide categories were recorded. Because the wide and very wide category of cracks was also continuously observed even without termination at compressible filler joints. For this reason, the cracks were recorded for comparison purposes as meters per km. The total cumulative length of cracks for the first km was 2144m/km. but the very wide, wide, and thin category of cracks was not observed in the precast part of the concrete canal.

3.1.2 Pop-outs

Pop-outs are observed only at a few places of both precast and *in situ* concrete canals. The recorded pop-outs were observed more at the sides and the corners of the precast elements. Because care should be taken during the loading and unloading of precast elements. The breaking away of small portions of a concrete surface leaves a shallow, typically conical, depression with a broken coarse aggregate at the bottom [12].

3.1.3 Scaling

From the visual inspection results, it was observed that all part of the lined canal under the freeboard is subjected to light scaling. So, the light defect category of scaling is generalized throughout the longitudinal section of the canal lining below the freeboard. But the severe loss of mortar and more exposure of coarse aggregate is observed more at the *in situ* linings.

3.1.4 Spalling

Spalling is observed on both precast and *in situ* concrete parts of the canal. But the spalling at the precast concrete is more than the *in situ* ones. Because the precast concrete elements are subjected to joint spalls. According to Stowe and Thornton [11], Joint spalls occur along joints in concrete members.

3.1.5 Other visual observations

a. Differential movements on precast concrete canals

In most precast concrete canals, simple differential settlements were observed at the bed of the canal as shown in Fig. 4. But the *in situ* concrete passes the settlements as a slab.

Table 2. Visual inspections of cracks

No.	Location		Concrete type	Structural defect		Crack Dimension, mm	
				Crack		Length	Width
	North	East		Wide	Very Wide		
1	11^{0}20.766’	037^{0}08.476’	Insitu		✓	500	2.0
2	11^{0}20.766’	037^{0}08.472’	Insitu	✓		500	1.0
3	11^{0}20.771’	037^{0}08.468’	Insitu	✓		400	1.0
4	11^{0}20.796’	037^{0}08.386’	Insitu		✓	600	2.0
5	11^{0}20.798’	037^{0}08.383’	Insitu		✓	1000	3.0
6	11^{0}20.808’	037^{0}08.830’	Insitu		✓	700	3.0
7	11^{0}20.803’	037^{0}08.376’	Insitu		✓	1000	2.0
8	11^{0}20.810’	037^{0}08.368’	Insitu	✓		800	1.2
9	11^{0}20.814’	037^{0}08.364’	Insitu		✓	650	3.0
10	11^{0}20.818’	037^{0}08.360’	Insitu		✓	700	3.0
11	11^{0}20.827’	037^{0}08.350’	Insitu	✓		600	1.0
12	11^{0}20.828’	037^{0}08.348’	Insitu		✓	500	2.0

Table 3. Visual inspection of pop-outs

No.	Location		Concrete type	Structural defect			Pop-out Diameter (mm)
				Pop-out Category of pop-out			
	North	East		Small 0 to 10mm	Medium 10-50mm	Large >50mm	
1	11^{0}20.996’	037^{0}08.066’	Insitu		✓		19
2	11^{0}20.995’	037^{0}08.052’	Insitu		✓		15
3	11^{0}20.994’	037^{0}08.044’	Precast		✓		30
4	11^{0}20.967’	037^{0}08.338’	Precast		✓		30
5	11^{0}20.984’	037^{0}08.023’	Precast		✓		25
6	11^{0}20.982’	037^{0}08.008’	Precast		✓		20
7	11^{0}20.981’	037^{0}08.002’	Precast		✓		50
8	11^{0}20.981’	037^{0}08.997’	Precast		✓		25
9	11^{0}20.985’	037^{0}08.984’	Precast		✓	✓	60
10	11^{0}20.980’	037^{0}08.976’	Precast		✓		32
11	11^{0}20.984’	037^{0}08.973’	Precast		✓		40
12	11^{0}20.972’	037^{0}08.958’	Precast		✓		30

Table 4. Visual inspection of scaling

No.	Location		Concrete Type	Structural defect				
				Scaling				Depth (mm)
				Category of scaling				
	North	East		Light	Medium	Severe	Very severe	
1	11^{0}20.768’	037^{0}08.477’	Insitu	All parts of the canal under the freeboard		✓		12
2	11^{0}20.771’	037^{0}08.464’	Insitu			✓		12
3	11^{0}20.829’	037^{0}08.364’	Insitu			✓		12
4	11^{0}20.830’	037^{0}08.347’	Insitu			✓		15
5	11^{0}20.81.3’	037^{0}07.562’	Insitu			✓		15

Fig. 3. Observation of concrete face above and below the free board

Fig. 4. Differential movements on the precast concrete

Table 5. Visual inspection of spalling

No.	Concrete type	Location		Category of spalling (mm)				Spalling Dimension (mm)	
				Small		Large		Depth	Any dimension
		North	East	Depth <20	Any dimension <150	Depth >20	Any dimension >150		
1	Insitu	$11^0$20.800’	$037^0$08.379’			✓	✓	20	250
2	Insitu	$11^0$20.807’	$037^0$08.371’			✓	✓	30	160
3	Precast	$11^0$20.858’	$037^0$08.329’			✓	✓	30	150
4	Precast	$11^0$20.862’	$037^0$08.327’			✓	✓	30	160
5	Precast	$11^0$20.924’	$037^0$08.298’			✓	✓	20	300
6	Precast	$11^0$20.936’	$037^0$08.282’	✓	✓			15	140
7	Precast	$11^0$20.838’	$037^0$07.571’	✓	✓			10	100

Fig. 5. Effect of rainfall on the *in situ* concrete

3.1.6 The effect of rainfall before setting of concrete on *in situ* sections

One of the main problems of *in situ* concrete construction is that, when the rain falls on concrete before its setting time in the summer season, the jell of the concrete mix will be washed away by the rainfall. But the precast elements were assembled after setting and curing time was finished in a controlled and protected room.

3.1.7 Vegetation and Grass Growth on Joints of precast concrete elements

As shown in Fig. 6, the precast concrete part of the canal is sensitive to the growth of vegetation and grass at the closely spaced joints due to the accumulation of dust inside the openings.

3.1.8 Vegetation and grass growth and cracks *in situ* sections

It was observed that Vegetation and grass have grown on the wider cracks of *in situ* concrete canal linings.

3.1.9 Removal of joints on the precast elements by high pressure of water

Some of the mortar-filled joints, which are the weak points of precast concrete canals, were removed by the scouring action of water (Fig. 8).

Fig. 6. Vegetation and grass growth on the joints of precast concrete

Fig. 7. Vegetation and grass growth in the cracks of *in situ* concrete

Fig. 8. Removal of joints between the joints of precast concrete

3.2 Assessment of Compressive Strength and Uniformity by Rebound Hammer

3.2.1 Number of test points of rebound hammer

According to ACI 228.1R, to establish a relationship between rebound number and concrete strength, a minimum of 2 replicates, from 6 or more locations with different rebound results should be taken. According to the ASTM C805 standard, test locations should be selected such that a wide range of rebound numbers in the structure is obtained. The strong relationship will be applicable for the same orientation as used. If the rebound number is affected by the orientation of the instrument during testing, a strong relationship is applicable for the same orientation as used to obtain the correlation date.

According to the results of the rebound test, the compressive strength values are very much increased when compared with the design characteristic strength values. This situation is observed on both precast and *in situ* concrete elements. But, based on a comparison of the average and standard deviation of CCS results, the strength and uniformity of the compressive strength of precast concrete are much better than the *in situ* concrete. The rebound hammer test results, which are conducted by making grid squares at six places and 10 points at each place and their corresponding locations, are listed in Table 6.

3.2.2 Summary of rebound test results

The compressive strength results of the rebound hammer test for different thicknesses of plain and reinforced concrete canal structures are summarized in Table 6. Generally, the summary of the rebound test results from Fig. 10 shows that; the average compressive strength of the precast concrete part of the canal is much greater than that of the *in situ* concrete and the standard deviation of the compressive strength of the precast concrete is less than the *in situ* types of concrete. This is due to the high quality control of precast concrete in factory than cast *in situ* concrete which is done with low quality control in site compared to precast. The standard deviation of precast is also lower due to high quality control during production compared to cast *in situ* concrete.

Fig. 9. Conducting rebound hammer test

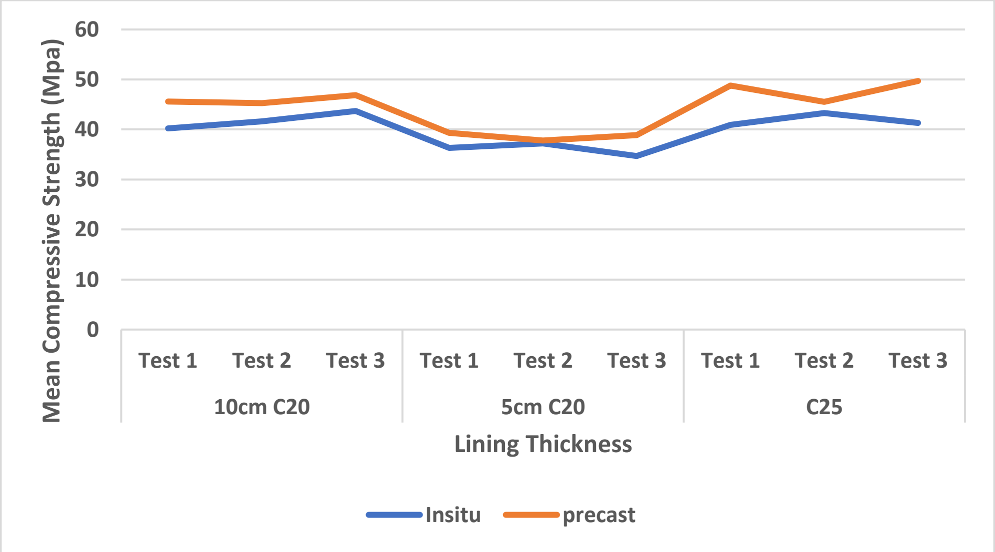

Fig. 10. *In situ* versus precast canal lining rebound test (Compressive Strength)

Table 6. rebound hammer test results

	CCS in MPa		Location		1	2	3	4	5	6	7	8	9	10	Average	Mean of Average	Standard Deviation
10cm C20	Insitu	Test 1	N11^{0}20’77.2’	E037^{0}17.638’	42.0	48.8	33.6	27.2	42.3	37.8	47.6	40.9	40.5	41.5	40.2	41.8	6.0
		Test 2	N11^{0}20.827’	E037^{0}08.351’	40.1	52.1	41.4	34.2	38.1	44.1	40.5	45.5	38.1	41.7	41.6		4.6
		Test 3	N11^{0}20.941’	E037^{0}08.282’	43.9	47.8	49.7	38.2	45.2	48.7	38.2	44.3	34.2	46.6	43.7		4.9
	Precast	Test 1	N11^{0}20’948’	E037^{0}08.240’	43.3	40.5	43.1	42.8	47.1	48.4	48.3	48.0	47.9	46.2	45.6	45.9	2.7
		Test 2	N11^{0}20.992’	E037^{0}08.037’	44.9	45.0	44.0	43.2	46.1	46.2	47.8	46.4	44.0	45.5	45.3		1.3
		Test 3	N11^{0}20.941’	E037^{0}17.282’	45.0	51.6	51.7	46.4	48.4	48.1	43.7	42.6	45.7	45.5	46.9		2.9
5cm C20	Insitu	Test 1	N11^{0}25.842’	E037^{0}07.388’	31.1	37.5	48.2	28.9	32.6	31.3	29.3	36.5	32.5	55.4	36.3	36.1	8.3
		Test 2	N11^{0}25.844’	E037^{0}07.388’	39.7	36.3	31.0	31.4	42.6	28.7	47.7	49.7	33.2	31.2	37.2		7.1
		Test 3	N11^{0}25.864’	E037^{0}07.383’	29.3	32.3	44.8	34.0	42.0	25.5	44.0	36.1	28.6	30.2	34.7		6.5
	Precast	Test 1	N11^{0}25.353’	E037^{0}07.467’	38.3	43.8	38.9	39.5	37.8	39.1	38.6	37.9	39.7	39.3	39.3	38.7	1.6
		Test 2	N11^{0}21.516’	E037^{0}07.128’	39.1	39.5	37.3	37.7	37.8	36.9	36.2	38.5	39.4	35.4	37.8		1.3
		Test 3	N11^{0}25.844’	E037^{0}07.386’	38.2	37.4	37.3	40.1	38.0	37.6	41.8	38.8	40.1	39.3	38.9		1.4
C25	Insitu	Test 1	N11^{0}25.959’	E037^{0}07.375’	34.2	32.4	48.3	33.9	48.1	44.4	49.2	41.0	36.6	41.2	40.9	41.8	6.1
		Test 2	N11^{0}25.926’	E037^{0}07.381’	48.3	38.2	41.2	45.3	37.1	45.7	36.5	44.4	40.1	56.2	43.3		5.7
		Test 3	N11^{0}25.889’	E037^{0}07.387’	53.2	37.5	43.1	39.1	34.2	38.2	34.2	39.1	51.0	43.5	41.3		6.2
	Precast	Test 1	N11^{0}25.929’	E037^{0}07.364’	47.0	51.5	49.4	50.2	45.5	47.5	45.5	47.1	55.9	48.8	48.8	48.0	3.0
		Test 2	N11^{0}25.924’	E037^{0}07.362’	48.1	45.8	49.8	44.0	42.5	46.1	43.5	45.0	45.8	44.1	45.5		2.1
		Test 3	N11^{0}25.951’	E037^{0}07.357’	51.1	52.5	48.4	49.1	52.2	49.2	51.3	50.9	49.7	42.3	49.7		2.8

Table 7. Summary of comparison of *in situ* and precast concrete

Objective	Cast-*in situ* Concrete Lining	Precast Concrete Lining
Visual Inspection and Characterization of defects	Very severe cracks, which are characterized as extremely long, wide, and very wide in dimension, are observed on the *in situ*	Cracks are fine, thin, and very small in dimension
	Fewer pop-outs are characterized as medium pop-outs	More Pop-outs are medium pop-outs. A large pop-out is observed at one point
	Sever Scaling is observed in addition to light and medium scaling	Only light scaling is observed
	Large spalls	Small and large spalls
	Grass and vegetation grow on the cracks	Grass growth on the joints
	Less and compressible filler joints are available	Very closely spaced joints that are sensitive to defects are observed
Strength	Less Compressive Strength compared to precast	More Compressive Strength
Concrete Uniformity	Less Uniformity	More Uniformity Compared with the *in situ*
	Other considerations	
Suitability to pour and vibrate	Very difficult, because the concrete will either slide down or the coarse Aggregates will segregate	The precast concretes are brought to the site only for assembly
Maintainability	Not suitable for satisfactory maintenance	The precast concrete elements can be either reused or simply changed by other similar precast concrete
Durability	Less durable than the precast concrete lining	Precast Concrete has a longer service period and minimum maintenance
Safe working platform	Not safe	Precast elements can be stocked so it reduces the necessity of traditional formworks and props, wastage

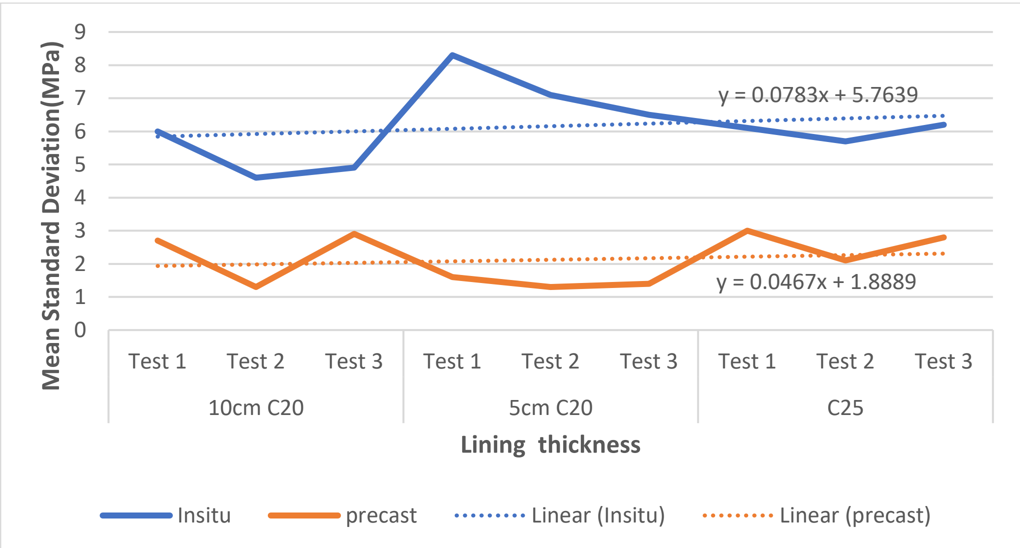

Fig. 11. Uniformity of the *in situ* versus precast canal lining

Precast uniformity is higher when compared with *in situ* concrete. As shown in Fig. 11, the trend line of *in situ* concrete is steeper than precast concrete. The standard deviation of the *in situ* is higher than precast concrete lining in all thickness levels as stipulated in Table 6; implying that precast has higher uniformity.

4. CONCLUSION

Long very severe cracks and wider in dimension, are observed on the *insitu* plain concrete lining. The main cause of concrete canal lining cracks is structural overload from foundation support due to settlement, expansion, internal erosion, or excess hydrostatic load. This is because of concrete is relatively weak in tension, and concrete canal linings are not reinforced.

The light scaling defects are very significant and observed on both precast and *in situ* concrete linings throughout the longitudinal section under the freeboard of the canals. But the severe scaling defect is observed more on the *in situ* concrete linings. This defect occurs in concrete that does not have proper air entrainment and is continuously saturated.

The pop-outs and spalling defects can be taken as acceptable defects and insignificant problems. These problems will not further continue and are observed rarely in a few places only. In most precast concrete canals, simple differential settlements are observed at the bed of the canal. But the *in situ* concrete may bridge over those settlements.

Properly placing, finishing, and curing concrete can have a big impact on service life. It is difficult to construct *in situ* concrete linings in the rainy season, because the rain may wash and erode the fresh concrete before its setting time. But in the case of precast concrete linings, the contractor can manufacture the precast elements in a rainy time in a protected room and assemble them during a dry time. Precast concrete linings are susceptible to the growth of grasses in the closely spaced joints. In the same manner, the cracks of *in situ* concrete are sensitive to the growth of grass and vegetation. The joints of precast concrete lining canals are scoured and removed.

To prevent the occurrences of cracks on *in situ* canal linings, tensile reinforcement should be provided on the side slopes. Otherwise, such larger canals should be lined by precast concretes having watertight stronger, and more durable joints. Air-entraining admixtures must be added to the concrete mix to prevent freezing and thawing damages since the scaling defects are caused due to improper air entrainment and continuous saturation. Sub-surface and interceptor drains must be regularly maintained to prevent settlement, due to internal erosion or excess hydrostatic load. From the visual inspection, evaluating the extent of the damage must be done to know how much concrete has been damaged or in other words how long, how wide, how deep, and how much of the structure or lining needs maintenance.

The concrete canal structures must be designed based on the intended exposure, loading conditions, and possible damage mechanisms. Different concrete mixes must be used, depending on the function of the concrete. The concrete strength must be designed based on exposure conditions. Because the needed strength depends on the early age exposure and loading conditions. Care should be taken during the design to prevent the uplift of the canal lining which can occur when expansive soils are present in the foundation which can cause large uplift pressures on the underside of the canal lining. These pressures can also cause the lining to buckle outwards or separate from adjacent linings.

The possible causes of defects must be determined to prevent further damage from the same cause. Since damage from one event, like a crack, is unlikely to occur again. Fixing the defects without analyzing what caused the problem means that the repair will be damaged in the same manner. Concrete must be protected until it reaches sufficient strength to avoid damage.

There must be observations and periodic inspections to predict the potential causes of concrete damage and to prolong the service life of the structures. As the structure gets older, all concrete cracks therefore, a periodic and routine maintenance schedule must be developed to manage concrete structures. Visual inspections and other non-destructive tests should be conducted regularly to know the extent of concrete defects. Concrete structures and canal linings are exposed to defects as they deliver water and result in damage to the concrete. Therefore, proper monitoring and routine maintenance or repair of concrete structures and canal linings are mandatory to keep structures in good condition. Defects must be identified and resolved early to prevent expensive repairs or replacements. Cracks in the concrete linings of the canal should be sealed to make them water-tight by different mechanisms like grouting with cementitious materials. Deterioration must be assessed early because early detection allows time for planning and budgeting for repairs.

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COMPETING INTERESTS

Authors have declared that no competing interests exist.

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